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Sapphire

Sapphire Hydro Limited

The Registrar
National Electric Power Regulatory Authority (NEPRA),
NEPRA Tower Attaturk Avenue (East),
Sector G-5/1, Islamabad.

Date: September 30, 2020
Reference: SHL/NEPRA/0086

Subject: **Application of Generation License for 152.12 MW Sharmai Hydro Power Project Located at Panjkora River, Upper Dir KPK.**

Pakhtunkhwa Energy Development Organization **("PEDO")** issued the Letter of Intent vide letter No. 595-603/PEDO/DPP/SECL dated 20/03/2017, for 152.12 MW Sharmai Hydro Power Project (hereinafter referred to as the **"Project"**) to the Consortium of Sapphire Electric Company Limited and Sino Hydro Corporation Limited (hereinafter referred to as the "**Consortium"**). As per clause 1 of the Letter of Intent, the Consortium has completed the feasibility study for the Project. PEDO's Panel of Experts approved the feasibility study for the project vide letter 162- 68/ PEDO/DRE/FS, dated 25th January 2019. Pursuant to the Issuance of LOI, Sapphire Hydro Limited was incorporated as a special purpose vehicle for the development of Sharmai Hydro Power Project.

As above, application for grant of Generation License to Sapphire Hydro Limited for its 152.12 MW Gross (150.6 MW Net) Sharmai Hydropower Project is hereby submitted before NEPRA pursuant to the Regulation of Generation, Transmission and Distribution of Electric Power Act, 1997. The application for grant of generation license has been prepared in accordance with Schedule-1 of the Application and Modification Procedure Regulations, 1999.

It is certified that the documents-in-support attached with this application are prepared and submitted in conformity with the provisions of the National Electric Power Regulatory Authority Regulations.

Demand Drafts in the sum of Rs. 934,270/- (Rupees: Nine Hundred Thirty Four Thousand Two Hundred Seventy Only) and Rs. 450/- (Rupees: Four Hundred Fifty Only) are being the non-refundable license application fee calculated in accordance with Schedule II to the National Electric Power Regulatory Authority Regulations.

Yours Truly,

Shahid Abdullah
Chief Executive Officer
Sapphire Hydro Limited

THE COMPANIES ACT, 2017 (XIX of 2017)

(COMPANY LIMITED BY SHARES)

MEMORANDUM OF ASSOCIATION

OF

SAPPHIRE HYDRO LIMITED

1. The name of the Company is **SAPPHIRE HYDRO LIMITED**.
2. The registered office of the Company will be situated in Province of Punjab.
3. The objects for which the Company is established are the following.

(i) The principal business of the company shall be to construct, establish and setup Hydro Electric power generation project or projects and to enter into lease agreements with any party including public sector organizations and government departments and to sell and distribute the electricity so produced in accordance with prevalent rules and policies and to perform all other acts which are necessary or incidental to the business of electricity generation, transmission, distribution and supply, including in the term electricity all power that may be directly or indirectly derived therefrom or may be incidentally hereafter discovered in dealing with electricity subject to permission required from NEPRA/other relevant authorities.

(ii) Except for the businesses mentioned in sub-clause (iii) hereunder, the company may engage in all the lawful businesses and shall be authorized to take all necessary steps and actions in connection therewith and ancillary thereto.

(iii) Notwithstanding anything contained in the foregoing sub-clauses of this clause nothing contained herein shall be construed as empowering the Company to undertake or indulge, directly or indirectly in the business of a Banking Company, Non-banking Finance Company (Mutual Fund, Leasing, Investment Company, Investment Advisor, Real Estate Investment Trust management company, Housing Finance Company, Venture Capital Company, Discounting Services, Microfinance or Microcredit business), Insurance Business, Modaraba management company, Stock Brokerage business, forex, real estate business, managing agency, business of providing the services of security guards or any other business restricted under any law for the time being in force or as may be specified by the Commission.

4. The liability of the members is limited.
5. The authorized share capital of the company is Rs. 500,000,000/- (Rupees Five Hundred Million) divided into 50,000,000 ordinary shares of Rs. 10/- each with powers to the company from time to time to increase and reduce its capital subject to any permission required under the law.

Sapphire Hydro Limited

We, the several persons whose names and addresses are subscribed below are desirous of being formed into a Company in pursuance of the Memorandum of Association and we respectively agree to take the number of shares in the capital of the Company as set opposite to our respective names:

	Name and surname (Present & former) in full (in block letters) and CNIC #	Father's/ Husband's name (in full)	Nationality with any former Nationality	Occupation	Residential address (in full)	Number of shares taken by each subscriber	Signatures
1	Sapphire Electric Company Limited.		Pakistani	Company	316 - Cotton Exchange Building, I.I. Chundrigar Road, Karachi.	1,999,970	
	Authorized Signatory on behalf of Subscriber Company						
	Mr. Shahid Abdullah (42201-5225618-1)	Mr. Muhammad Abdullah	Pakistani	Business	B – 31, KDA Scheme No. 1, Karachi		
	Nominees of Sapphire Electric Company Limited						
2	Mr. Shahid Abdullah (42201-5225618-1)	Mr. Muhammad Abdullah	Pakistani	Business	B – 31, KDA Scheme No. 1, Karachi	10	
3	Mr. Amer Abdullah (42201-2089751-3)	Mr. Muhammad Abdullah	Pakistani	Business	B – 31, KDA Scheme No. 1, Karachi	10	
4	Mr. Yousuf Abdullah (42201-5234263-1)	Mr. Muhammad Abdullah	Pakistani	Business	B – 31, KDA Scheme No. 1, Karachi	10	
					Total Number of shares taken	2,000,000 (Two Million Only)	

Dated this 18th Day of August, 2017

CERTIFIED TO BE TRUE COPY

21/9/2017

DEPUTY REGISTRAR OF COMPANIES
COMPANY REGISTRATION OFFICE
LAHORE.

Sapphire Hydro Limited

THE COMPANIES ACT, 2017

(PUBLIC COMPANY LIMITED BY SHARES)

Articles of Association

of

SAPPHIRE HYDRO LIMITED

PRELIMINARY

1. (1) In these regulations-

(a) "section" means section of the Act;

(b) "the Act" means the Companies Act, 2017; and

(c) "the seal" means the common seal or official seal of the company as the case may be.

(2) Unless the context otherwise requires, words or expressions contained in these regulations shall have the same meaning as in the Act; and words importing the singular shall include the plural, and vice versa, and words importing the masculine gender shall include feminine, and words importing persons shall include bodies corporate.

PUBLIC LIMITED COMPANY

2. The Company is a Public Company within the meaning of Clause (52) of Section 2(1) of the Companies Act, 2017.

BUSINESS

3. The directors shall have regard to the restrictions on the commencement of business imposed by section 19 if, and so far as, those restrictions are binding upon the company. The minimum subscription upon which the Directors may proceed to make the first allotment has been fixed as **Rs. 5,000,000/- (Rupees Five Million only).**

SHARES

4. In case of shares in the physical form, every person whose name is entered as a member in the member shall, without payment, be entitled to receive, within thirty days after

Sapphire Hydro Limited

allotment or within fifteen days of the application for registration of transfer, a certificate under the seal specifying the share or shares held by him and the amount paid up thereon:

Provided that if the shares are in book entry form or in case of conversion of physical shares and other transferable securities into book-entry form, the company shall, within ten days after an application is made for the registration of the transfer of any shares or other securities to a central depository, register such transfer in the name of the central depository.

5. The company shall not be bound to issue more than one certificate in respect of a share or shares in the physical form, held jointly by several persons and delivery of a certificate for a share to one of several joint holders shall be sufficient delivery to all.

6. If a share certificate in physical form is defaced, lost or destroyed, it may be renewed on payment of such fee, if any, not exceeding one hundred rupees, and on such terms, if any, as to evidence and indemnity and payment of expenses incurred by the company in investigating title as the directors think fit.

7. Except to the extent and in the manner allowed by section 86, no part of the funds of the company shall be employed in the purchase of, or in loans upon the security of, the company's shares.

TRANSFER AND TRANSMISSION OF SHARES

8. The instrument of transfer of any share in physical form in the company shall be executed both by the transferor and transferee, and the transferor shall be deemed to remain holder of the share until the name of the transferee is entered in the register of members in respect thereof.

9. Shares in physical form in the company shall be transferred in the following form, or in any usual or common form which the directors shall approve: -

Form for Transfer of Shares

(1st Schedule to the Companies Act, 2017)

I................. s/o ....r/o...................... (hereinafter called "the transferor") in consideration of the sum of rupees .... paid to me by............................ s/o ....r/o...................... (hereinafter called "the transferee"), do hereby transfer to the said transferee....................the share (or shares) with distinctive numbers from ....to..................inclusive, in the............................Limited, to hold unto the said transferee, his executors, administrators and assigns, subject to the several conditions on which I held the same at the time of the execution hereof, and I, the said transferee, do hereby agree to take the said share (or shares) subject to the conditions aforesaid.

As witness our hands this....................................day of.........................., 20...............

Signature....................................	Signature....................................
Transferor	**Transferee**
Full Name, Father's / Husband's Name, CNIC Number (in case of foreigner, Passport Number) Nationality Occupation and usual Residential Address	Full Name, Father's / Husband's Name CNIC Number (in case of foreigner, Passport Number) Nationality Occupation and usual Residential Address Cell Number Landline Number, if any Email Address
Witness 1:	**Witness 2:**
Signature Date Name, CNIC and Full Address	Signature Date Name, CNIC and Full Address

Bank Account Details of Transferee for Payment of Cash Dividend

It is requested that all my cash dividend amounts declared by the company, may be credited into the following bank account:

Title of Bank Account	
Bank Account #	
Bank's Name	
Branch Name And Address	

It is stated that the above mentioned information is correct and that I will intimate the changes in the above-mentioned information to the company and the concerned Share Registrar as soon as these occur.

....
Signature of the Transferee(s)

10. (1) Subject to the restrictions contained in regulation 10 and 11, the directors shall not refuse to transfer any share unless the transfer deed is defective or invalid. The directors may also suspend the registration of transfers during the ten days immediately preceding a general meeting or prior to the determination of entitlement or rights of the shareholders by giving seven days' previous notice in the manner provided in the Act. The directors may, in case of shares in physical form, decline to recognise any instrument of transfer unless—

a) a fee not exceeding fifty rupees as may be determined by the directors is paid to the company in respect thereof; and

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b) the duly stamped instrument of transfer is accompanied by the certificate of the shares to which it relates, and such other evidence as the directors may reasonably require to show the right of the transferor to make the transfer.

(2) If the directors refuse to register a transfer of shares, they shall within fifteen days after the date on which the transfer deed was lodged with the company send to the transferee and the transferor notice of the refusal indicating the defect or invalidity to the transferee, who shall, after removal of such defect or invalidity be entitled to re-lodge the transfer deed with the company.

Provided that the company shall, where the transferee is a central depository the refusal shall be conveyed within five days from the date on which the instrument of transfer was lodged with it notify the defect or invalidity to the transferee who shall, after the removal of such defect or invalidity, be entitled to re-lodge the transfer deed with the company.

TRANSMISSION OF SHARES

11. The executors, administrators, heirs, or nominees, as the case may be, of a deceased sole holder of a share shall be the only persons recognised by the company to deal with the share in accordance with the law. In the case of a share registered in the names of two or more holders, the survivors or survivor, or the executors or administrators of the deceased survivor, shall be the only persons recognised by the company to deal with the share in accordance with the law.

12. The shares or other securities of a deceased member shall be transferred on application duly supported by succession certificate or by lawful award, as the case may be, in favour of the successors to the extent of their interests and their names shall be entered to the register of members.

13. A person may on acquiring interest in a company as member, represented by shares, at any time after acquisition of such interest deposit with the company a nomination conferring on a person, being the relatives of the member, namely, a spouse, father, mother, brother, sister and son or daughter, the right to protect the interest of the legal heirs in the shares of the deceased in the event of his death, as a trustee and to facilitate the transfer of shares to the legal heirs of the deceased subject to succession to be determined under the Islamic law of inheritance and in case of non-Muslim members, as per their respective law.

14. The person nominated under regulation 12 shall, after the death of the member, be deemed as a member of company till the shares are transferred to the legal heirs and if the deceased was a director of the company, not being a listed company, the nominee shall also act as director of the company to protect the interest of the legal heirs.

15. A person deemed as a member under regulation 11, 12 and 13 to a share by reason of the death or insolvency of the holder shall be entitled to the same dividends and other advantages to which he would be entitled if he were the registered holder of the share and exercise any right conferred by membership in relation to meetings of the company.

ALTERATION OF CAPITAL

b) the duly stamped instrument of transfer is accompanied by the certificate of the shares to which it relates, and such other evidence as the directors may reasonably require to show the right of the transferor to make the transfer.

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ALTERATION OF CAPITAL

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16. The company may, by special resolution-

(e) increase its authorised capital by such amount as it thinks expedient;

(f) consolidate and divide the whole or any part of its share capital into shares of larger amount than its existing shares;

(g) sub-divide its shares, or any of them, into shares of smaller amount than is fixed by the memorandum;

(h) cancel shares which, at the date of the passing of the resolution in that behalf, have not been taken or agreed to be taken by any person, and diminish the amount of its share capital by the amount of the share so cancelled.

17. Subject to the provisions of the Act, all new shares shall at the first instance be offered to such persons as at the date of the offer are entitled to such issue in proportion, as nearly as the circumstances admit, to the amount of the existing shares to which they are entitled. The offer shall be made by letter of offer specifying the number of shares offered, and limiting a time within which the offer, if not accepted, will deem to be declined, and after the expiration of that time, or on the receipt of an intimation from the person to whom the offer is made that he declines to accept the shares offered, the directors may dispose of the same in such manner as they think most beneficial to the company. The directors may likewise so dispose of any new shares which (by reason of the ratio which the new shares bear to shares held by persons entitled to an offer of new shares) cannot, in the opinion of the directors, be conveniently offered under this regulation.

18. The new shares shall be subject to the same provisions with reference to transfer, transmission and otherwise as the shares in the original share capital.

19. The company may, by special resolution-

(a) consolidate and divide its share capital into shares of larger amount than its existing shares;

(b) sub-divide its existing shares or any of them into shares of smaller amount than is fixed by the memorandum of association, subject, nevertheless, to the provisions of section 85;

(c) cancel any shares which, at the date of the passing of the resolution, have not been taken or agreed to be taken by any person.

20. The company may, by special resolution, reduce its share capital in any manner and with, and subject to confirmation by the Court and any incident authorised and consent required, by law.

GENERAL MEETINGS

21. The statutory general meeting of the company shall be held within the period required by section 131.

22. A general meeting, to be called annual general meeting, shall be held, in accordance with the provisions of section 132, within sixteen months from the date of incorporation of the company and thereafter once at least in every year within a period of one hundred and twenty days following the close of its financial year.

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23. All general meetings of a company other than the statutory meeting or an annual general meeting mentioned in sections 131 and 132 respectively shall be called extraordinary general meetings.

24. The directors may, whenever they think fit, call an extraordinary general meeting, and extraordinary general meetings shall also be called on such requisition, or in default, may be called by such requisitionists, as provided by section 133. If at any time there are not within Pakistan sufficient directors capable of acting to form a quorum, any director of the company may call an extraordinary general meeting in the same manner as nearly as possible as that in which meetings may be called by the directors.

25. The company may provide video-link facility to its members for attending general meeting at places other than the town in which general meeting is taking place after considering the geographical dispersal of its members:

Provided that in case of listed companies if the members holding ten percent of the total paid up capital or such other percentage of the paid up capital as may be specified, are resident in any other city, the company shall provide the facility of video-link to such members for attending annual general meeting of the company, if so required by such members in writing to the company at least seven days before the date of the meeting.

NOTICE AND PROCEEDINGS OF GENERAL MEETINGS

26. Twenty-one days' notice at the least (exclusive of the day on which the notice is served or deemed to be served, but inclusive of the day for which notice is given) specifying the place, the day and the hour of meeting and, in case of special business, the general nature of that business, shall be given in manner provided by the Act for the general meeting, to such persons as are, under the Act or the regulations of the company, entitled to receive such notice from the company; but the accidental omission to give notice to, or the non-receipt of notice by, any member shall not invalidate the proceedings at any general meeting.

27. All the business transacted at a general meeting shall be deemed special other than the business stated in sub-section (2) of section 134 namely; the consideration of financial statements and the reports of the board and auditors, the declaration of any dividend, the election and appointment of directors in place of those retiring, and the appointment of the auditors and fixing of their remuneration.

28. No business shall be transacted at any general meeting unless a quorum of members is present at that time when the meeting proceeds to business. The quorum of the general meeting shall be-

(a) in the case of a public listed company, not less than ten members present personally, or through video-link who represent not less than twenty-five percent of the total voting power, either of their own account or as proxies;

(b) in the case of any other company having share capital, two members present personally, or through video-link who represent not less than twenty-five percent of the total voting power, either of their own account or as proxies.

29. If within half an hour from the time appointed for the meeting a quorum is not present, the meeting, if called upon the requisition of members, shall be dissolved; in any other case, it

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shall stand adjourned to the same day in the next week at the same time and place, and, if at the adjourned meeting a quorum is not present within half an hour from the time appointed for the meeting, the members present, being not less than two, shall be a quorum.

30. The chairman of the board of directors, if any, shall preside as chairman at every general meeting of the company, but if there is no such chairman, or if at any meeting he is not present within fifteen minutes after the time appointed for the meeting, or is unwilling to act as chairman, any one of the directors present may be elected to be chairman, and if none of the directors is present, or willing to act as chairman, the members present shall choose one of their number to be chairman.

31. The chairman may, with the consent of any meeting at which a quorum is present (and shall if so directed by the meeting), adjourn the meeting from time to time but no business shall be transacted at any adjourned meeting other than the business left unfinished at the meeting from which the adjournment took place. When a meeting is adjourned for fifteen days or more, notice of the adjourned meeting shall be given as in the case of an original meeting. Save as aforesaid, it shall not be necessary to give any notice of an adjournment or of the business to be transacted at an adjourned meeting.

32. (1) At any general meeting a resolution put to the vote of the meeting shall be decided on a show of hands unless a poll is (before or on the declaration of the result of the show of hands) demanded. Unless a poll is so demanded, a declaration by the chairman that a resolution has, on a show of hands, been carried, or carried unanimously, or by a particular majority, or lost, and an entry to that effect in the book of the proceedings of the company shall be conclusive evidence of the fact, without proof of the number or proportion of the votes recorded in favour of, or against, that resolution.

(2) At any general meeting, the company shall transact such businesses as may be notified by the Commission, only through postal ballot.

33. A poll may be demanded only in accordance with the provisions of section 143.

34. If a poll is duly demanded, it shall be taken in accordance with the manner laid down in sections 144 and 145 and the result of the poll shall be deemed to be the resolution of the meeting at which the poll was demanded.

35. A poll demanded on the election of chairman or on a question of adjournment shall be taken at once.

36. In the case of an equality of votes, whether on a show of hands or on a poll, the chairman of the meeting at which the show of hands takes place, or at which the poll is demanded, shall have and exercise a second or casting vote.

37. Except for the businesses specified under sub-section (2) of section 134 to be conducted in the annual general meeting, the members of a private company or a public unlisted company (having not more than fifty members), may pass a resolution (ordinary or special) by circulation signed by all the members for the time being entitled to receive notice of a meeting. The resolution by circulation shall be deemed to be passed on the date of signing by the last of the signatory member to such resolution.

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VOTES OF MEMBERS

38. Subject to any rights or restrictions for the time being attached to any class or classes of shares, on a show of hands every member present in person shall have one vote except for election of directors in which case the provisions of section 159 shall apply. On a poll every member shall have voting rights as laid down in section 134.

39. In case of joint-holders, the vote of the senior who tenders a vote, whether in person or by proxy or through video-link shall be accepted to the exclusion of the votes of the other joint-holders; and for this purpose seniority shall be determined by the order in which the names stand in the register of members.

40. A member of unsound mind, or in respect of whom an order has been made by any court having jurisdiction in lunacy, may vote, whether on show of hands or on a poll or through video link, by his committee or other legal guardian, and any such committee or guardian may, on a poll, vote by proxy.

41. On a poll votes may be given either personally or through video-link, by proxy or through postal ballot:

Provided that no body corporate shall vote by proxy as long as a resolution of its directors in accordance with the provisions of section 138 is in force.

42. (1) The instrument appointing a proxy shall be in writing under the hand of the appointer or of his attorney duly authorised in writing.

(2) The instrument appointing a proxy and the power-of-attorney or other authority (if any) under which it is signed, or a notarially certified copy of that power or authority, shall be deposited at the registered office of the company not less than forty-eight hours before the time for holding the meeting at which the person named in the instrument proposes to vote and in default the instrument of proxy shall not be treated as valid.

43. An instrument appointing a proxy may be in the following form, or a form as near thereto as may be:

INSTRUMENT OF PROXY

.... Limited

"I .... s/o .... r/o .... being a member of the .... Limited, hereby appoint .... s/o .... r/o .... as my proxy to attend and vote on my behalf at the (statutory, annual, extraordinary, as the case may be) general meeting of the company to be held on the .... day of ...., 20...... and at any adjournment thereof."

Sapphire Hydro Limited

44. A vote given in accordance with the terms of an instrument of proxy shall be valid notwithstanding the previous death or insanity of the principal or revocation of the proxy or of the authority under which the proxy was executed, or the transfer of the share in respect of which the proxy is given, provided that no intimation in writing of such death, insanity, revocation or transfer as aforesaid shall have been received by the company at the office before the commencement of the meeting or adjourned meeting at which the proxy is used.

DIRECTORS

45. The following subscribers of the memorandum of association shall be the first directors of the company, so, however, that the number of directors shall not in any case be less than that specified in section 154 and they shall hold office until the election of directors in the first annual general meeting. Following shall be the first directors of the company:

1. Mr. Shahid Abdullah
2. Mr. Amer Abdullah
3. Mr. Yousuf Abdullah

46. The remuneration of the directors shall from time to time be determined by the company in general meeting subject to the provisions of the Act.

47. Save as provided in section 153, no person shall be appointed as a director unless he is a member of the company.

POWERS AND DUTIES OF DIRECTORS

48. The business of the company shall be managed by the directors, who may pay all expenses incurred in promoting and registering the company, and may exercise all such powers of the company as are not by the Act or any statutory modification thereof for the time being in force, or by these regulations, required to be exercised by the company in general meeting, subject nevertheless to the provisions of the Act or to any of these regulations, and such regulations being not inconsistent with the aforesaid provisions, as may be prescribed by the company in general meeting but no regulation made by the company in general meeting shall invalidate any prior act of the directors which would have been valid if that regulation had not been made.

49. The directors shall appoint a chief executive in accordance with the provisions of sections 186 and 187.

50. The amount for the time being remaining undischarged of moneys borrowed or raised by the directors for the purposes of the company (otherwise than by the issue of share capital) shall not at any time, without the sanction of the company in general meeting, exceed the issued share capital of the company.

51. The directors shall duly comply with the provisions of the Act, or any statutory modification thereof for the time being in force, and in particular with the provisions in regard to the registration of the particulars of mortgages, charges and pledge affecting the property of the company or created by it, to the keeping of a register of the directors, and to the sending to the registrar of an annual list of members, and a summary of particulars relating thereto and notice of any consolidation or increase of share capital, or sub-division of shares, and copies of special resolutions and a copy of the register of directors and notifications of any changes therein.

MINUTE BOOKS

52. The directors shall cause records to be kept and minutes to be made in book or books with regard to-

(a) all resolutions and proceedings of general meeting(s) and the meeting(s) of directors and Committee(s) of directors, and every member present at any general meeting and every director present at any meeting of directors or Committee of directors shall put his signature in a book to be kept for that purpose;

(b) recording the names of the persons present at each meeting of the directors and of any committee of the directors, and the general meeting; and

(c) all orders made by the directors and Committee(s) of directors:

Provided that all records related to proceedings through video-link shall be maintained in accordance with the relevant regulations specified by the Commission which shall be appropriately rendered into writing as part of the minute books according to the said regulations.

THE SEAL

53. The directors shall provide for the safe custody of the seal and the seal shall not be affixed to any instrument except by the authority of a resolution of the board of directors or by a committee of directors authorized in that behalf by the directors and in the presence of at least two directors and of the secretary or such other person as the directors may appoint for the purpose; and those two directors and secretary or other person as aforesaid shall sign every instrument to which the seal of the company is so affixed in their presence.

DISQUALIFICATION OF DIRECTORS

54. No person shall become the director of a company if he suffers from any of the disabilities or disqualifications mentioned in section 153 or disqualified or debarred from holding such office under any of the provisions of the Act as the case may be and, if already a director, shall cease to hold such office from the date he so becomes disqualified or disabled:

Sapphire Hydro Limited

Provided, however, that no director shall vacate his office by reason only of his being a member of any company which has entered into contracts with, or done any work for, the company of which he is director, but such director shall not vote in respect of any such contract or work, and if he does so vote, his vote shall not be counted.

PROCEEDINGS OF DIRECTORS

55. The directors may meet together for the dispatch of business, adjourn and otherwise regulate their meetings, as they think fit. A director may, and the secretary on the requisition of a director shall, at any time, summon a meeting of directors. Notice sent to a director through email whether such director is in Pakistan or outside Pakistan shall be a valid notice.

56. The directors may elect a chairman of their meetings and determine the period for which he is to hold office; but, if no such chairman is elected, or if at any meeting the chairman is not present within ten minutes after the time appointed for holding the same or is unwilling to act as chairman, the directors present may choose one of their number to be chairman of the meeting.

57. At least one-third (1/3rd) of the total number of directors or two (2) directors whichever is higher, for the time being of the company, present personally or through video-link, shall constitute a quorum.

58. Save as otherwise expressly provided in the Act, every question at meetings of the board shall be determined by a majority of votes of the directors present in person or through video-link, each director having one vote. In case of an equality of votes or tie, the chairman shall have a casting vote in addition to his original vote as a director.

59. The directors may delegate any of their powers not required to be exercised in their meeting to committees consisting of such member or members of their body as they think fit; any committee so formed shall, in the exercise of the powers so delegated, conform to any restrictions that may be imposed on them by the directors.

60. (1) A committee may elect a chairman of its meetings; but, if no such chairman is elected, or if at any meeting the chairman is not present within ten minutes after the time appointed for holding the same or is unwilling to act as chairman, the members present may choose one of their number to be chairman of the meeting.

(2) A committee may meet and adjourn as it thinks proper. Questions arising at any meeting shall be determined by a majority of votes of the members present. In case of an equality of votes, the chairman shall have and exercise a second or casting vote.

61. All acts done by any meeting of the directors or of a committee of directors, or by any person acting as a director, shall, notwithstanding that it be afterwards discovered that there was some defect in the appointment of any such directors or persons acting as aforesaid, or

Sapphire Hydro Limited

that they or any of them were disqualified, be as valid as if every such person had been duly appointed and was qualified to be a director.

62. A copy of the draft minutes of meeting of the board of directors shall be furnished to every director within seven working days of the date of meeting.

63. A resolution in writing signed by all the directors for the time being entitled to receive notice of a meeting of the directors shall be as valid and effectual as if it had been passed at a meeting of the directors duly convened and held.

FILLING OF VACANCIES

64. At the first annual general meeting of the company, all the directors shall stand retired from office, and directors shall be elected in their place in accordance with section 159 for a term of three years.

65. A retiring director shall be eligible for re-election.

66. The directors shall comply with the provisions of sections 154 to 159 and sections 161, 162 and 167 relating to the election of directors and matters ancillary thereto.

67. Any casual vacancy occurring on the board of directors may be filled up by the directors, but the person so chosen shall be subject to retirement at the same time as if he had become a director on the day on which the director in whose place he is chosen was last elected as director.

68. The company may remove a director but only in accordance with the provisions of the Act.

DIVIDENDS AND RESERVE

69. The company in general meeting may declare dividends but no dividend shall exceed the amount recommended by the directors.

70. The directors may from time to time pay to the members such interim dividends as appear to the directors to be justified by the profits of the company.

71. Any dividend may be paid by a company either in cash or in kind only out of its profits. The payment of dividend in kind shall only be in the shape of shares of listed company held by the distributing company.

72. Dividend shall not be paid out of unrealized gain on investment property credited to profit and loss account.

73. Subject to the rights of persons (if any) entitled to shares with special rights as to dividends, all dividends shall be declared and paid according to the amounts paid on the shares.

Sapphire Hydro Limited

74. (1) The directors may, before recommending any dividend, set aside out of the profits of the company such sums as they think proper as a reserve or reserves which shall, at the discretion of the directors, be applicable for meeting contingencies, or for equalizing dividends, or for any other purpose to which the profits of the company may be properly applied, and pending such application may, at the like discretion, either be employed in the business of company or be invested in such investments (other than shares of the company) as the directors may, subject to the provisions of the Act, from time to time think fit.

(2) The directors may carry forward any profits which they may think prudent not to distribute, without setting them aside as a reserve.

75. If several persons are registered as joint-holders of any share, any one of them may give effectual receipt for any dividend payable on the share.

76. (1) Notice of any dividend that may have been declared shall be given in manner hereinafter mentioned to the persons entitled to share therein but, in the case of a public company, the company may give such notice by advertisement in a newspaper circulating in the Province in which the registered office of the company is situate.

(2) Any dividend declared by the company shall be paid to its registered shareholders or to their order. The dividend payable in cash may be paid by cheque or warrant or in any electronic mode to the shareholders entitled to the payment of the dividend, as per their direction.

(3) In case of a listed company, any dividend payable in cash shall only be paid through electronic mode directly into the bank account designated by the entitled shareholders.

77. The dividend shall be paid within the period laid down under the Act.

ACCOUNTS

78. The directors shall cause to be kept proper books of account as required under section 220.

79. The books of account shall be kept at the registered office of the company or at such other place as the directors shall think fit and shall be open to inspection by the directors during business hours.

80. The directors shall from time to time determine whether and to what extent and at what time and places and under what conditions or regulations the accounts and books or papers of the company or any of them shall be open to the inspection of members not being directors, and no member (not being a director) shall have any right of inspecting any account and book or papers of the company except as conferred by law or authorised by the directors or by the company in general meeting.

Sapphire Hydro Limited

81. The directors shall as required by sections 223 and 226 cause to be prepared and to be laid before the company in general meeting the financial statements duly audited and reports as are referred to in those sections.

82. The financial statements and other reports referred to in regulation 80 shall be made out in every year and laid before the company in the annual general meeting in accordance with sections 132 and 223.

83. A copy of the financial statements and reports of directors and auditors shall, at least twenty-one days preceding the meeting, be sent to the persons entitled to receive notices of general meetings in the manner in which notices are to be given hereunder.

84. The directors shall in all respect comply with the provisions of sections 220 to 227.

85. Auditors shall be appointed and their duties regulated in accordance with sections 246 to 249.

NOTICES

86. (1) A notice may be given by the company to any member to his registered address or if he has no registered address in Pakistan to the address, if any, supplied by him to the company for the giving of notices to him against an acknowledgement or by post or courier service or through electronic means or in any other manner as may be specified by the Commission.

(2) Where a notice is sent by post, service of the notice shall be deemed to be effected by properly addressing, prepaying and posting a letter containing the notice and, unless the contrary is proved, to have been effected at the time at which the letter will be delivered in the ordinary course of post.

87. A notice may be given by the company to the joint-holders of a share by giving the notice to the joint-holder named first in the register in respect of the share.

88. A notice may be given by the company to the person entitled to a share in consequence of the death or insolvency of a member in the manner provided under regulation 85 addressed to them by name, or by the title or representatives of the deceased, or assignees of the insolvent, or by any like description, at the address, supplied for the purpose by the person claiming to be so entitled.

89. Notice of every general meeting shall be given in the manner hereinbefore authorised to (a) every member of the company and also to (b) every person entitled to a share in consequence of the death or insolvency of a member, who but for his death or insolvency would be entitled to receive notice of the meeting, and (c) to the auditors of the company for the time being and every person who is entitled to receive notice of general meetings.

Sapphire Hydro Limited

WINDING UP

90. (1) In the case of members' voluntary winding up, with the sanction of a special resolution of the company, and, in the case of creditors' voluntary winding up, of a meeting of the creditors, the liquidator shall exercise any of the powers given by sub-section (1) of section 337 of the Act to a liquidator in a winding up by the Court including *inter-alia* divide amongst the members, in specie or kind, the whole or any part of the assets of the company, whether they consist of property of the same kind or not.

(2) For the purpose aforesaid, the liquidator may set such value as he deems fair upon any property to be divided as aforesaid and may determine how such division shall be carried out as between the members or different classes of members.

(3) The liquidator may, with the like sanction, vest the whole or any part of such assets in trustees upon such trusts for the benefit of the contributories as the liquidator, with the like sanction, thinks fit, but so that no member shall be compelled to accept any shares or other securities whereon there is any liability.

INDEMNITY

91. Every officer or agent for the time being of the company may be indemnified out of the assets of the company against any liability incurred by him in defending any proceedings, whether civil or criminal, arising out of his dealings in relation to the affairs of the company, except those brought by the company against him, in which judgment is given in his favour or in which he is acquitted, or in connection with any application under section 492 in which relief is granted to him by the Court.

Sapphire Hydro Limited

We, the several persons whose names and addresses are subscribed below are desirous of being formed into a Company in pursuance of the Articles of Association and we respectively agree to take the number of shares in the capital of the Company as set opposite to our respective names:

	Name and surname (Present & former) in full (in block letters) and CNIC #	Father's/ Husband's name (in full)	Nationality with any former Nationality	Occupation	Residential address (in full)	Number of shares taken by each subscriber	Signatures
1	Sapphire Electric Company Limited.		Pakistani	Company	316 - Cotton Exchange Building, I.I. Chundrigar Road, Karachi.	1,999,970	
	Authorized Signatory on behalf of Subscriber Company						
	Mr. Shahid Abdullah (42201-5225618-1)	Mr. Muhammad Abdullah	Pakistani	Business	B – 31, KDA Scheme No. 1, Karachi		
	Nominees of Sapphire Electric Company Limited						
2	Mr. Shahid Abdullah (42201-5225618-1)	Mr. Muhammad Abdullah	Pakistani	Business	B – 31, KDA Scheme No. 1, Karachi	10	
3	Mr. Amer Abdullah (42201-2089751-3)	Mr. Muhammad Abdullah	Pakistani	Business	B – 31, KDA Scheme No. 1, Karachi	10	
4	Mr. Yousuf Abdullah (42201-5234263-1)	Mr. Muhammad Abdullah	Pakistani	Business	B – 31, KDA Scheme No. 1, Karachi	10	
					Total Number of shares taken	2,000,000 (Two Million Only)	

Dated this 18th Day of August, 2017

CERTIFIED TO BE TRUE COPY
7/9/2017
DEPUTY REGISTRAR OF COMPANIES
COMPANY REGISTRATION OFFICE
LAHORE.

Sapphire Hydro Limited

152.12 MW Sapphire Hydro Power Project Prospectus

Profile of the Sponsor

Sapphire is one of the largest conglomerates in the country with investment in textile, dairy and the power sector. Sapphire Group started its first production facility in Pakistan in 1971 in Textile Spinning. In the coming decades it continued to broaden its expanse in the value chain. A decade later Sapphire further forayed into Home Textiles Sewing units (2002), Finishing Plant (2003), Knit Stitching (2004) and Work Wear Apparel (2007). In 2007, Sapphire Group took the initiative of diversifying into "foods". Its dairy farm was started in 2007. Sapphire plans to expand its production and eventually move into the value-added products as well. Sapphire successfully erected 234MW combined-cycle power plant in 2010 which is now assisting WAPDA. Recently, Sapphire has added 150 MW to wind power generation in addition to 50 MW wind power plant already operational in 2015.

Salient Features of the Project

152.12 MW Gross Installed Capacity (Net Capacity of 150.6 MW) Sapphire Hydro Power Limited is a milestone project for Sapphire Group located on the Panjkora River in Upper Dir Valley. The Project offers extensive positive externalities apart from reducing the Energy Demand Supply gap. The Salient Features of the Project are mentioned below:

Technical Information:

• Gross Capacity	152.12 MW
• Net Capacity	150.6 MW
• Gross Annual Generation	696.81 GWH
• Auxiliary Consumption	6.97 GWH
• Expected Annual Generation	689.84 GWH
• Design Discharge	90 Cumecs
• Plant Factor	52.29%
• Construction Period	4.5 years
• Investment Model:	Build, Own, Operate and Transfer (BOOT)
• Funding Sources:	International DFI's Chinese CPEC Funding *Consortium* of Commercial Banks's
• Power Purchaser:	Central Power Purchasing Authority
• Tariff approved by:	National Electric Power Regulatory Authority

Proposed Investment:

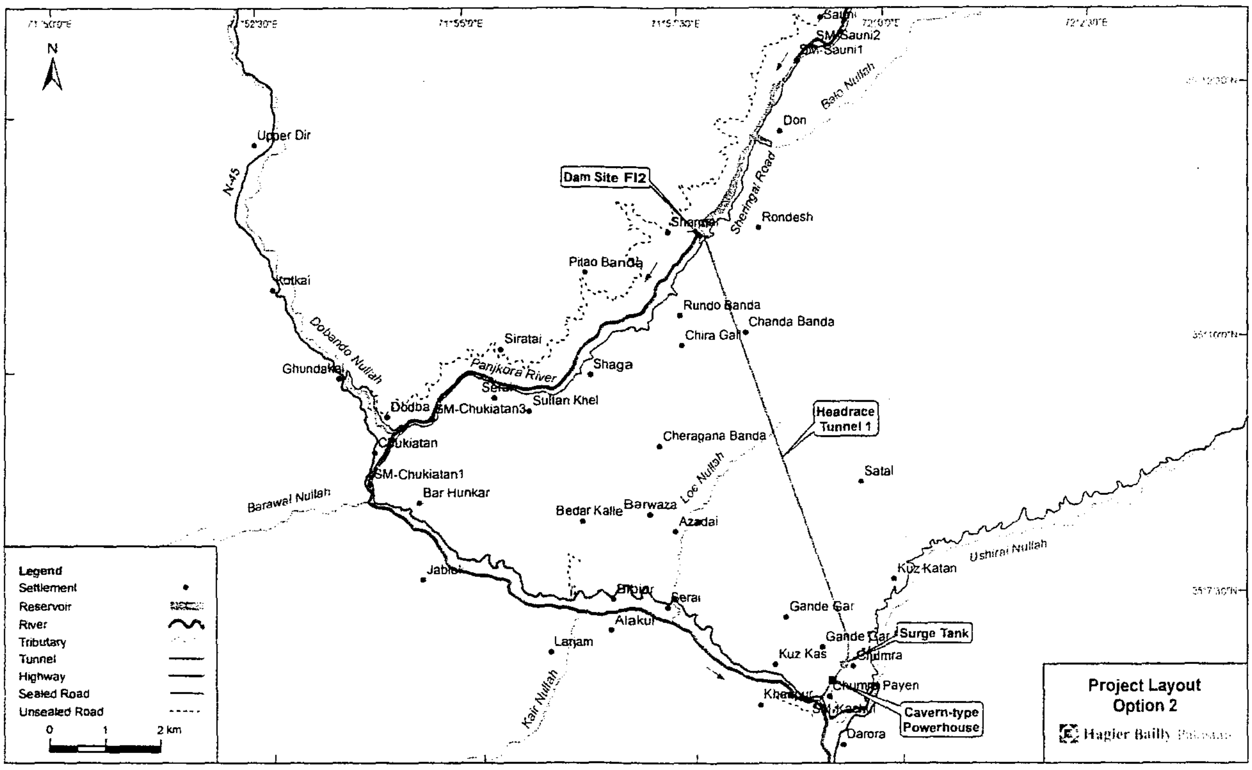
The Total Cost of the Project is estimated at ***USD 400.778 million*** at 2.635 million USD per MW

Sapphire Hydro Limited

Part B – Schedule 3A(C)1

1) Location

SHARMAI Hydropower Project is located on the Panjkora River near Dir with location coordinates 35° 11'18.30"N, 71°57'45.16"E.



2) Plant Type (Run of River, Storage, Weir etc)

Sharmai Hydro Power Project is a Run of the River Hydro Power Project.

3) Head: Minimum and Maximum

The gross head at maximum Discharge level is 200.8 while the minimum gross head is 195.8

4) Technology: Francis Pelton, size and no of units

3 x Vertical Francis Turbines of 50.7 MW each. = 152.12 MW (Gross Installed Capacity)

5) Tunnel if Proposed:

Length and Diameter: The tunnel is 8.5 Km Long and has an internal dia of 6.75m.

Sapphire Hydro Limited

6) **ESSA:**

The Detailed environment and social impact assessment study has been conducted by a renowned company "Hagler Bailey, Pakistan".

- The study has been conducted in-line with the EPA – KPK IFC and ABD guidelines.
- The environment report has been submitted to EPA-KPK on January 18 , 2019 (Submitted Letter Annexed as "Annexure - 1")
- The Public hearing has been conducted on September 04, 2019.
- The Non-technical summary along with the Executive summary of Environmental report is Annexed as "Annexure 4"
- The report is at the final stages of approval with KPK-EPA.

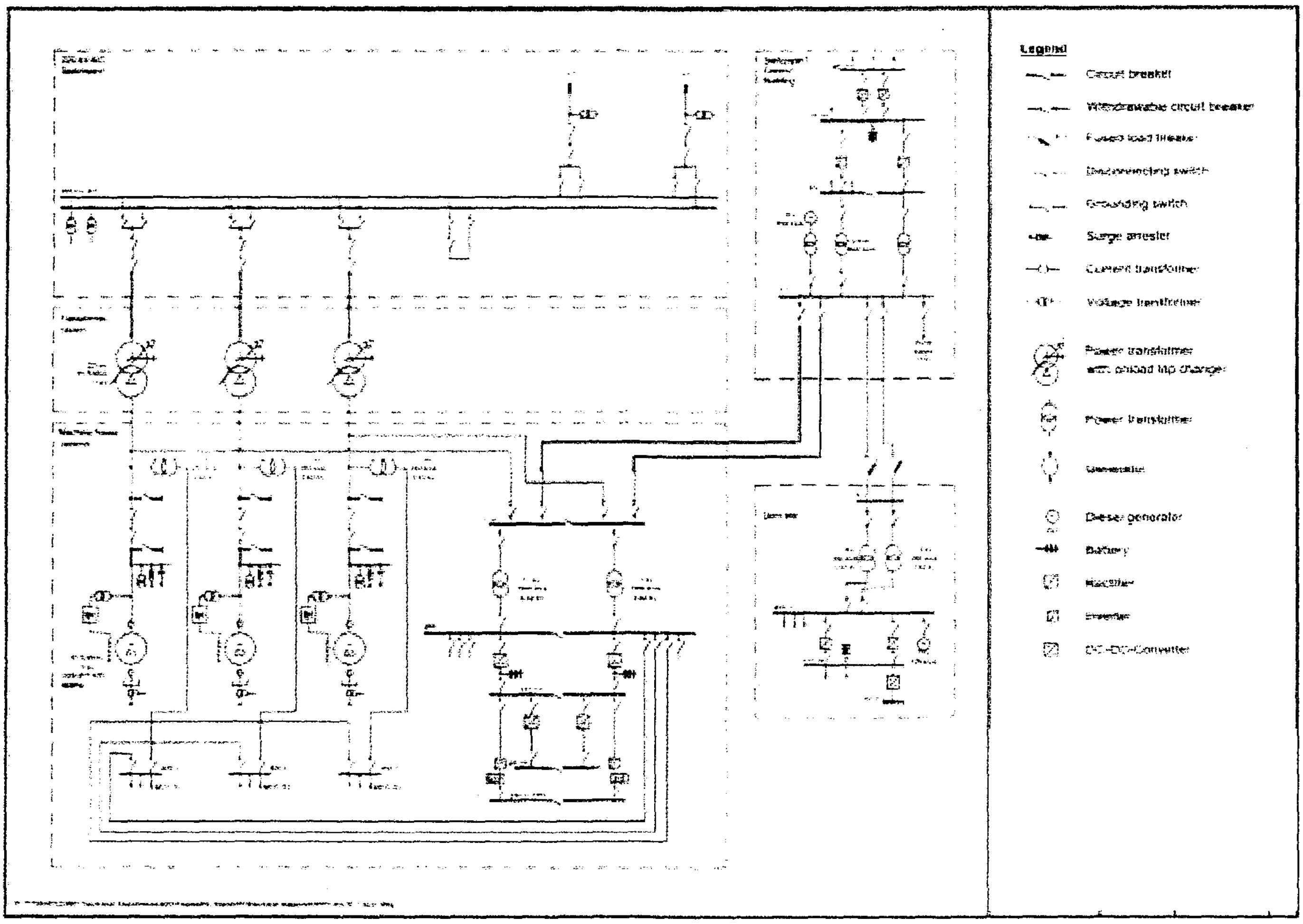
7) **Resettlement Issues:** Lands in Sharmai Hydropower Project area comprise low cultivated lands, and settlements and larger area is unoccupied. The project will consume 112 hectares of land, of which only 6.2 hectares are under cultivation whilst the remaining land is barren or unoccupied. The Project will have an overall beneficial impact in the region and the construction of the project will lead to a boost in regional economic activity through creation of multiple direct/indirect opportunities. The resettlement is only 5 will lose residence. No mosques, graves and sties of archaeological and historical importance will be affected due to implementation of proposed project.

8) **Consents: (Attached as "Annexure 2"**

PESCO NOC
EPA-KPA NOC shall be provided Shortly

Sapphire Hydro Limited

9) Infrastructure Development – SLD – Attached Below



Dam:

Dam would be a concrete dam with integrated gated spillway with height of 45 m. The dam shall have top width of 8m at the elevation 1265 masl and a vertical upstream face. Dam shall be founded on sound rock. Dam width in the crest is about 150 m.

Power house;

Powerhouse cavern shall accommodate 3 Francis vertical units. Main cavern shall have the following dimensions: W x H x L = 20 x 30 x 60 m, all subject to possible small scale adjustments related to the final equipment size. The cavern will have the assembling bay and the control rooms. There will be a separate cavern for transformers. The caverns will be accessible by an access tunnel from the main N-45 road. Besides access tunnel, there will be separate cable and aeration tunnels, connecting the powerhouse cavern to the switchyard.

Tunnel:

Power tunnel shall be 8.45 km long with 6.75 m inner diameter. It will be excavated by drill-and-blast technique, from both sides. Possibly and subject to the contractor's work plan, an intermediate adit at the tunnel chainage 7.800 km may need to be considered

10) Interconnection with National grid – Length + SLD

As per the GIS attached as "Annexure 3".

11) Project Cost Information regarding sources and amount of Debt and Equity

The total cost of the project is 400.778 Million USD of which 80.156 Million USD (20%) will be equity arranged by the Sponsors, while the remaining 320.622 Million USD (80%) will be in Debt arranged through multilateral lenders.

12) Project Schedule and Expected Life

The project being developed as an IPP under KPK power Policy, 2016 , Concession period is of 30 Years and project expected life is more than 50 year. The project schedule is provided in attachment as part of the feasibility study.

13) Operation Type

Run of river project. It will be a baseload operation during high flow season and can by a peaking during low flow season.

14) Plant characteristics: generation voltage, power factor, frequency, automatic generation control, ramping rate, control metering and instrumentation. The details is provided in Draft GIS Study

Generation Voltage:	**220kv**
Power Factor:	**0.8 lagging**
Frequency:	**50 hertz + 3%**
Metering Systems:	Using fully static energy meters and energy recorders will be provided. For the metering and billing of each of the outgoing lin one main and one back-up energy meter with minimum accurac 0.5 will be foreseen.
Instrumentation:	**As per Feasibility Study**

15) System studies load flow, short circuit, stability

Attached as "Annexure 3 "of Draft GIS.
PESCO consent is attached as Annexure.

Sapphire Hydro Limited

16) Training and Development:

The Training and development and will be provided to the employees, workers. Training has been made part of the environmental report.

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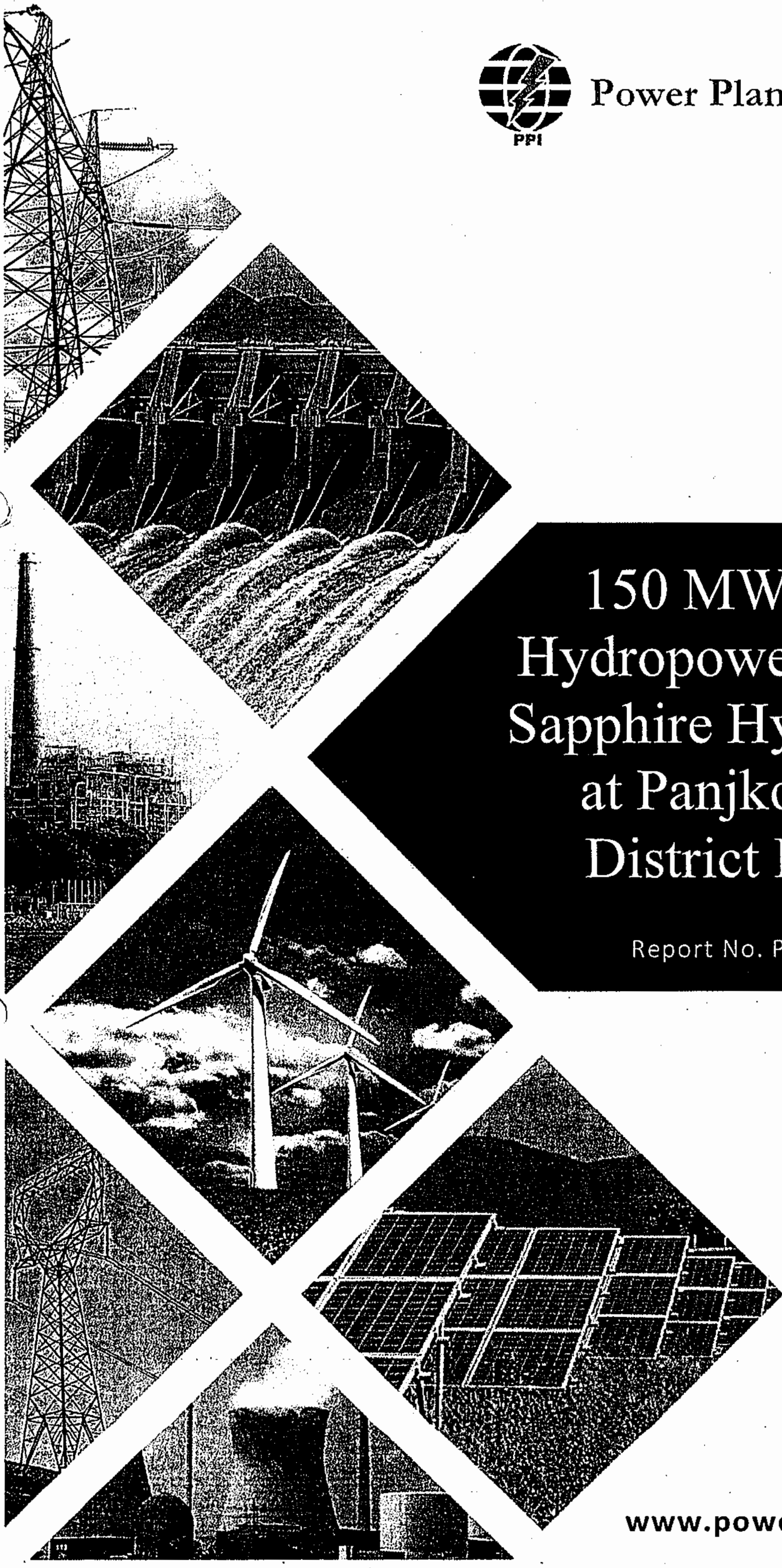

150 MW Sharmai Hydropower Project by Sapphire Hydro Limited at Panjkora River, District Dir, KPK

Report No. PPI-375.1-Draft/20

www.powerplannersint.com

Interconnection Study of 150 MW Sharmai Hydropower Project

By

Sapphire Hydro- Limited at Panjkora River, District Dir, Khyber Pukhtunkhwa

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Version	Date	Authors	Checked By	Comments
1	31/08/2020	Ahmad Usman Khan	Ayesha Haroon	

Executive Summary

- The Draft Report of 150 MW Sharmai Hydro Power Project by Sapphire Hydro Limited at District Dir, Khyber Pakhtunkhwa, referred to as Sharmai HPP in the remainder of the report, is submitted herewith.
- The updated transmission plan and load forecast from PESCO has been used for the study, vide data permission letter no. CE (Dev) 5100-04 dated 20/07/2020.
- The study objective, approach and methodology have been described and the plant's data received from the Client is validated.
- Two options were investigated for the interconnection of this project. One possible interconnection scheme is to connect the plant to 132 kV Chakdara-New Grid Station. Another possible point of interconnection is to connect the said plant to 220 kV Chakdara Grid Station.
- In view of planned COD of Sharmai HPP in 2025, the proposed interconnection schemes have been assessed for steady state conditions through detailed load flow studies for summer 2025.
- Steady state analysis by load flow reveals that the proposed scheme is adequate to evacuate the maximum power of 150 MW of the plant under normal conditions and no constraints are caused by the interconnection of Sharmai HPP in the 132 kV network of PESCO or 220 kV network of NTDC in the load flow scenarios of summer 2025.
- The short circuit levels of the Sharmai HPP 132 kV are 6.92 kA and 2.61 kA for 3-phase and 1-phase faults, respectively, in the year 2024-25. Similarly, the short circuit levels of the Sharmai HPP 220 kV are 6.73 kA and 6.92 kA for 3-phase and 1-phase faults, respectively, in the year 2024-25. Therefore, industry standard switchgear of a short circuit rating of 40 kA would be sufficient for installation at switchyard of Sharmai HPP, as the maximum short circuit levels for the year 2024-25 were also found to be well within this range, taking care of any future generation additions and system reinforcements in its electrical vicinity and also fulfilling the NEPRA Grid Code requirements specified for 132 kV and 220 kV switchgears. There are no violations of the power rating of the equipment in the vicinity of Sharmai HPP in the event of fault conditions.
- The dynamic stability analysis of proposed schemes of interconnection has been carried out. The stability has been tested for the worst cases, i.e. three phase fault right on

the bus bar of Sharmai HPP substation followed by trip of a single circuit from Sharmai HPP has been performed for fault clearing of 5 cycles (100 ms), as understood to be the normal fault clearing time of protection system. Also the extreme worst case of stuck breaker (breaker failure) has been studied where the fault clearing time is assumed 9 cycles i.e. 180 ms for single phase fault. The stability of the system for far end faults of 3-phase occurring at Sharmai bus bar has also been checked. The system is stable for all the tested fault conditions.

- The proposed scheme of interconnection have been subjected to Load Flow, Short Circuit and Dynamic Stability Analysis and found to be feasible for interconnection of Sharmai HPP with the PESCO network.

TABLE OF CONTENTS

1. Introduction

1.1. Background

Khyber Pakhtunkhwa has a rich potential of small and big hydropower projects in the province. A lot of private investors are coming in to tap this huge natural resource. Sapphire Hydro Limited is one such investor which plans to develop a 150 MW hydropower plant at Chakdara, Upper Dir. The project site is located about 95 km from Chakdara 220/132 kV G/S. The net output planned to be generated from the site is about 150 MW of electrical power. The electricity generated from this project would be supplied to the grid system of NTDC through 220/132 kV Chakdara Grid Station. Sharmai HPP is expected to start commercial operation by 2025. The approximate location of Sharmai HPP can be seen in the map attached in Appendix – B and the neighboring network is evident from Sketch-2 attached in Appendix - B.

A number of hydropower projects are present in the vicinity of the project such as 102 MW Shigokas HPP, 63 MW Artistic-I HPP, 56 MW Artistic-II HPP, 81 MW Malakand HPP, 69 MW Lavi HPP along with several small HPPs such as Dargai, Koto and Jabban hydropower projects.

1.2. Objectives

The overall objective of the Study is to evolve an interconnection scheme between Sharmai HPP and PESCO network, for stable and reliable evacuation of 150 MW of electrical power generated from this plant, fulfilling the N-1 reliability criteria. The specific objectives of this report are:

- To develop scheme of interconnections at 132 kV or 220 kV for which right of way (ROW) and space at the terminal substations would be available.
- To determine the performance of interconnection scheme during steady state conditions of system, normal and N-1 contingency, through load-flow analysis.
- To check if the contribution of fault current from the plant unit increases the fault levels at the adjoining substations at 132 kV or 220 kV voltage levels to be within the rating of equipment of these substations, and also determine the short circuit ratings of the proposed equipment of the substation at Sharmai HPP.
- To check if the interconnection withstands dynamic stability criteria of post fault recovery with good damping.

1.3. Planning Criteria

The planning criteria required to be fulfilled by the proposed interconnection is as follows:

Steady State:

Voltage	± 5 %, Normal Operating Condition
	± 10 %, Contingency Conditions
Frequency	50 Hz Nominal
	49.8 Hz to 50.2 Hz variation in steady state
	49.4 - 50.5 Hz, Min/Max Contingency Freq. Band
Power Factor	0.80 Lagging; 0.9 Leading

Short Circuit:

132 kV Substation Equipment Rating of 40 kA

220 kV Substation Equipment Rating of 40 kA

Dynamic/Transient:

The system should revert to normal condition after transients die out with good damping, without losing synchronism. The system is tested under the following fault conditions:

a) Permanent three-phase fault on any primary transmission element; including: transmission circuit, substation bus section, transformer or circuit breaker. It is assumed that such a fault shall be cleared by the associated circuit breaker action in 5 cycles.

b) Failure of a circuit breaker to clear a fault ("Stuck Breaker" condition) in 9 cycles after fault initiation.

2. Assumptions of Data

As per the data provided by the client following data has been modeled:

2.1. Sharmai HPP Data

No. of Units	= 3
Gross Capacity of Power Project	= 152.12 MW
Auxiliary Load	= 1% (1.52 MW)
Net Capacity of the Power Project	= 150 MW
Lump sum MVA capacity	= 167 MVA
Generating Voltage	= 11 kV

The detailed parameters, which have been used in this study, for all the machines are attached in Appendix – B.

2.2. Network Data

The 132 kV network in the area near Sharmai HPP, is shown in Sketches in Appendix-B. The latest Generation Expansion Plan and Load Forecast of NTDC, as available, and latest network data and load forecast of PESCO, vide data permission letter no. CE (Dev) 5100-04 dated 20/07/2020 has been used as shown in Appendix-A.

3. Study Approach and Methodology

3.1. Understanding the Problem

Sapphire Hydro Limited is developing a hydropower project in the Upper Dir on River Panjkora with the aim of exporting a maximum of 150 MW supply to the grid. The site of proposed project is located at a distance of about 95 km from the 220/132 kV Chakdara Grid Station. The proposed Sharmai Hydropower Project is going to be embedded in the transmission network of PESCO through the most feasible network.

The adequacy of power system network in and around the proposed site of Sharmai HPP has been investigated in this study for absorbing and transmitting this power fulfilling the reliability criteria.

3.2. Approach to the Problem

The following approach has been applied to the problem:

- The scenario of summer 2025 has been selected for the study of Sharmai HPP. Thus, lines in the vicinity of this plant will be loaded to the maximum extent, allowing us to judge the complete impact of the plant on the transmission system in its vicinity.
- The scenario of summer 2025 has also been completely analyzed for the system, considering maximum hydel dispatches and the maximum power demand in the system.
- An interconnection scheme without any physical constraints, such as right of way or availability of space in the terminal substations, have been identified.
- Technical system studies have been conducted for peak load conditions, to confirm technical feasibility of the interconnection. The schemes will be subjected to standard analyses such as load flow, short circuit, and transient stability to gauge the strength of the machines and the proposed interconnection under disturbed conditions.

4. Development of Interconnection Scheme

4.1. The Existing Network and the Proposed Scheme of Interconnection

The existing 132 kV network available around the proposed location of Sharmai HPP is shown in Sketch-1 in Appendix-B.

Two options were considered for interconnection of Sharmai HPP. First option is to connect the HPP to 132 kV grid station of Chakdara. The other option is to connect Sharmai HPP to 220 kV Chakdara grid station.

Sharmai HPP can be connected to 220/132 kV with a 95 km double circuit. The conductor for the scheme is Rail, so that power can easily be evacuated even during N-1 contingency.

Hence, this scheme of interconnection has been studied for load flow analysis in this report.

5. Detailed Load Flow Studies

The base cases have been developed for the peak conditions of summer 2025 using the network data of NTDC available with PPI and the updated transmission plan and load forecast of PESCO. Detailed load flow studies have been carried out for summer 2025. The plant has been modelled in detail according to the client provided information mentioned in Chapter- 2 and attached in Appendix - B. Option-1 is when Sharmai HPP is connected to Chakadara 132 kV. Option-2 has Sharmai HPP connected to Chakdara 220 kV.

5.1. Peak Load Flow Case Summer 2025 (Option-1)

5.1.1. Peak Load Flow Case Summer 2025 – Without Sharmai HPP

The results of load flow for this base case are plotted in Exhibit 0.0 of Appendix-C. The system plotted in this Exhibit shows 132 kV network in the vicinity of Sharmai HPP including the 132 kV substations of Munda, Khar Bajawar, Chakdara, Timergara, etc.

The load flow results show that the power flows on all circuits are within their specified normal current carrying rating. The voltages are also within the permissible limits.

N-1 contingency analysis has been carried out and the plotted results are attached in Appendix – C as follows:

Exhibit-0.1	Timergara to Chakdara New 132 kV Single Circuit Out
Exhibit-0.2	Chakdara New to Chakdara 132 kV Single Circuit Out
Exhibit-0.3	Kabal to Chakdara New 132 kV Single Circuit Out
Exhibit-0.4	Chakdara to Shergarh 132 kV Single Circuit Out
Exhibit-0.5	Shergarh to Mardan-II 132 kV Single Circuit Out
Exhibit-0.6	Chakdara to Jabban PH 132 kV Single Circuit Out
Exhibit-0.7	Jabban PH to Jalala 132 kV Single Circuit Out
Exhibit-0.8	Chakdara New 220/132 kV Single Transformer Out

5.1.2. Peak Load Flow Case Summer 2025 – With Sharmai HPP

The results of load flow for the base case with Sharmai HPP interconnected are shown in Exhibit 1.0 of Appendix-C. The power flows on the circuits under normal conditions are seen well within the rated capacities. Also, the voltages on the bus bars are within the permissible operating range of ± 5 % off the nominal

We find no capacity constraints on the 132 kV circuits under normal conditions i.e. without any outages of circuits as shown in Exhibit 1.0 in Appendix - C

N-1 contingency analysis has been carried out and the plotted results are attached in Appendix – C as follows:

Exhibit-1.1 Sharmai HPP to Chakdara New 132 kV Single Circuit Out

Exhibit-1.2 Timergara to Chakdara New 132 kV Single Circuit Out

Exhibit-1.3 Chakdara New to Chakdara 132 kV Single Circuit Out

Exhibit-1.4 Kabal to Chakdara New 132 kV Single Circuit Out

Exhibit-1.5 Chakdara to Shergarh 132 kV Single Circuit Out

Exhibit-1.6 Shergarh to Mardan-II 132 kV Single Circuit Out

Exhibit-1.7 Chakdara to Jabban PH 132 kV Single Circuit Out

Exhibit-1.8 Jabban PH to Jalala 132 kV Single Circuit Out

Exhibit-1.9 Chakdara New 220/132 kV Single Transformer Out

We find that power flows on the circuits are seen well within the rated capacities and the voltages on the bus bars are also within the permissible operating range of ± 10 % off the nominal for contingency conditions' criteria. We find no capacity constraints on 132 kV circuits under normal and contingency conditions.

5.2. Off- Peak Load Flow Case Summer 2025

An off-peak case has been developed from the peak 2025 case considering 80% loads and off-peak hydel dispatches in the system. The normal case for this analysis is shown in Exhibit – 2.0 in Appendix – C.

The power flows on the circuits are seen well within the rated capacities and the voltages on the bus bars are also within the permissible operating range of ± 5 % off the nominal.

We find no capacity constraints on 132 kV circuits under normal conditions i.e. without any outages of circuits, as shown in Exhibit 2.0 in Appendix - C.

N-1 contingency analysis has been carried out and the plotted results are attached in Appendix – C as follows:

Exhibit-2.1 Sharmai HPP to Chakdara New 132 kV Single Circuit Out

Exhibit-2.2 Timergara to Chakdara New 132 kV Single Circuit Out

Exhibit-2.3 Chakdara New to Chakdara 132 kV Single Circuit Out

Exhibit-2.4 Kabal to Chakdara New 132 kV Single Circuit Out

Exhibit-2.5 Chakdara to Shergarh 132 kV Single Circuit Out

Exhibit-2.6 Shergarh to Mardan-II 132 kV Single Circuit Out

Exhibit-2.7 Chakdara to Jabban PH 132 kV Single Circuit Out

Exhibit-2.8 Jabban PH to Jalala 132 kV Single Circuit Out

Exhibit-2.9 Chakdara New 220/132 kV Single Transformer Out

The power flows on the circuits are seen well within the rated capacities and the voltages on bus bars are also within the permissible operating range of ± 10 % off the nominal for contingency conditions' criteria.

We find that there are no capacity constraints in the proposed connectivity scheme even in the off-peak scenario.

5.3. Peak Load Flow Case Summer 2025 (Option-2)

5.3.1. Peak Load Flow Case Summer 2025 – Without Sharmai HPP

The results of load flow for this base case are plotted in Exhibit 0.0 of the 220 kV connectivity section of Appendix-C. The system plotted in this Exhibit shows 132 kV network in the vicinity of Sharmai HPP including the 220 kV substations of Swabi, Nowshera, Peshawar, Mardan, etc.

The load flow results show that the power flows on all circuits are within their specified normal current carrying rating. The voltages are also within the permissible limits.

N-1 contingency analysis has been carried out and the plotted results are attached in Appendix – C as follows:

Exhibit-0.1 Chakdara to Nowshera 220 kV Single Circuit Out

Exhibit-0.2 Nowshera to Swabi 220 kV Single Circuit Out

Exhibit-0.3 Nowshera to Shaibag-N 220 kV Single Circuit Out

Exhibit-0.4 Nowshera to Kohat 220 kV Single Circuit Out

Exhibit-0.5 Chakdara to Mardan 220 kV Single Circuit Out

Exhibit-0.6 Tarbela to Mardan 220 kV Single Circuit Out

Exhibit-0.7 Chakdara New 220/132 kV Single Transformer Out

5.3.2. Peak Load Flow Case Summer 2025 – With Sharmai HPP

The results of load flow for this case with Sharmai HPP interconnected are shown in Exhibit 1.0 of the 220 kV connectivity section of Appendix-C. The system plotted in this Exhibit shows 220 kV network in the vicinity of Sharmai HPP including the 220 kV substations of Chakdara, Nowshera, Swabi, Peshawar, Mardan etc.

The power flows on the circuits under normal conditions are seen well within the rated capacities. Also, the voltages on the bus bars are within the permissible operating range of ± 5 % off the nominal

We find no capacity constraints on the 220 kV circuits under normal conditions i.e. without any outages of circuits as shown in Exhibit 1.0 in Appendix - C

N-1 contingency analysis has been carried out and the plotted results are attached in Appendix – C as follows:

Exhibit-1.1	Sharmai HPP to Chakdara 220 kV Single Circuit Out
Exhibit-1.2	Chakdara to Nowshera 220 kV Single Circuit Out
Exhibit-1.3	Nowshera to Swabi 220 kV Single Circuit Out
Exhibit-1.4	Nowshera to Shaibag-N 220 kV Single Circuit Out
Exhibit-1.5	Nowshera to Kohat 220 kV Single Circuit Out
Exhibit-1.6	Chakdara to Mardan 220 kV Single Circuit Out
Exhibit-1.7	Tarbela to Mardan 220 kV Single Circuit Out
Exhibit-1.8	Chakdara New 220/132 kV Single Transformer Out

We find that power flows on the circuits are seen well within the rated capacities and the voltages on the bus bars are also within the permissible operating range of ± 10 % off the nominal for contingency conditions' criteria. We find no capacity constraints on 220 kV circuits under normal and contingency conditions.

5.4. Off- Peak Load Flow Case Summer 2025

An off-peak case has been developed from the peak 2025 case considering 80% loads and off-peak hydel dispatches in the system. The normal case for this analysis is shown in Exhibit – 2.0 in Appendix – C.

The power flows on the circuits are seen well within the rated capacities and the voltages on the bus bars are also within the permissible operating range of ± 5 % off the nominal.

We find no capacity constraints on 220 kV circuits under normal conditions i.e. without any outages of circuits, as shown in Exhibit 2.0 in Appendix - C.

N-1 contingency analysis has been carried out and the plotted results are attached in Appendix – C as follows:

Exhibit-2.1	Sharmai HPP to Chakdara 220 kV Single Circuit Out
Exhibit-2.2	Chakdara to Nowshera 220 kV Single Circuit Out

Exhibit-2.3	Nowshera to Swabi 220 kV Single Circuit Out
Exhibit-2.4	Nowshera to Shaibag-N 220 kV Single Circuit Out
Exhibit-2.5	Nowshera to Kohat 220 kV Single Circuit Out
Exhibit-2.6	Chakdara to Mardan 220 kV Single Circuit Out
Exhibit-2.7	Tarbela to Mardan 220 kV Single Circuit Out
Exhibit-2.8	Chakdara New 220/132 kV Single Transformer Out

The power flows on the circuits are seen well within the rated capacities and the voltages on bus bars are also within the permissible operating range of ± 10 % off the nominal for contingency conditions' criteria.

We find that there are no capacity constraints in the proposed connectivity scheme even in the off-peak scenario.

5.5. Conclusion of Load Flow Analysis

From the analysis discussed above. we conclude that the proposed connection of Sharmai HPP with network according to proposed interconnection schemes is adequate to evacuate its power under normal as well as contingency conditions. Hence, there are no constraints in connecting Sharmai HPP to the 220/132 kV Chakdara Grid Station for the evacuation of 150 MW power.

6. Short Circuit Analysis

6.1.1. Methodology and Assumptions

The methodology of IEC 909 has been applied in all short circuit analyses in this report for which provision is available in the PSS/E software used for these studies.

The maximum fault currents have been calculated with the following assumptions under IEC 909:

- Set tap ratios to unity
- Set line charging to zero
- Set shunts to zero in positive sequence
- Desired voltage magnitude at bus bars set equal to 1.10 P.U. i.e. 10 % higher than nominal, which is the maximum permissible voltage under contingency condition.

For evaluation of maximum short circuit levels we have assumed contribution in the fault currents from all the installed generation capacity of hydel, thermal and nuclear plants in the system in the years 2025 i.e. all the generating units have been assumed on-bar in fault calculation's simulations.

The assumptions about the generator and the transformers data are the same as mentioned in Chapter.2 of this report.

6.2. Fault Current Calculations Year 2025 (Option-1)

6.2.1. Fault Current Calculations Year 2025 - without Sharmai HPP

In order to assess the short circuit strength of the 132 kV network without Sharmai HPP, three-phase and single-phase fault currents have been calculated for PESCO in the vicinity of the site of the Plant near Chakdara. The results are attached in Appendix – D.

The short circuit levels have been calculated and plotted on the bus bars of 132 kV of substations lying in the electrical vicinity of our area of interest and are shown plotted in the Exhibit 3.0 attached in Appendix-D. Both 3-phase and 1-phase fault currents are indicated in the Exhibit 3.0 which are given in polar coordinates i.e. the magnitude and the angle of the current. The total fault currents are shown below the bus bar.

The tabular output of the short circuit calculations is also attached in Appendix-D for the 132 kV and 11 kV bus bars of our interest. The total maximum fault currents for 3-phase and 1-phase short circuit at these substations are summarized in Table 6.1. We see that the maximum fault currents do not exceed the short circuit ratings of the equipment at these 132 kV substations which normally are 31.5 kA for older substations and 40 kA for new substations.

Table-6.1
Maximum Short Circuit Levels without Sharmai HPP – Year 2025

Substation	3-Phase fault current, kA	1-Phase fault current, kA
Chakdara New 132 kV	20.82	5.38
Chakdara 132 kV	19.93	5.38
Timergara 132 kV	11.06	3.02
Salarzai 132	9.37	1.16
Barikot 132 kV	9.54	2.17
Shergarh 132 kV	9.20	1.99
Lal Qila 132 kV	5.63	1.29
Koto HPP 132 kV	9.30	2.56.

6.2.2. Fault Current Calculations Year 2025 - with Sharmai HPP

Fault currents have been calculated for the electrical interconnection of proposed scheme. Fault types applied are three phase and single-phase at the 132 kV bus bar of Sharmai HPP itself and other bus bars of the 132 kV and 11 kV substations in the electrical vicinity of Sharmai HPP. The graphic results are shown in Exhibit 3.1.

The tabulated results of short circuit analysis showing all the fault current contributions with short circuit impedances on 132 kV bus bars of the network in the electrical vicinity of Sharmai HPP and the 132 kV bus bars of Sharmai HPP itself are placed in Appendix-D. Brief summary of fault currents at significant bus bars of our interest are tabulated in Table 6.2.

Table-6.2
Maximum Short Circuit Levels with Sharmai HPP – Year 2025

Substation	3-Phase fault current, kA	1-Phase fault current, kA
Sharmai HPP 132 kV	6.92	2.61
Chakdara New 132 kV	22.64	5.92
Chakdara 132 kV	21.20	5.73
Timergara 132 kV	11.30	3.07
Salarzai 132	9.69	1.18

Barikot 132 kV	9.80	2.22
Shergarh 132 kV	9.24	2.00
Lal Qila 132 kV	5.68	1.30
Koto HPP 132 kV	9.45	2.59

Comparison of Tables 6.1 and 6.2 shows an increase in short circuit levels for three-phase and single-phase faults due to connection of Sharmai HPP on the 132 kV bus bars in its vicinity. We find that even after some increase, these fault levels are much below the rated short circuit values of the equipment installed on these substations.

6.3. Fault Current Calculations Year 2025 (Option-2)

6.3.1. Fault Current Calculations Year 2025 - without Sharmai HPP

In order to assess the short circuit strength of the 220 kV network without Sharmai HPP, three-phase and single-phase fault currents have been calculated for NTDC/PESCO in the vicinity of the site of the Plant near Chakdara. The results are attached in Appendix – D.

The short circuit levels have been calculated and plotted on the bus bars of 220 kV of substations lying in the electrical vicinity of our area of interest and are shown plotted in the Exhibit 3.0 attached in Appendix-D. Both 3-phase and 1-phase fault currents are indicated in the Exhibit 3.0 which are given in polar coordinates i.e. the magnitude and the angle of the current. The total fault currents are shown below the bus bar.

The tabular output of the short circuit calculations is also attached in Appendix-D for the 220 kV bus bars of our interest. The total maximum fault currents for 3-phase and 1-phase short circuit at these substations are summarized in Table 6.1. We see that the maximum fault currents do not exceed the short circuit ratings of the equipment at these 220 kV substations.

Table-6.3
Maximum Short Circuit Levels without Sharmai HPP – Year 2025

Substation	3-Phase fault current, kA	1-Phase fault current, kA
Chakdara 220 kV	13.71	8.92
Nowshera 220 kV	47.76	32.12
Swabi 220 kV	22.28	11.55
Shaibag-N 220 kV	32.55	21.61
Kohat 220 kV	15.40	10.47

Peshawar 220 kV	37.46	25.73
Noshera-1 220 kV	42.92	29.87
Mardan 220 kV	30.41	19.33

6.3.2. Fault Current Calculations Year 2025 - with Sharmai HPP

Fault currents have been calculated for the electrical interconnection of proposed scheme. Fault types applied are three phase and single-phase at the 220 kV bus bar of Sharmai HPP itself and other bus bars of the 220 kV substations in the electrical vicinity of Sharmai HPP. The graphic results are shown in Exhibit 3.1.

The tabulated results of short circuit analysis showing all the fault current contributions with short circuit impedances on 220 kV bus bars of the network in the electrical vicinity of Sharmai HPP and the 220 kV bus bars of Sharmai HPP itself are placed in Appendix-D. Brief summary of fault currents at significant bus bars of our interest are tabulated in Table 6.2.

Table-6.4
Maximum Short Circuit Levels with Sharmai HPP – Year 2025

Substation	3-Phase fault current, kA	1-Phase fault current, kA
Sharmai HPP 220 kV	6.73	6.92
Chakdara 220 kV	15.16	10.95
Nowshera 220 kV	48.26	32.48
Swabi 220 kV	22.40	11.59
Shaibag-N 220 kV	32.73	21.73
Kohat 220 kV	15.43	10.48
Peshawar 220 kV	37.62	25.84
Noshera-1 220 kV	43.25	30.10
Mardan 220 kV	30.81	19.67

Comparison of Tables 6.1 and 6.2 shows an increase in short circuit levels for three-phase and single-phase faults due to connection of Sharmai HPP on the 220 kV bus bars in its vicinity. We find that even after some increase, these fault levels are much below the rated short circuit values of the equipment installed on these substations.

6.3.3. Conclusion of Short Circuit Analysis

The short circuit analysis results show that for the proposed scheme of interconnection of Sharmai HPP, we don't find any violations of short circuit ratings of the already installed equipment on the 132 kV and 220 kV bus bars in the vicinity of the plant due to fault current contributions from Sharmai HPP. Therefore industry standard switchgear of the short circuit rating of 40 kA would serve the purpose as per NTDC requirement taking care of any future generation additions and system reinforcements in its electrical vicinity.

7. Dynamic Stability Analysis

7.1. Assumptions and Methodology

7.1.1. Dynamic Models Assumptions

The assumptions about the generator and its parameters are the same as mentioned in Chapter 2 of this report.

We have employed the generic dynamic models available in the PSS/E model library for dynamic modeling of the generator, exciter and the governor as follows;

Generator	GENSAL
Excitation System	EXST1
Speed Governing System	HYGOV

7.1.2. Dynamic Models Assumptions

The scenario of summer 2025 has been selected for the study because it represents the peak load season after the COD of Sharmai HPP and thus the loading on the lines in the vicinity of Sharmai HPP will be maximum, allowing us to judge the full impact of the plant.

The proposed Sharmai HPP has been modeled in the dynamic simulation as per data provided by client. All the power plants of WAPDA/NTDC from Tarbela to Hub have been dynamically represented in the simulation model.

7.1.3. Presentation of Results

The plotted results of the simulations runs are placed in Appendix-E. Each simulation is run for its first one second for the steady state conditions of the system prior to fault or disturbance. This is to establish the pre fault/disturbance conditions of the network under study were smooth and steady. Post fault recovery has been monitored for ten seconds. Usually all the transients due to non-linearity die out within a few seconds after disturbance is cleared in the system.

7.2. Worst Fault Cases (Option-1)

Three phase faults are considered as the worst disturbance in the system. We have considered 3-phase fault in the closest vicinity of Sharmai HPP i.e. right at the 132 kV bus bar of Sharmai HPP substation, cleared in 5 cycles, as normal clearing time for 132 kV i.e. 100 ms, followed by a permanent trip of a 132 kV single circuit from Sharmai HPP to Chakdara. Also to fulfil the Grid Code criteria case of stuck breaker (breaker failure) single phase fault has also been studied where the fault clearing time is assumed 9 cycles i.e. 180 ms.

7.2.1. Dynamic Stability Simulations Results

The detailed parameters used for the Stability Analysis have been tabulated in Appendix – E.

a) **Fault at 132 kV Sharmai HPP**

Fault Type: 3-Phase			
Fault Location: Sharmai 132 kV bus bar			
Fault Duration: 5 cycles (100 ms)			
Line Tripping: Sharmai HPP to Chakdara New 132 kV single circuit			
Variable	**Bus/Line**	**Response**	**Figure No.**
Voltage	1. Sharmai HPP 11 kV 2. Sharmai HPP 132 kV 3. Chakdara New 132 kV 4. Kabal 132 kV 5. Barikot 132 kV 6. Chirat Ind 132 kV	The voltages of all the bus bars recover after fault clearance	1.1
Frequency	Sharmai 132 kV	Recovers after fault clearance	1.2
MW/MVAR Output of the Plant	Sharmai unit-1 11 kV	Recovers after damping down oscillations	1.3
Speed and $P_{mechanical}$ of the Plant	Sharmai unit-1 11 kV	Recovers after damping down oscillations	1.4
Line Flows (MW/MVAR)	Sharmai to Chakdara 132 kV intact single circuit	Attains steady state value after damping of oscillations	1.5
Rotor Angles	1. Sharmai HPP 11 kV 2. Shigokas 132 kV 3. Artistic-I 132 kV 4. Koto HPP 132 kV 5. Malakand 132 kV 6. G.Brotha 500 kV(reference angle)	Damps down quickly and attain a steady state value	1.6

b) Fault at 132 kV Sharmai HPP (Stuck Breaker)

Fault Type: 1-Phase			
Fault Location: Sharmai 132 kV bus bar			
Fault Duration: 9 cycles (180 ms)			
Line Tripping: Sharmai to Chakdara New 132 kV single circuit			
Variable	**Bus/Line**	**Response**	**Figure No.**
Voltage	1. Sharmai HPP 11 kV 2. Sharmai HPP 132 kV 3. Chakdara New 132 kV 4. Kabal 132 kV 5. Barikot 132 kV 6. Chirat Ind 132 kV	The voltages of all the bus bars recover after fault clearance	2.1
Frequency	Sharmai 132 kV	Recovers after fault clearance	2.2
MW/MVAR Output of the Plant	Sharmai Unit-1 11 kV	Recovers after damping down oscillations	2.3
Speed and $P_{mechanical}$ of the Plant	Sharmai Unit-1 11 kV	Recovers after damping down oscillations	2.4
Line Flows (MW/MVAR)	Sharmai to Chakdara 132 kV intact single circuit	Attains steady state value after damping of oscillations	2.5
Rotor Angles	1. Sharmai HPP 11 kV 2. Shigokas 132 kV 3. Artistic-I 132 kV 4. Koto HPP 132 kV 5. Malakand 132 kV 6. G.Brotha 500 kV(reference angle)	Damps down quickly and attain a steady state value	2.6

c) Fault at 132 kV Chakdara New

Fault Type: 3-Phase			
Fault Location: Chakdara New 132 kV bus bar			
Fault Duration: 5 cycles (100 ms)			
Line Tripping: Sharmai HPP to Chakdara New 132 kV single circuit			
Variable	**Bus/Line**	**Response**	**Figure No.**
Voltage	1. Chakdara New 132 kV 2. Sharmai HPP 132 kV 3. Kabal 132 kV 4. Barikot 132 kV 5. Chirat Ind 132 kV 6. Timergara 132 kV	The voltages of all the bus bars recover after fault clearance	3.1
Frequency	Sharmai 132 kV	Recovers after fault clearance	3.2
MW/MVAR Output of the Plant	Sharmai Unit-1 11 kV	Recovers after damping down oscillations	3.3
Speed and $P_{mechanical}$ of the Plant	Sharmai Unit-1 11 kV	Recovers after damping down oscillations	3.4
Line Flows (MW/MVAR)	Sharmai to Chakdara 132 kV intact single circuit	Attains steady state value after damping of oscillations	3.5
Rotor Angles	1. Sharmai HPP 11 kV 2. Shigokas 132 kV 3. Artistic-I 132 kV 4. Koto HPP 132 kV 5. Malakand 132 kV 6. G.Brotha 500 kV(reference angle)	Damps down quickly and attain a steady state value	3.6

7.3. Worst Fault Cases (Option-2)

Three phase faults are considered as the worst disturbance in the system. We have considered 3-phase fault in the closest vicinity of Sharmai HPP i.e. right at the 220 kV bus bar of Sharmai HPP substation, cleared in 5 cycles, as normal clearing time for 220 kV i.e. 100 ms, followed by a permanent trip of a 220 kV single circuit from Sharmai HPP to Chakdara. Also to fulfil the Grid Code criteria case of stuck breaker (breaker failure) single phase fault has also been studied where the fault clearing time is assumed 9 cycles i.e. 180 ms.

7.3.1. Dynamic Stability Simulations Results

The detailed parameters used for the Stability Analysis have been tabulated in Appendix – E.

a) Fault at 220 kV Sharmai HPP

Fault Type: 3-Phase			
Fault Location: Sharmai 220 kV bus bar			
Fault Duration: 5 cycles (100 ms)			
Line Tripping: Sharmai to Chakdara 220 kV single circuit			
Variable	**Bus/Line**	**Response**	**Figure No.**
Voltage	1. Sharmai 11 kV 2. Sharmai 220 kV 3. Chakdara 220 kV 4. Chakdara New 132 kV 5. Nowshera 220 kV 6. Swabi 220 kV	The voltages of all the bus bars recover after fault clearance	1.1
Frequency	Sharmai 220 kV	Recovers after fault clearance	1.2
MW/MVAR Output of the Plant	Sharmai unit-1 11 kV	Recovers after damping down oscillations	1.3
Speed and $P_{mechanical}$ of the Plant	Sharmai unit-1 11 kV	Recovers after damping down oscillations	1.4
Line Flows (MW/MVAR)	Sharmai to Chakdara 220 kV intact single circuit	Attains steady state value after damping of oscillations	1.5
Rotor Angles	1. Sharmai HPP 11 kV 2. Mohmand HYD 220 kV 3. Unit-1 Tarbela 13.8 kV 4. Artistic-1 11 kV 5. Koto HPP 132 kV 6. G.Brotha 500 kV(reference angle)	Damps down quickly and attain a steady state value	1.6

b) Fault at 220 kV Sharmai HPP (Stuck Breaker)

Fault Type: 1-Phase			
Fault Location: Sharmai 220 kV bus bar			
Fault Duration: 9 cycles (180 ms)			
Line Tripping: Sharmai to Chakdara 220 kV single circuit			
Variable	**Bus/Line**	**Response**	**Figure No.**
Voltage	1. Sharmai 11 kV 2. Sharmai 220 kV 3. Chakdara 220 kV 4. Chakdara New 132 kV 5. Nowshera 220 kV 6. Swabi 220 kV	The voltages of all the bus bars recover after fault clearance	2.1
Frequency	Sharmai 220 kV	Recovers after fault clearance	2.2
MW/MVAR Output of the Plant	Sharmai unit-1 11 kV	Recovers after damping down oscillations	2.3
Speed and $P_{mechanical}$ of the Plant	Sharmai unit-1 11 kV	Recovers after damping down oscillations	2.4
Line Flows (MW/MVAR)	Sharmai to Chakdara 220 kV intact single circuit	Attains steady state value after damping of oscillations	2.5
Rotor Angles	1. Sharmai HPP 11 kV 2. Mohmand HYD 220 kV 3. Unit-1 Tarbela 13.8 kV 4. Artistic-1 11 kV 5. Koto HPP 132 kV 6. G.Brotha 500 kV(reference angle)	Damps down quickly and attain a steady state value	2.6

c) Fault at 220 kV Chakdara

Fault Type: 3-Phase			
Fault Location: Chakdara 220 kV bus bar			
Fault Duration: 5 cycles (100 ms)			
Line Tripping: Sharmai to Chakdara 220 kV single circuit			
Variable	**Bus/Line**	**Response**	**Figure No.**
Voltage	1. Sharmai 11 kV 2. Sharmai 220 kV 3. Chakdara 220 kV 4. Chakdara New 132 kV 5. Nowshera 220 kV 6. Swabi 220 kV	The voltages of all the bus bars recover after fault clearance	3.1
Frequency	Sharmai 220 kV	Recovers after fault clearance	3.2
MW/MVAR Output of the Plant	Sharmai unit-1 11 kV	Recovers after damping down oscillations	3.3
Speed and $P_{mechanical}$ of the Plant	Sharmai unit-1 11 kV	Recovers after damping down oscillations	3.4
Line Flows (MW/MVAR)	Sharmai to Chakdara 220 kV intact single circuit	Attains steady state value after damping of oscillations	3.5
Rotor Angles	1. Sharmai HPP 11 kV 2. Mohmand HYD 220 kV 3. Unit-1 Tarbela 13.8 kV 4. Artistic-1 11 kV 5. Koto HPP 132 kV 6. G.Brotha 500 kV(reference angle)	Damps down quickly and attain a steady state value	3.6

7.4. Conclusion of Dynamic Stability Analysis

The results of dynamic stability carried out for summer 2025 show that the system is very strong and stable for the proposed scheme for the severest possible faults of 132 kV and 220 kV systems near to and far from Sharmai HPP under all events of disturbances. Therefore there is no problem of dynamic stability for interconnection of sharmai HPP; it fulfills all the criteria of dynamic stability.

8. Conclusions

- The Final Report of 150 MW Sharmai Hydro Power Project by Sapphire Hydro Limited in Upper Dir, Khyber Pakhtunkhwa, is submitted herewith.
- Two options were investigated for the interconnection of this project. One possible interconnection scheme is to connect the plant to 132 kV Chakdara-New Grid Station. Another possible point of interconnection is to connect the said plant to 220 kV Chakdara Grid Station.
- In view of planned COD of Sharmai HPP in 2025, the proposed interconnection schemes have been assessed for steady state conditions through detailed load flow studies for summer 2025.
- Steady state analysis by load flow reveals that the proposed scheme is adequate to evacuate the maximum power of 150 MW of the plant under normal conditions and no constraints are caused by the interconnection of Sharmai HPP in the 132 kV network of PESCO or 220 kV network of NTDC in the load flow scenarios of summer 2025.
- The short circuit levels of the Sharmai HPP 132 kV are 6.92 kA and 2.61 kA for 3-phase and 1-phase faults, respectively, in the year 2024-25. Similarly, the short circuit levels of the Sharmai HPP 220 kV are 6.73 kA and 6.92 kA for 3-phase and 1-phase faults, respectively, in the year 2024-25. Therefore, industry standard switchgear of a short circuit rating of 40 kA would be sufficient for installation at switchyard of Sharmai HPP, as the maximum short circuit levels for the year 2024-25 were also found to be well within this range, taking care of any future generation additions and system reinforcements in its electrical vicinity and also fulfilling the NEPRA Grid Code requirements specified for 132 kV and 220 kV switchgears. There are no violations of the power rating of the equipment in the vicinity of Sharmai HPP in the event of fault conditions.
- The dynamic stability analysis of proposed schemes of interconnection has been carried out. The stability has been tested for the worst cases, i.e. three phase fault right on the bus bar of Sharmai HPP substation followed by trip of a single circuit from Sharmai HPP has been performed for fault clearing of 5 cycles (100 ms), as understood to be the normal fault clearing time of protection system. Also the extreme worst case of stuck breaker (breaker failure) has been studied where the fault clearing time is assumed 9 cycles i.e. 180 ms for single phase fault. The stability of the system for far end faults of 3-phase occurring at Sharmai bus bar has also been checked. The system is stable for all the tested fault conditions.

- The proposed scheme of interconnection have been subjected to Load Flow, Short Circuit and Dynamic Stability Analysis and found to be feasible for interconnection of Sharmai HPP with the PESCO network.

Feasibility Study
for
Sharmai Hydropower Project
Pakistan

Feasibility Study Report

November 2018

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0	18.09.2020	Consolidated Feasibility Study Report	Vuckovic	Palt
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List of Acronyms and Abbreviations

B/C	Benefit/Cost
DEM	Digital Elevation Model
etc.	Et cetera
e.g.	Exempli gratia
ESIA	Environmental and Social Impact Assessment
EUR, €	Euro
FDC	Flow Duration Curve
FS	Feasibility Study
HPP	Hydro Power Plant/Project
i.e.	Id est
IRR	Internal Rate of Return
masl	meter above sea level
Mio.	Million
min_{OWL}	minimum Operation Water Level
max_{OWL}	Maximum Operation Water Level
NOL	Normal Operating Level
NPV	Net Present Value
O&M	Operation and Maintenance
PEDO	Pakhtunkhwa Energy Development Organization
PMF	Probable Maximum Flood
PH	Powerhouse
Q	Discharge
RAP	Resettlement Action Plan
RCC	Roller Compacted Concrete
ref.	Reference
ToR	Terms of Reference
WL	Water Level
y	Year

1. Executive Summary

1.1 Authorization

Sapphire Electric Company Limited, Pakistan (The Client)

appointed

Fichtner GmbH and Co KG, Germany (The Consultant)

to perform the consulting services on preparation of Feasibility Study Report for

Sharmai Hydropower Project in Pakistan.

The contract entered into force on 17 July 2017 for the period of 17 months.

1.2 Project Background

Sapphire Electric Company Limited appointed Fichtner GmbH & Co. KG to carry out a bankable feasibility study for Sharmai hydropower project in Pakistan. Upon conclusion of the feasibility stage, the project is envisioned to be developed to the tender design level and implemented as an EPC contract.

The project is located on Panjkora River, in Upper Dir province. It is envisioned as a 150 MW powerplant, able to operate as a daily storage / peaking plant. Sharmai HPP was approved by CDWP / ECNEL to be developed through PPP model.

1.3 Objective of the Feasibility Study Report

During the initial phase of the Feasibility Study, layout alternatives of the Sharmai Hydropower Project were identified and studied on comparative basis. As the result the most promising project concept was selected from several alternatives to be developed to the full feasibility level.

The selected project concept of the Sharmai HPP was analyzed and adjusted to the project-specific conditions, which are defined already during the inception phase and fine tuned later, during field investigation works. As the consequence, a new, optimized project layout was identified and developed up to the full feasibility level whereas the program for geotechnical investigation works is adjusted accordingly.

The key objectives of the Feasibility Study, among others, are:

- to undertake a feasibility study to international best practice acceptable to the Client and international financial institutions
- to work effectively with the Client and other involved parties

- to carry out, i.e. supervise execution of the required field investigations including geological and geotechnical field investigations, topographic survey and analysis of data
- to review and optimize from technical and financial aspects the alternative project concepts and to select the most promising layout for detailed studies
- to supervise and govern the socio-environmental impact assessment study performed by a different consultant
- to estimate duration of construction works and calculate the total construction costs of the project based on detailed evaluations, designs etc.; and
- to evaluate the financial and economic characteristics of the Project and to define the resulting financial parameters.

1.4 Structure of the Feasibility Study Report

The Feasibility Study for Sharmai Hydropower Project was executed in 3 stages: Inception stage, Optimization stage and Finalization stage. Following additional reports were submitted:

- Executive Summary
- Report on Hydrology and Sediments, submitted in January 2018
- Topographic Survey Report, submitted in March 2018
- Report on Site infrastructure, Transport and Accessibility, submitted in March 2018
- Optimization analyses including conceptual drawings, submitted in March 2018
- Geotechnical Baseline Report, submitted in November 2018
- Report on FS Design (including drawings), Energy Production, Costs and Implementation, submitted in November 2018
- Financial and Economic Analyses and Financing Plan, including the financial model, submitted in November 2018.

The present report represents a single, consolidated Feasibility study Report.

1.5 Performed Engineering Works

1.5.1 Design criteria

At the outset of the Feasibility Design Stage, the set of the Design Criteria to be applied for all engineering works was established. It compiles various operational, safety, stability and other world-wide recognized criteria and standards for designing of the most relevant project structures.

1.5.2 Topographic and Bathymetric Survey

Preparation of the topographic data base for Sharmai HPP was performed for the following areas:

- Tunnel area
- Dam and powerhouse sites
- Particular sites of interest: roads, bridges, man-made structures.

Topographic data base for the tunnel area was based on the satellite data with a resolution of 50 cm for the area of approx. 110 km². The works performed comprise:

- Processing of satellite imagery
- Geo-referencing
- Ortho-rectification
- Mosaicking
- Development of a Digital Elevation Model (DEM) for the area of approx. 250 km²
- Preparation of contour lines with an equidistance of 5 m and 25 m.

For the dam and powerhouse areas terrestrial investigation works were performed by Geomatics and Engineering Services (Pvt.) Limited, contracted by the Client. Based on the terrestrial data, more accurate topographic data base was created, including the following:

- Densification of the terrain points
- Construction of break lines such as river shore lines and road edges
- Development of a Digital Terrain Model (DTM) for the Dam project area of 0,56 km² and creation of contour lines with an equidistance of 1 m and 5 m
- Development of a Digital Terrain Model (DTM) for the Powerhouse project area of 1,43 km² and creation of contour lines with an equidistance of 1 m and 5 m.

1.5.3 Hydrological and Sediment Analysis

The main purpose of the hydrological and sediment study was to analyze, update and quantify:

- the water availability at the intake site
- the design floods required for the design of diversion works and other different civil and hydraulic structures of the project
- the sediment inflow over the project-lifetime, the expected siltation of the pond and the adequate design of the structures.

Results of the performed analyses were considered as the inputs to the project design, sizing and energy production analysis.

1.5.4 Geological and Geotechnical Investigation Works

In the framework of the Feasibility Study, different geological, geotechnical and geophysical investigation works and have been carried out by Decon International (Pvt) Ltd, contracted by the Client. The field works were supervised by the Consultant.

Main geological and geotechnical works carried out within the scope of the feasibility study are:

- Core-drillings in total length of about 1300 m (18 straight rotary drillings in overburden and NQ size thin walled double tube core barrel in rock varying between 22 m and 250 m)
- Test pits (14 pits - 0.8 m to 3 m deep)
- Permeability tests (58 Lugeon tests at different boreholes/ depths)
- Seismic refraction Survey (10 lines of 2,465 meters in total)
- Standard penetration test (SPTs) in boreholes (2 in total)
- Plate load test (1 in total)
- Laboratory tests
- Installation of piezometers (6 in total)
- Geological mapping
- Incorporation of Campaign results in an updated geological map
- Seismic hazard assessment study.

1.5.5 Optimization analyses

During the initial phase of the Feasibility Study, various layout alternatives of Sharmai Hydropower Project were identified and studied on comparative basis. As the result of such analysis the most promising project concept was selected from several alternatives and developed to the full feasibility level.

The optimization analysis had as the main target definition of the key technical parameters, such as dam site, dam type, number and type of generating units, reservoir water level and installed discharge. These parameters were defined based on the financial parameters different project options offered.

Secondary optimization analyses of the components within the selected project concept were carried out and resulted in further improvement of the relevant techno-economical parameters. Optimum project layout comprises the following main components:

- reservoir, capable to be operated as a daily storage, with the maximum operating level at 1260 masl and minimum operating level at 1255
- a concrete dam with the integrated gated spillway and a stilling basin
- a diversion tunnel at the right abutment
- an open air desander with 3 chambers designed to 50% installed powerhouse flow each
- system of power waterways

  - power intake
  - low pressure power tunnel
  - surge tank
  - steel lined vertical pressure shaft
- cavern powerhouse, equipped with 3 vertical Francis units
- downstream surge tank
- pressurized tailrace tunnel
- transmission line to Chakdara Substation
- access roads
- construction camps, quarry and depo sites and other temporary facilities.

1.5.6 Design of Civil Structures, HM and EM Equipment and Hydraulic Steel Structures, Connection to the Grid

Once the optimum technical project's layout was defined, the design of the civil structures and equipment fine-tuned the general project's concept. It resulted in definition of the technical design of all major project components such as civil structures, access roads, hydro- and electro-mechanical equipment and hydraulic steel structures on a feasibility level.

1.5.7 Energy production analysis

Power production analysis was performed for Sharmai HPP, assuming its seasonal operation: the plant is envisioned to operate as a peaking plant during the low flow periods, being in operation over 4 hrs/day as a general rule; more than 4 hrs/day the plant shall operate only in order to avoid loss of flow over the spillway. During the high flow season, the plant shall operate as a run-off river plant, preserving the normal operating level in the reservoir.

The reservoir simulation model was based on the series of daily reservoir inflow values at the project site for the period 1961-2017 and did consider the following inputs:

- environmental minimum release (as per the ESIA)
- reservoir operating water levels (maximum, minimum and flood)
- tailwater curve
- actual gross head (in daily steps)
- hydraulic losses along the power waterways
  - calculated friction head losses along the power waterways
  - estimated local head losses
- actual (daily) flow availability and the considered installed discharges
- efficiency of the generating equipment.

This analysis reveals that the annual energy generation at the plant amounts to about 690 GWh per year, out of which 200 GWh can be regarded as peak power. The plant is capable of supplying 146 to 150 MW to the system over the peaking hours throughout the year.

1.5.8 Environmental and Social Impact Assessment

Environmental and social impact assessment (ESIA) was carried out by Hagler Bailly Pakistan. The study concluded on the E-flow, major ESIA-related impacts and the corresponding mitigation measures and provided a detailed land acquisition plan for the project features (Section 3.3 and Exhibit 3.4 of ESIA Main report refers to land requirement).

1.5.9 Project costs

The cost estimate which is elaborated within the scope of the Feasibility Design stage of the Sharmai HPP offers an overview to the calculated volumes of materials and works and the estimated unit and total prices which cover all relevant positions of the current technical design.

The direct project costs (without taxes, duties and financing costs) are estimated at around 363 million USD (or about 2,385 USD/kW of the installed power) excluding price escalation during the duration of construction works/project implementation, i.e. on the reference date end 2018.

1.5.10 Construction and implementation schedule

Construction is foreseen in 54 months, mainly depending on the supply and the installation of the equipment and the construction works on the powerhouse cavern. Construction works on the headrace tunnel may easily intrude on the critical path, due to the uncertainties the underground works carry.

1.5.11 Economic and financial analyses

Economic and financial analyses, including sensitivity analyses have been carried out. Given the framework, under which the project is developed, the financial analysis demonstrates that the project is financially viable in all considered scenarios. The NPVs are in all financing scenarios positive.

Consumer surplus tariff approach confirms economic feasibility of the project under different sensitivities, with the positive NPV and an IRR of close to 20%.

Economic feasibility on LRMC basis confirms project's feasibility, concluding on the IRR of 17% even at the cost increase and benefits decrease of 10%. Alternate equivalent thermal plant approach confirms same.

Financial analysis gives the FIRR of 9.8% (WACC of 7.8%) and the payback period of 7 years. The Levelized tariff for the term of the PPA has been calculated as US cent 8.9555/kWh.

1.5.12 Salient project features

The following table summarizes the salient project features.

Table 1-1: Salient features

Project Name	Sharmai HPP
General Information	
Position	Dam: 769 775.25; 3 897 700.84 PH: 772 495.26; 3 889 680.94
Catchment Area	approx. 1,874 km^2
Structures	
Reservoir	
Total Volume	5.04 Mm^3
Active Volume	1.3 Mm^3
Flood Level (PMF flood event)	1264.8 masl
Maximum Operating Level	1260.0 masl
Minimum Operating Level	1255.0 masl
Reservoir Area at MWL	0.4 km^2
Average Reservoir Inflow	71.4 m^3/s
Spillway Design Flood	5040 m^3/s
Dam, River Diversion and Spillway	
Dam Type	Concrete gravity
Dam Crest Elevation	1265.00 masl (with 1.2m parapet wall)
Dam Maximum Height	45 m (to foundation)
Dam Crest Length	150 m
Diversion Tunnel - Length	500 m
Diversion Tunnel - Inner Diameter	8.0 m
Upstream Cofferdam Type	Rockfill with diaphragm
Upstream Cofferdam Crest Elevation	1245.50 masl
Downstream Cofferdam Type	Rockfill with clay carpet
Upstream Cofferdam Crest Elevation	1231.00 masl
Spillway Type	Gated, Radial Gates
Number of Gates, Dimensions (W:H)	3, 10.5 x 15.5 m
Spillway Elevation	1245.00 masl
Energy Dissipation Structure	Stilling basin
Dimensions of Energy Dissipation Structure (L x W; bottom elevation)	150 x 37.5m; 1211.00 masl
Bottom Outlet	
Number of Outlets	1; equipped with a radial gate
Gate Dimensions (W x H)	6.5 x 7.0m
Desander	
Number of Chambers	3
Chamber Dimensions (L x W x H)	245 x 13.5 x 25m

Power Waterways	
Number of Power Tunnels	1
Tunnel Length	8500 m
Tunnel Inner Diameter	6.75 m (circular)
Number of Surge Tanks	1
Surge Shaft Inner Diameter	20 m
Length of the Surge Tank	
Number of Pressure Shafts	1
Pressure Shaft Length	180 m
Pressure Shaft Inner Diameter	6.0 m
Powerhouse	
Powerhouse Type	Cavern
Number of Units	3
Turbine Type	Francis
Total Installed Turbine Discharge	90 m^3/s
Installed Power	152.12 MW
Rated Head	192.9 m
Powerhouse Dimensions (L x W x H)	54.5 x 20 x 27 m
Transformer Cavern Dimensions (L x W x H)	50 x 11 x 9.3 m
Downstream Surge Tank	
Number of Surge Tanks	1
Surge Tank Dimensions (L x H x W)	70 x 16.7 x 29 m
Tailrace Tunnel	
Length	735 m
Inner Diameter	6.75 m
Transmission Line	
Length	approx. 83 km
Voltage	220 kV
System	Double Circuit
Commercial Data	
Peak Power Generation	200 GWh/y
Off-peak Power Generation	489 GWh/y
EPC Costs	363 million USD
Construction Time	54 months

2. Introduction

2.1 Objectives of the Project

2.1.1 Excerpt from ToR

The key objectives of Sharmai HPP are outlined in the ToR:

"The main objective of the services is to undertake all possible investigations, studies and reports required by PEDO and various authorities for feasibility study approval, subsequent issuance of Letter of Support (LOS), tariff and for grant of approvals & licenses and signing of various agreements with respective authorities and Governmental agencies for the successful implementation and development of Sharmai Hydro Electric Power Project.

The feasibility study shall be bankable having all necessary investigations, reports, studies and information that is required by local banks and financial institutions as well as International Financial Instructions (IFI's) for the financing.

The key objectives for the Bankable Feasibility Study are as follows:

- *Undertake the Bankable Feasibility Study according to international best practice for similar Projects and acceptable to the Client, financial institutions (including multilateral lending agencies), PPIB and other authorities.*
- *Work with the Client and the Owners Engineer effectively and efficiently during the study.*
- *Carry out detailed geological mapping and detailed ground investigations by using specialist sub-contracted parties – these investigations shall comprise seismic refraction, sub-surface drillings, investigation pits, and field and laboratory tests.*
- *Review from a technical and economic basis the various alternative layouts and select a preferred layout for detailed study.*
- *Assess the Project in the context of the cascade scheme.*
- *Carry out socio-environmental impact studies as detailed hereinafter.*
- *Carry out a detailed risk analysis*
- *Based on detailed evaluations, designs and costing calculate the total construction costs.*
- *Evaluate the financial and economic characteristics, and determine the tariff".*

2.2 Scope of Work and Project Phasing

The activities on the elaboration of the Feasibility Study for the Sharmai HPP are grouped under 4 main stages. Each stage output has to be approved by the Client/PEDO.

Stage 1: Inception stage

- site visit and project Kick-off
- data collection
- preliminary planning/conceptual design and data analysis
- preparation of technical specifications for field investigations: topographic survey, geological field and laboratory investigations, technical specifications for full-scale ESIA.

Stage 2: Optimization stage

- outcomes of Inception stage
- supervision over field works
- obtaining of the results and deliverables of the topographic survey
- hydrological and sediment load analysis
- geological investigations and factual reporting (if timely doable)
- preliminary cost estimate for different options
- preliminary energy production analysis for different options
- preliminary financial analysis and comparison of options
- conclusion on the optimum project layout and key technical parameters
- discussion / approval of optimization stage outcome.

Stage 3: Feasibility Study stage

- outcomes of the Optimization stage
- development of feasibility design of project structures: civil works, equipment, transmission line, etc.
- detailed power generation analysis
- summary of ESIA findings
- cost estimate
- implementation time schedule
- financial and economic analyses
- risk analysis
- submission of Feasibility Study Report.

2.3 Kick-off meeting and project commencement

The Project Kick-off / Wrap-up meeting was held on 17 and 19 July 2017 in Islamabad, Pakistan in the premises of the Client. The main goal was to introduce the representatives of the parties involved in the project, to give a brief overview to the project activities and to discuss on important issues. The following agenda was agreed:

- presentation of the Consultant
- background of the project and the main goals of the study
- brief presentation of the methodology for the various tasks
- overall time schedule
- communication
- key findings of the Site Visit
- preliminary findings on the projects' conceptual layouts as proposed in the previous studies

- next steps in development of the project.

The commencement date of services was agreed to be that of the Kick-off meeting, 17 July 2017. As per the Client's request, all field works will be contracted directly by the Client and supervised by the Consultant. As such, there are number of activities for which the Client's action represents a condition precedent to commencement of further works on the project:

- Delivery of topographic maps
- Commencement and completion of geological field investigations
- Commencement and completion of ESIA-related activities
- Provision of data: hydrological, grid-related, etc.
- Provision of Client's comments to the deliverables.

The time schedule for the completion of works under the present project was consequently updated and is enclosed in the Annex 2.

2.4 Site Visit and Inception Stage

Project Kick-off meeting was followed by a site visit. The site visit was conducted on 18 and 19 July 2017, with the participants of the Client and Consultant.

During the Inception stage, several important milestones and activities for the further project development are accomplished. Primarily these are the following:

- Kick-off meeting on 17 July 2017
- Site Visit and the establishment of data base
- Collection of the available project-related documents and data and identification of the outstanding data
- Definition of the most promising conceptual layouts for Sharmai HPP
- Preparation of the specification for the field investigation works: geological, topographic and ESIA-related
- Identification of further steps in the project's development.

3. Hydrology and Sediment

3.1 Introduction

The main purpose of the hydrological study as a part of the feasibility study for any hydropower project is to analyze, update and quantify:

- the water availability at the intake site
- the design floods required for the design of diversion works and other different civil and hydraulic structures of the project
- the sediment inflow over the project-lifetime, the expected siltation of the pond and the adequate design of the structures.

Accordingly, based on the available data, documents and information, the hydrological and sedimentological investigations were carried out to provide the key parameters for hydropower computations, project components design and subsequent operation of the Sharmai HPP, as described in following sections.

3.2 Physical Characteristics of Project Area

3.2.1 Project location

The project site, Sharmai HPP in Panjkora River, is located in Dir district at the north of KPK or Khyber Pakhtunkhwa Province of the north-western region of Pakistan. The Province is surrounded by Northern Areas of Pakistan in the North, Kashmir in the East, Punjab Province of Pakistan in the Southeast, Balochistan Province in the Southwest and Afghanistan in the West.

The project site is accessible by the national road N-45. It is about 150 km towards north from Peshawar, the capital city of the Province and it is about 10 km towards east from Dir city, the headquarter of the district.

The Project site is situated in the upstream reach of the Panjkora River which confluences into the Swat River as its right tributary (the largest tributary of Swat River). The Panjkora River originates from a mountainous region (> 4000 m) and flows down, in the beginning towards south-west for about 80 km till it reaches the proposed damsite and then changes the direction towards south-east till Darora town where the proposed powerhouse site of the project is located.

The Panjkora River then flows down towards south receiving several small tributaries on the way till it confluences into the Swat River. Hence, the terrain of the Panjkora River Basin is in a northeast – southwest inclination. The Swat River then joins Kabul River in Peshawar city which flows further to finally confluence into the Indus River in Attock city.

The location of the project site is depicted in the figure below.

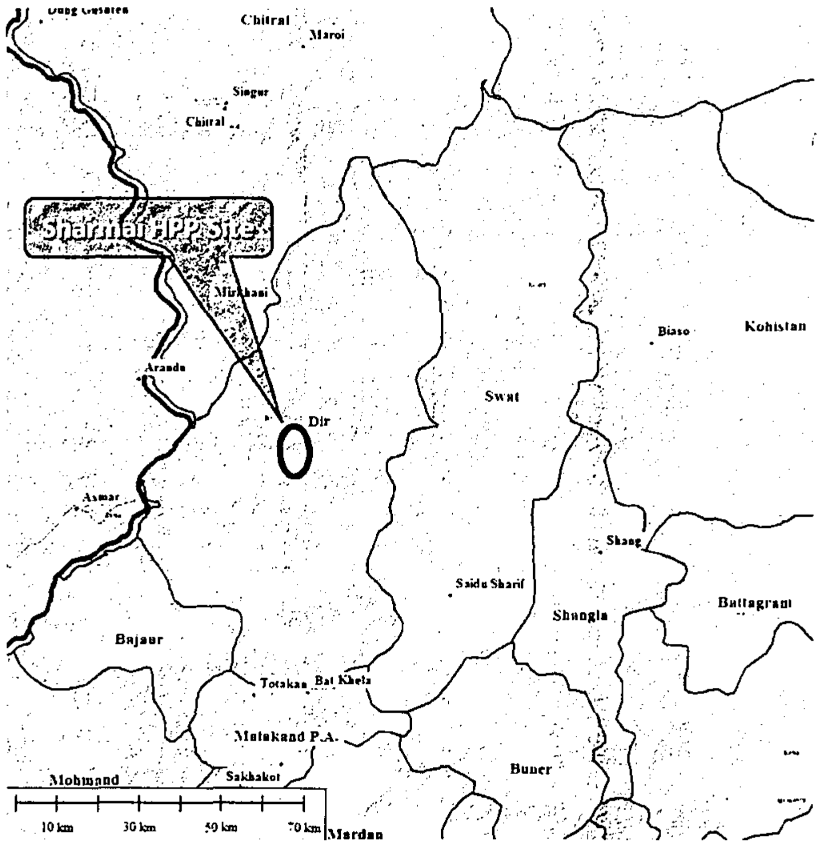


Figure 3-1: Location of the project site

3.2.2 Climatic conditions

In general, the climate of the project area is governed by two wind systems. One is the monsoon persisting during the summer months of June to September- the Easterlies; and the second system is the Westerlies which dominates in the winter months of November to March.

The climatic condition of the project area is mainly determined by the high altitude of surrounding mountains. According to the Koppen-Geiger climate clarification, one of the most widely used climate classification system, the project area mainly belongs to the "Dsb climate zone" which is explained as the area averaging below 0 °C, all months with average temperatures below 22 °C and at least four months averaging above 10 °C; at least three times as much precipitation in the wettest month of winter as in the driest month of summer, and driest month of summer receives less than 30 mm.

The main parameters describing the climatic conditions of the project area, i.e. the precipitation and temperature, are presented below with respect to the data measured at the "Dir station" which is the only meteorological station existing at project catchment area.

3.2.2.1 Precipitation

The distribution of precipitation during the year depends very largely on the topography of the area and season. The precipitation regime in the project area is dominated by the occurrence of eastward moving extra tropical zones of low pressure, also known locally as Western Disturbances. The Western Disturbances bring humidity to the project region from the Atlantic Ocean and the Mediterranean Sea. The Western Disturbances are more frequently and intense, during the Winter Season and they provoke the largest amount of precipitation over the project catchment. During the summer season the frequency and intensity of the Western Disturbances normally decrease, and the precipitation on the region also decreases. The intensity of monsoon summer rainfall is hence low.

The monthly average precipitation as observed at the existing Dir meteorological station, located near the proposed damsite, is shown in figure below. The annual average rainfall at the station shows 1,428 mm according to the observation data for the period from 1981 to 2006.

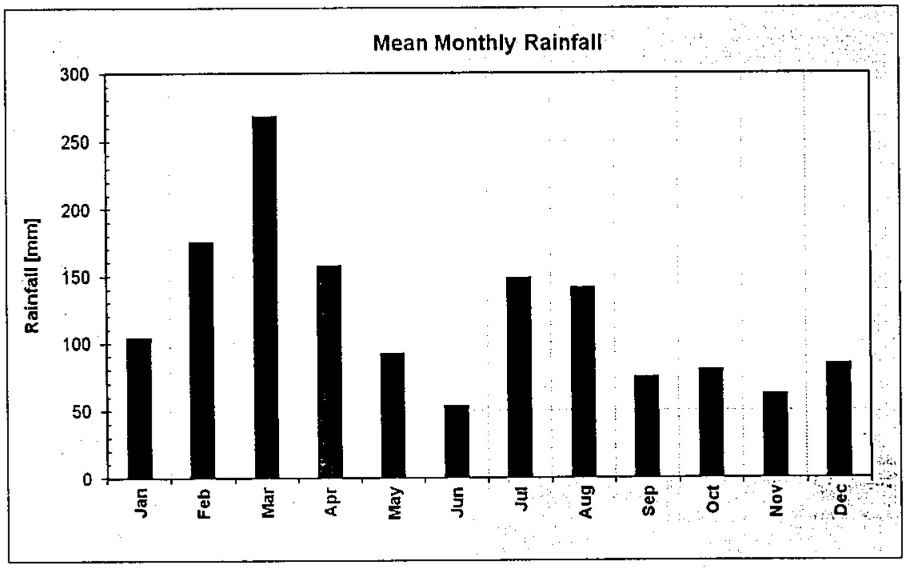


Figure 3-2: Monthly average rainfall observed at the Dir station

3.2.2.2 Temperature

The temperature regime follows the temperature pattern in the northern hemisphere. The temperature during the winter months from December to February falls below freezing point and during the summer months from June to August rises up to over thirty degrees. Also the difference on the temperature in a day seems comparatively large, and reaches around 15^{0}C in January and June. Figure below shows the variation of the average maximum, minimum and mean temperature during the year, as observed at the existing Dir meteorological station, located near the proposed damsite.

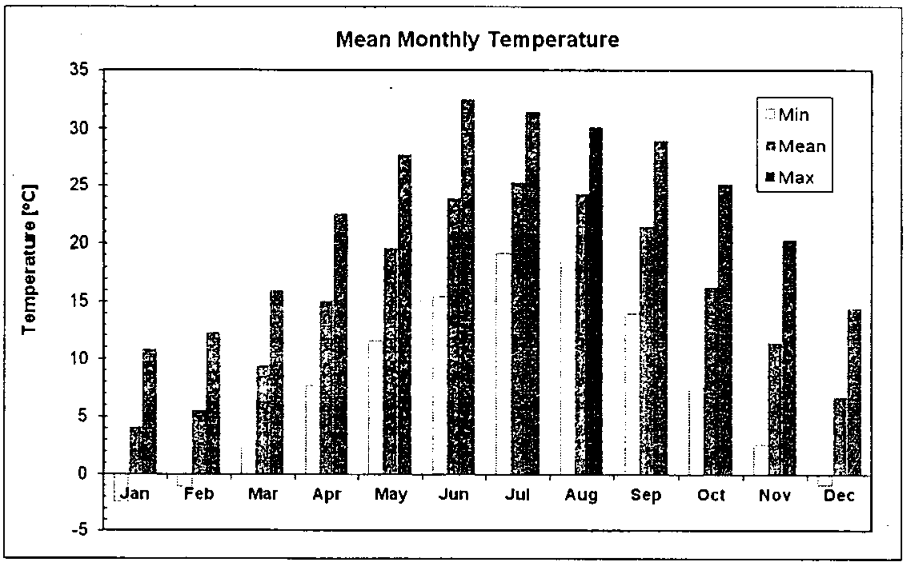


Figure 3-3: Monthly average maximum and minimum temperature observed at the Dir station

3.2.3 Hydrologic conditions

The hydrologic conditions at the project area are basically governed by its climatic conditions which are presented above in terms of the most important climatic variables- precipitation and temperature. The rainfall and temperature pattern at the project site (Panjkora River valley) are shown with the data observed at the Dir station (station located at the Panjkora River valley near damsite). Identical climatic patterns (precipitation and temperature) are observed and reported at the stations in other river valleys in the neighborhood of the project site like at Kalam in the adjacent Swat River valley, at Chitral in the nearby Chitral River valley etc. This indicates that the climatic conditions and hence the hydrologic conditions in the project region is similar and hence the observed data are transferrable from one to another.

Studies carried out for the Northern Areas of Pakistan, show that precipitation increases with the elevation up to certain elevation. Beyond this elevation the precipitation decreases or remains stable. The physical explanation of these phenomena is that precipitation increases as long as humidity is available to produce increasing amounts of precipitation. However, once a balance between the increasing rate of production of precipitation and availability of humidity is reached, the precipitation decreases or remains stable with the elevation, because the humidity does not provide sufficient water to produce increasing amounts of precipitation. The relative humidity observed in the project region is found to be comparatively constant throughout a year except May and June when it has comparatively low rainfall and high air temperature.

Distribution of precipitation with the elevation, in areas with important snow accumulation, may not be suitable to predict run-off, because an important process of mass transfer from higher to lower elevations takes place in the catchment. Large amounts of water in form of snow are transferred from higher to lower elevations through avalanches and mass movements. The run-off is produced at the place where the snow melts, and therefore precipitation does not give a precise indication of run-off origin. However, distribution of precipitation with the elevation accounts for the overall water balance in the catchment.

As can be observed from the precipitation and temperature pattern presented above, the period from September to November has low rainfall. The project area comes under heavy snowfall during the winter, which gets deposited and thus the flows are very nominal during winter. Snow starts falling at upper elevation in November and remains until April. When the temperature is at its maximum in July, flows are mostly snowmelt generated. It can be expected that largest flows or floods occur during this month.

The river flow is largely influenced by the snowmelts in the region from March to August along with effects by monsoon rains. High temperatures in addition to high precipitation during the previous winter result in high base flows.

This hydrologic condition or flow pattern as represented by mean monthly flows at different station in and nearby the project area can be seen in the figure below. The difference in magnitude is due to the difference in the respective catchment area (three times difference), but the hydrologic pattern is as explained above and are similar in both river valleys.

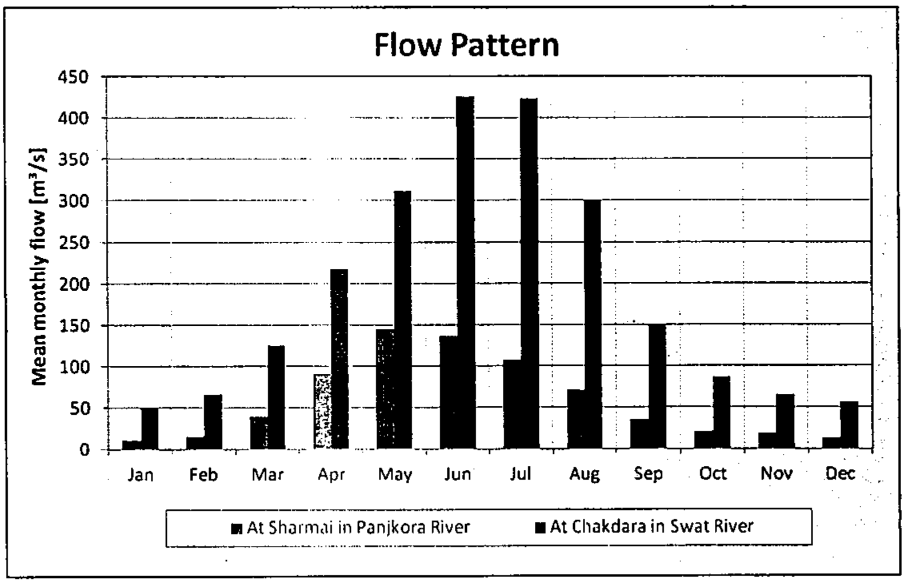


Figure 3-4: Monthly flow pattern observed at Panjkora River and the adjacent Swat River

3.2.4 Catchment characteristics

From the hydrological point of view, the relevant catchment characteristics includes catchment area, perimeter, maximum, minimum and average catchment elevation, maximum flow length, mean river slope, time of concentration etc. To delineate the catchment boundary and to estimate the relevant catchment characteristics, a "Digital Elevation Model (DEM)" which represents the continuous ground surface topography or terrain of the area has been used in "Geographic Information Systems (GIS)".

The Project site is located in the upstream reach of the Panjkora River which originates from a mountainous region (> 4000 m) and flows down to confluence into the Swat River which originates and flows almost parallel to the Panjkora River with adjacent catchment area/boundary. The drainage area or the catchment area of the project location, as well as that of the relevant hydrometric stations in the Pnajkora River and Swat River, is one of the most important parameter in carrying out the hydrological analysis for the site. The location of the different hydrometric stations available at the Panjkora River and the neighboring Swat River are shown in figure below. The records from Swat River and its tributaries like Gulshanabad in Gabral and Jildat in Ushu Rivers are also relevant and important for the hydrological estimation of the project site because Swat River is quite well investigated and has much longer records than at Panjkora River. The hydrology of the Swat River can be safely transposed to the Panjkora River as they share adjacent catchment area with identical climatic, topographic and geomorphologic conditions.

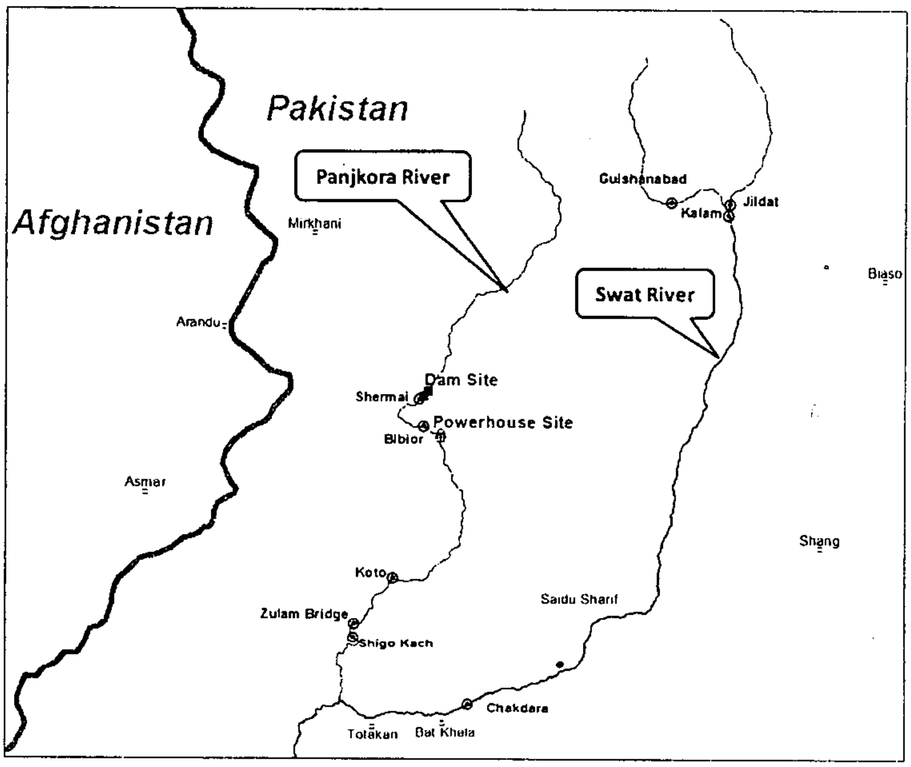
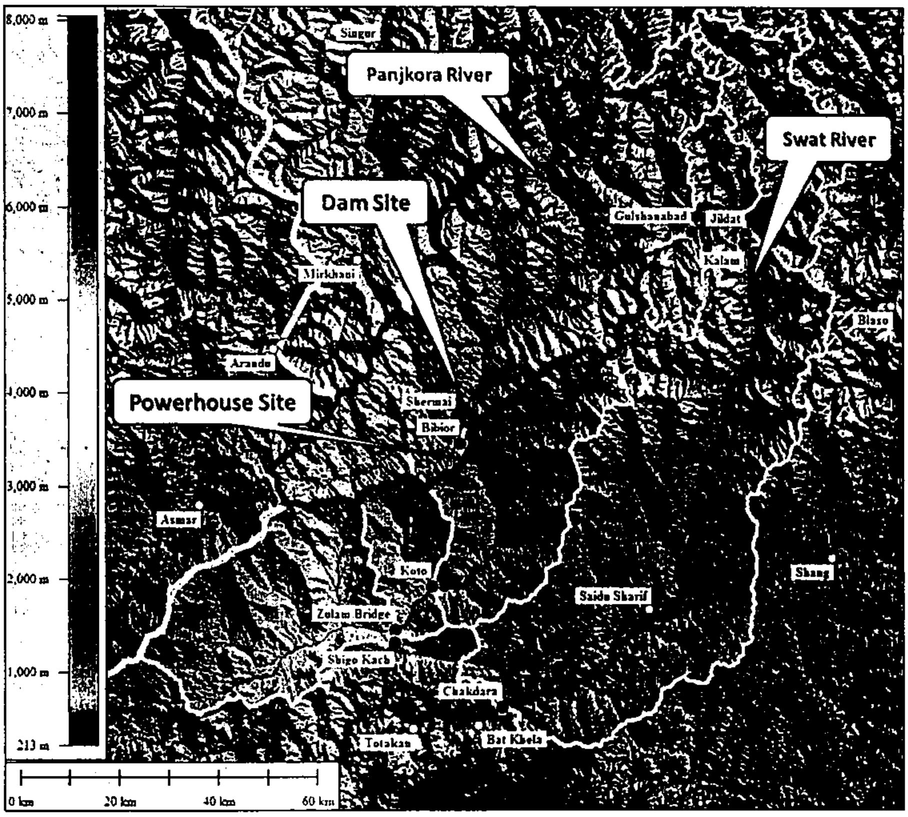


Figure 3-5: **Location of the relevant hydrometric stations**

By knowing the locations of the project site (proposed damsite and powerhouse site) and the relevant gauging stations, an estimation of the respective catchment area were done by using a Digital Elevation Model (DEM) of the area. The required DEM is obtained through CGIAR-CSI GeoPortal produced originally by NASA. The NASA Shuttle Radar Topographic Mission (SRTM) provides DEM in the spatial resolution of 1 arc second (about 30m x 30m).

The obtained version (SRTM plus v3) of the digital elevation data has been processed to fill data voids to provide seamless continuous topography surfaces using interpolation method described by Reuter et al. (2007) and made ready for input in the hydrological analysis.

Through the hydrological and terrain analysis of the DEM using Geographical Information System (GIS) capabilities, the catchment boundary for the project site and the relevant hydrometric stations is delineated, streams are extracted and other relevant catchment characteristics are estimated as shown in the figure and listed in the table below.



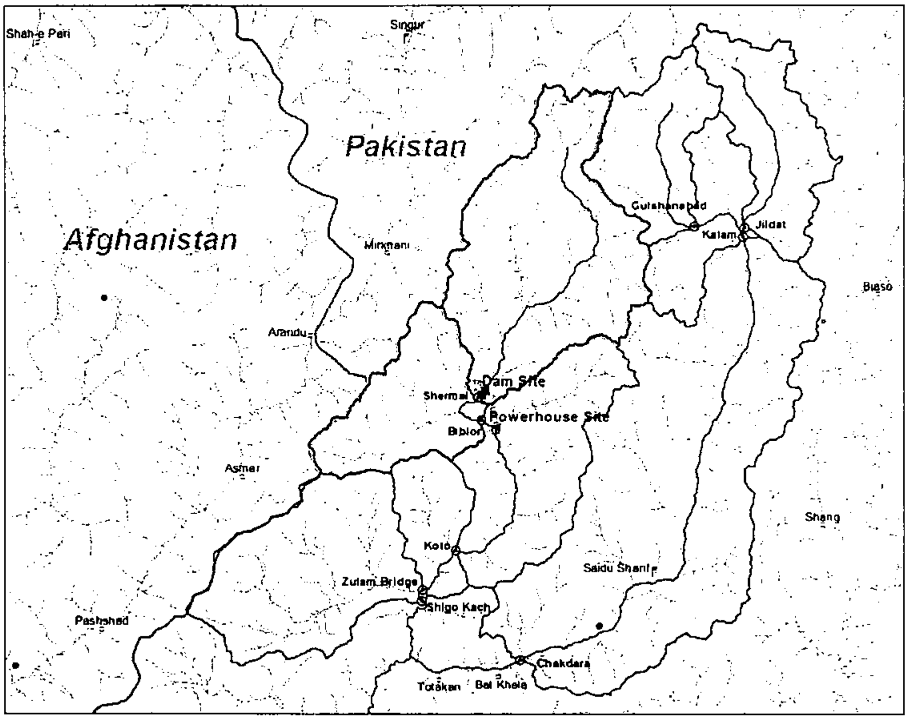


Figure 3-6: SRTM DEM of project area and the delineated catchment area of the project site and the relevant hydrometric stations

Table 3-1: Catchment characteristics of the project site and the relevant hydrometric stations

Name	Area	$Elev_{max}$	$Elev_{mean}$	$Elev_{min}$	L_{max}	MRS	t_c
[-]	[km²]	[m]	[m]	[m]	[km]	[%]	[hr]
Sharmai HPP Damsite	1,874	5,708	3,137	1,226	99	4.52	7.5
Sharmai HPP Powerhouse site	2,706	5,708	2,852	1,067	117	3.98	9.0
Relevant Hydrometric Stations							
Sharmai in Panjkora	1,878	5,708	3,133	1,212	100	4.49	7.6
Bibior in Panjkora	2,690	5,708	2,861	1,095	113	4.10	8.6
Koto in Panjkora	3,872	5,708	2,591	771	160	3.09	12.6
Zulam Bridge in Panjkora	4,285	5,708	2,476	680	174	2.89	13.8
Shigo Kach in Panjkora	5,646	5,708	2,187	657	177	2.85	14.1
Gulshanabad in Gabral	714	5,831	3,985	2,224	54	6.70	4.0
Jildat in Ushu	791	5,892	3,975	1,971	60	6.51	4.5
Kalam in Swat	2,041	5,892	3,881	1,949	70	5.64	5.3
Chakdara in Swat	5,752	5,892	2,706	680	211	2.47	17.0

where,

Area	:	Catchment area			
$Elev_{max}$	:	Maximum elevation in catchment	$Elev_{mean}$	:	Mean elevation in the catchment
$Elev_{min}$	:	Minimum elevation in the catchment	L_{max}	:	Maximum flow length
MRS	:	Mean River Slope	t_c	:	Time of concentration

As estimated using the SRTM 30m DEM and shown above, the catchment area at the damsite is found to be 1,874 km² and that at powerhouse site to be 2,706 km². The elevation varies from 1,226 masl at the damsite to a maximum of about 5,708 masl at the highest point of the catchment, the average elevation of the catchment being 3,137 masl. Topography of the project catchment area at damsite is further shown in figure below indicating different elevation ranges in the catchment area. Within the catchment area, the pattern of change in the drainage area with the change in elevation has been analyzed which provides a "hypsometric curve" as shown in figure below. This curve indicates that the drainage area of the project site is regularly steep with no significant flat area in between and therefore there are no slower flow-response locations within the catchment area. It also shows that about 90% of the catchment area lies above 2,000 masl, about 56% above 3,000 masl and about 17% above 4,000 masl. This indicates that there is considerable influence of snow-accumulation and snow-melt in the catchment area.

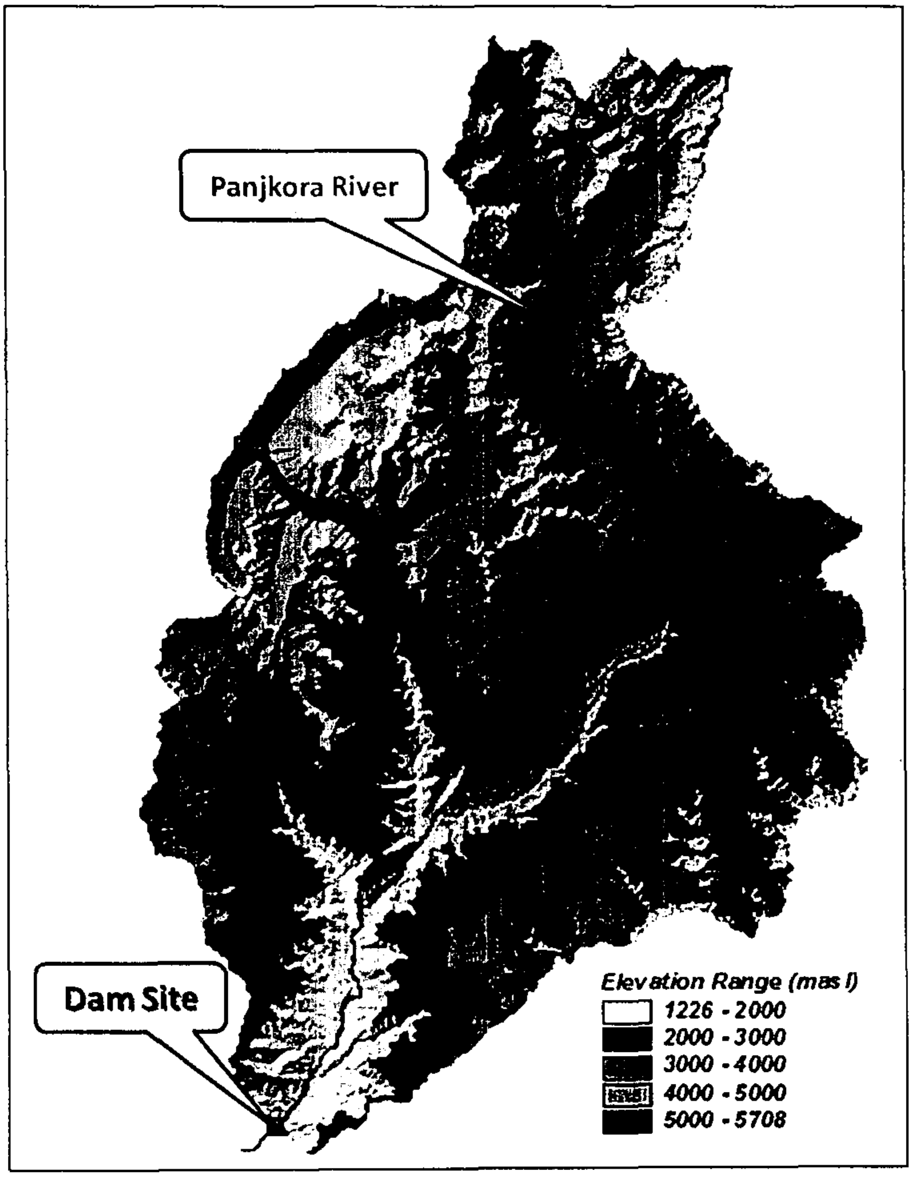


Figure 3-7: Catchment of project site indicating different elevation range

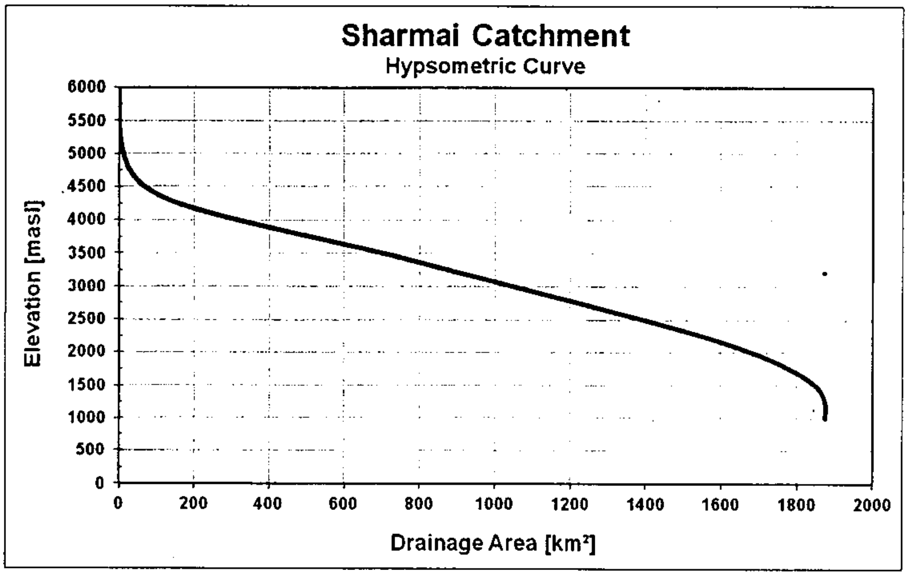


Figure 3-8: Hypsometric curve for the catchment area of the project site

3.3 Available Data and Information

Concerning the hydrological aspects of the project, the most important data are the measured flows/discharges. In this regard, the measured flow data series that are available for different hydrometric stations relevant to the Sharmai HPP project and considered for the analysis are shown in the table below. The location of these stations and their catchment characteristics are already presented in the figure and table above.

Table 3-2: The relevant hydrometric stations for which the measured flow data are available

Station	River	Basin	Installed date	Agency	Data Period	Resolution	Annual Instantaneous Peak
Sharmai	Panjkora	Kabul	16/1/2005	PEDO	2005 - 2016	Daily	not available
Bibior	Panjkora	Kabul	17/1/1995	SHYDO	1995 - 1999	Daily	available
Koto	Panjkora	Indus	17/01/2005	SHYDO	2005 - 2016	Daily	not available
Zulam Bridge	Panjkora	Indus	25/03/1999	SWHP	1999 - 2006	Daily	available
Shigo Kach	Panjkora	Kabul	19/11/2009	PEDO	2010 - 2016	Daily	not available
Gulshanabad	Gabral	Swat	1996	SWHP	2005 - 2015	Daily	available
Jildat	Ushu	Swat	1996	SWHP	2005 - 2015	Daily	available
Kalam	Swat	Indus	1961	SWHP	1961 - 2009	Daily	available
Chakdara	Swat	Indus	1960	SWHP	1961 - 2015	Daily	available

In addition, the following relevant documents are available whose information, in context to the hydrological aspects of the project, is also considered for the analysis.

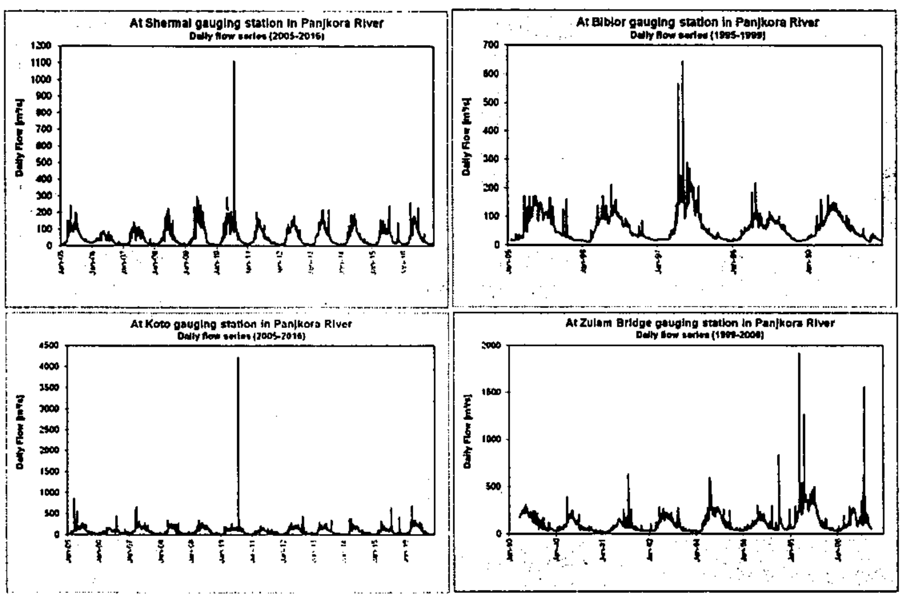
- *Feasibility Study for Sharmai Hydropower Project: Final Report*, June 2009

- *Sharmai Hydropower Project- Site Investigation Report*; By Zhongnan Engineering Corporation Limited, March 2017
- *Feasibility Study for the Madian Hydropower Project: Vol. II- Main Report and Vol. IV- Hydro-Meteorological Data base*; By FICHTNER, February 2009.
- *Gloen Gol Hydropower Project- Feasibility Study: Main Report and Final Design Report*; By Hydro Electric Planning Organization for Pakistan Water & Power Development Authority in collaboration with GTZ-German Agency for Technical Cooperation, July 1994 and 2007.
- Gabral-Kalam Hydropower Project- Hydrologic Study Report; By FICHTNER, 2005.

Based on these available data, documents and information, the hydrological analysis for the feasibility study of the Sharmai HPP project is carried out as described below.

3.4 Assessment of Available Data

The fore-most important hydrological data required for the different aspects of the hydrological analysis of a HPP site and to provide the results to be used for other part of the study/design are the available/measured flow series. In context to the Sharmai HPP project site, all the relevant hydrometric stations are shown and listed in the figure and table above along with the data available period at those stations. All the available measured daily flow series observed at those different hydrometric stations are shown in the figures below.



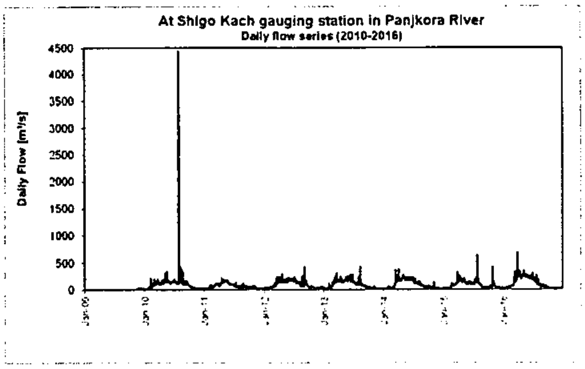


Figure 3-9: Available daily flow series observed at different hydrometric stations (1)

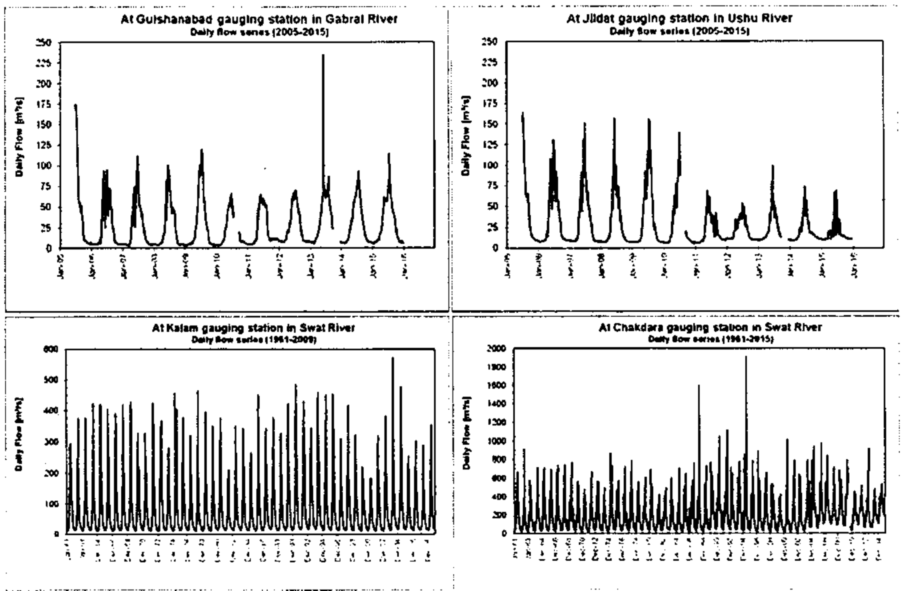


Figure 3-10: Available daily flow series observed at different hydrometric stations (2)

The first figure shows the daily flow series measured at five different hydrometric stations in the Panjkora River while the second figure shows the daily flow series measured at two hydrometric stations in the Swat River and two in its tributaries Gulshanabad and Jildat. It can be observed that there was extraordinary flood in July 2010 which is a very well reported flood in the region with catastrophic consequences. Among the data available, most of the stations have the record only from the year 2000 onwards and only the Kalam and Chakdara station in the Swat River has records starting from the year 1961. Therefore, theses records are important to enable the estimation of long-term hydrology at the project site.

At first, the consistency among the available daily flow series measured at the different hydrometric stations is assessed through the correlation coefficient between those data series. The estimated correlation coefficients are shown in the table below. The "n.a." in the table means "not applicable" because the data series from the two stations has no concurrent time period and hence the correlation cannot be estimated.

Table 3-3: Correlation coefficient of daily flow series observed at different hydrometric stations

	Sharmai	Bibior	Koto	Zulam Bridge	Shigo Kach	Gulshanabad	Jildat	Kalam	Chakdara
Sharmai	1	n.a.	0.78	0.72	0.80	0.66	0.63	0.73	0.79
Bibior	n.a.	1	n.a.	0.95	n.a.	n.a.	n.a.	0.52	0.72
Koto	0.78	n.a.	1	0.92	0.99	0.54	0.51	0.60	0.72
Zulam Bridge	0.72	0.95	0.92	1	n.a.	0.67	0.65	0.48	0.71
Shigo Kach	0.80	n.a.	0.99	n.a.	1	0.53	0.47	0.68	0.71
Gulshanabad	0.66	n.a.	0.54	0.67	0.53	1	0.84	0.97	0.83
Jildat	0.63	n.a.	0.51	0.65	0.47	0.84	1	0.98	0.84
Kalam	0.73	0.52	0.60	0.48	0.68	0.97	0.98	1	0.89
Chakdara	0.79	0.72	0.72	0.71	0.71	0.83	0.84	0.89	1

As can be seen, the observed flow series observed among the stations in the Panjkora River and among the stations in Swat River are quite highly correlated. Similarly the correlation between the flows observed in Panjkora River (at Sharmai, Koto, Shigo Kach) and that in the Swat River (mainly at Chakdara and also at Kalam) are also sufficiently high. The estimated correlation coefficients indicate that the measured flow series are, in general on average, consistent with each other.

Regarding the project Sharmai HPP in the Panjkora River, the most relevant hydrometric station is the Sharmai station itself which is located very close to the proposed damsite (about 100 m only). As already shown in the figure and table above, the Sharmai station and the project damsite has almost the same catchment area (only 4 km² difference, i.e. 0.2%) and hence the flow data measured at this station can be directly considered as the flows available at the project damsite. However, this station was installed on 16th Jan 2005 and the observed daily flow series are available since then till 2016. These 16 years of data series is not yet sufficiently long enough to derive a reliable long-term hydrology at the project site.
The other stations in the Panjkora River downstream also have only the short-term measured data series. For this, the measured data series from the other two stations, Kalma and Chakdara, located in the nearby Swat River (having adjacent catchment area with similar climatic, topographic and geomorphologic conditions, hence having similar runoff characteristics) which has relatively quite long daily flow series also becomes important to derive a more reliable long-term hydrology at the project site.

In this regard, the assessment of the short-term available flow series measured at the Sharmai station is carried out and the results are obtained as presented and explained below.

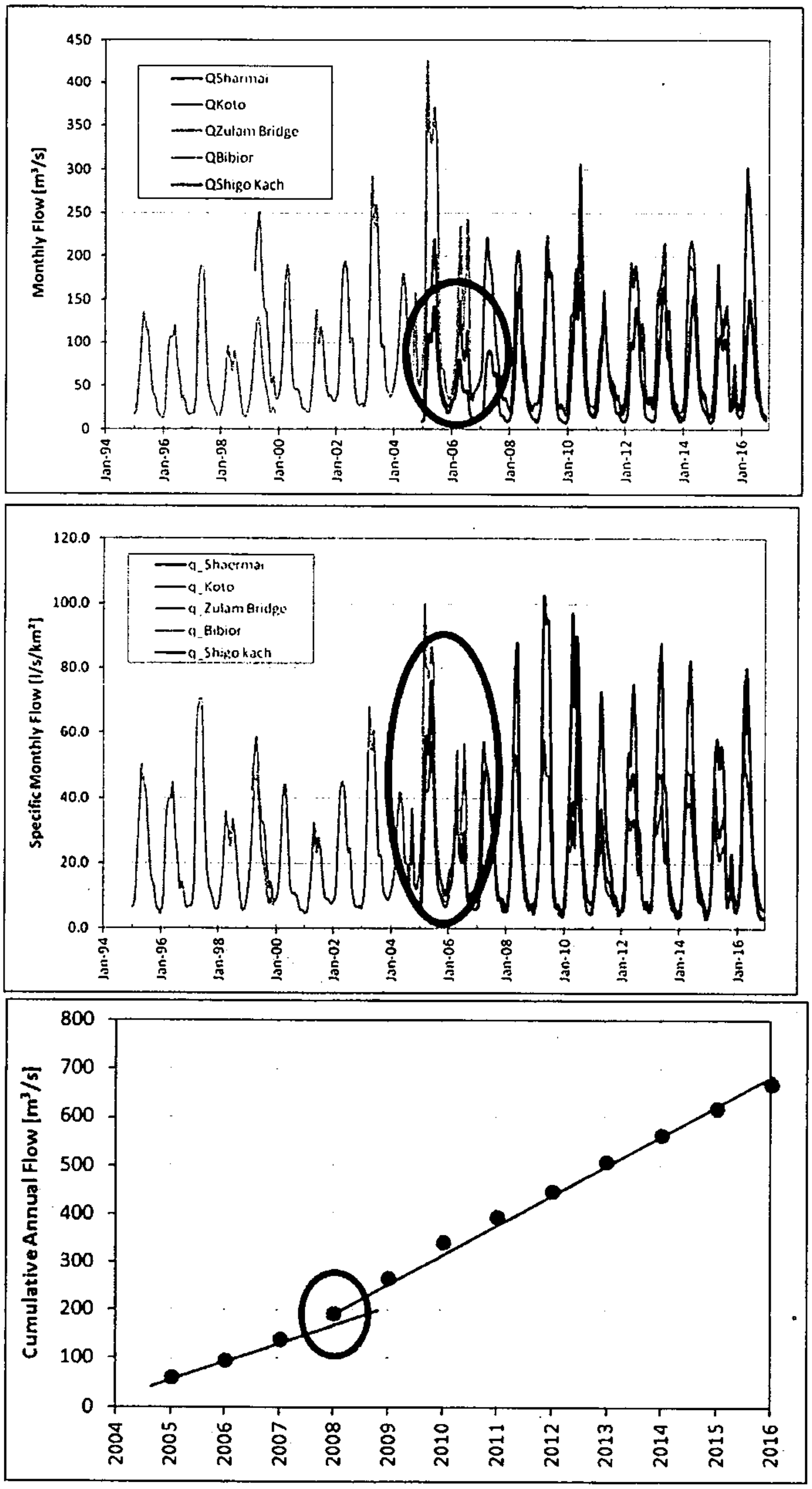


Figure 3-11: *Top*: Monthly flow series observed in Panjkora River at different stations; *Middle*: Specific monthly flow series observed in Panjkora River at different stations; *Bottom*: Single mass curve of annual flow series measured at Sharmai station

From the figures above, some discrepancies or inconsistencies in the available time series can be noticed as explained below:

- As shown by the plotted monthly flow series, the flow series measured before the year 2008 is quite low in comparison to that after the year 2008. The concurrent flow series measured at Koto station in the same River downstream does not show such miss-match.
- As shown by the plotted specific flow series, the higher specific flow at the upstream-most Sharmai station is maintained only after the year 2008 and not before (2005-2007).
- As shown by the plot of cumulative annual flow measured at Sharmai station, the flow series before the year 2008 is not consistent with that after the year 2008.

As the Sharmai gauging station was installed in the year 2005, probably the number of flow measurements is not sufficient enough and hence the rating curve of the station may not be reliable enough during the initial years of the installment. Large number of flow measurements in the year 2007 might have improved the rating curve and hence provides more consistent and reliable results since the year 2008. In this regard, the measured flow series at Sharmai station only from the year 2008 onwards is considered to be reliable enough to use in the hydrological analysis for the project.

Similarly, the assessment of the long-term available flow series measured at the Chakdara station is carried out and the results are obtained as presented and explained below.

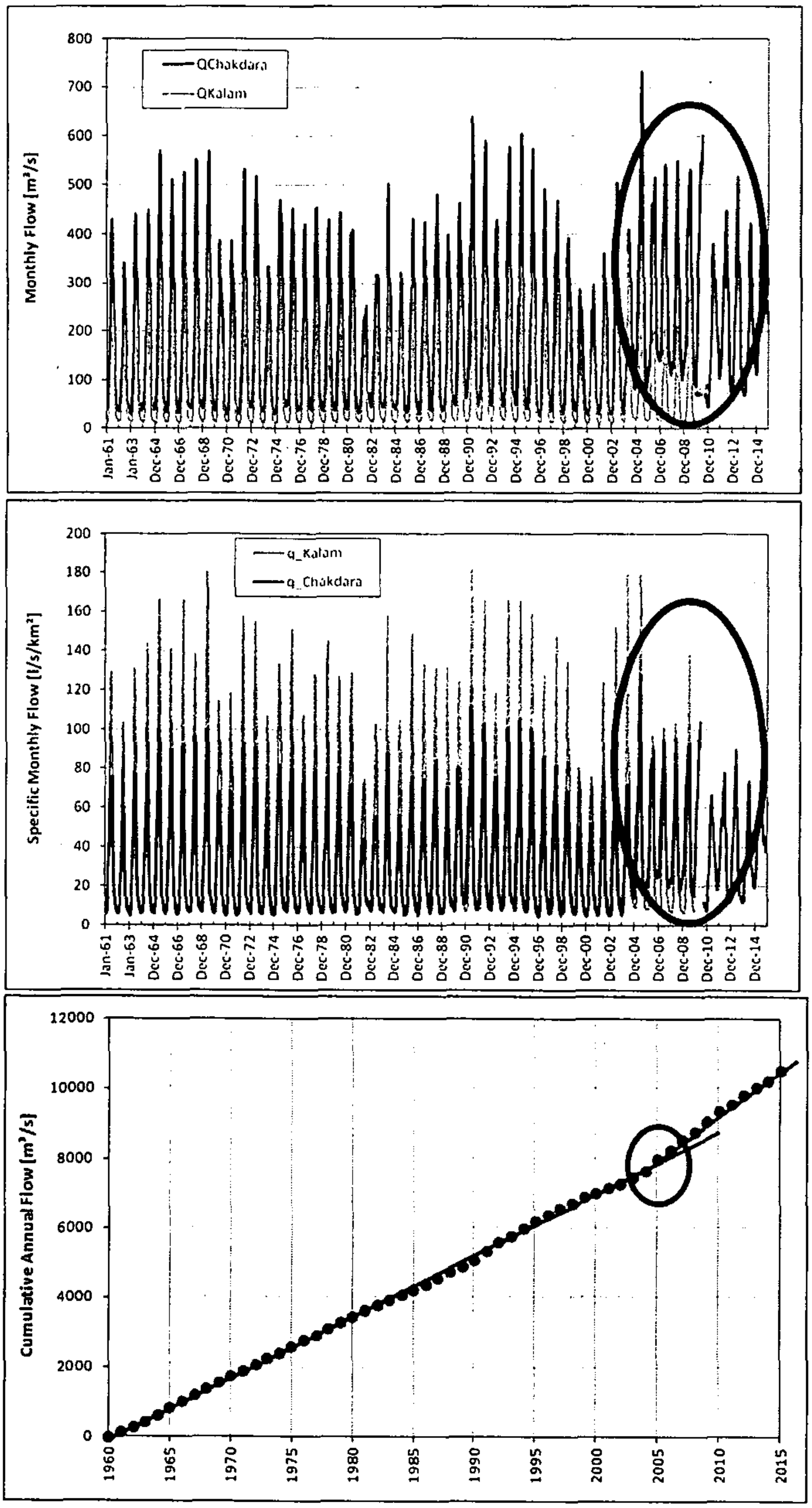


Figure 3-12: *Top*: Monthly flow series observed in Swat River at Chakdara and Kalam stations; *Middle*: Specific monthly flow series observed in Swat River at Chakdara and Kalam stations; *Bottom*: Single mass curve of annual flow series measured at Chakdara station

From the figures above, it is clear that there exist discrepancies or inconsistencies in the available time series which hare further explained below:

- As shown by the plotted monthly flow series, the flow series measured at the Chakdara station from the year 2004 onwards is quite high with a constant shift or jump in comparison to that before the year 2004. The concurrent flow series measured at Kalam station in the same River upstream does not show any such shift or jump.
- Specific discharge at higher elevation (Kalam) is higher than that at lower elevation (Chakdara) because of the contribution of snow melt concentrated in the area at higher elevation. As shown by the plotted specific flow series, the lower specific flow at the downstream Chakdara station is maintained only till the year 2003 and not from 2004 onwards.
- As shown by the plot of cumulative annual flow measured at Chakdara station, the flow series after the year 2003 (2004-2015) is not consistent with that before (1961-2003).

In this regard, the measured flow series at Chakdara station only till the year 2003 is considered to be reliable enough to use in the hydrological analysis for the project.

Based on these assessments of the plausibility and the reliability of the obtained flow series, the data available are used accordingly to estimate the long-term flows available for the Sharmai HPP project as described in following section.

3.5 Flows at the Project Site

As already stated above, the Sharmai station and the project damsite are located very close to each other and has almost same catchment area (only 4 km^2 difference, i.e. 0.2%) and hence the flow data measured at this station can be directly considered as the flows available at the project damsite.

The daily flow series are measured at this station only for the period of 2005-2016 from which the data for the period of 2005-2007 needs to be discarded as resulted from its plausibility and reliability assessment. These flow series directly measured at the project site are not sufficiently long enough, from the hydrological aspects, to estimate the possible long-term energy generation by the project with adequate reliability. However, the Swat River is hydrologically quite well investigated and has much longer records than at Panjkora River.

The Kalam and Chakdhara station in the Swat River has long term data series. The more recent data records from the tributaries to Swat River (Gulshanabad in Gabral and Jildat in Ushu) can further extend the data series at Kalam to more recent period. The hydrology of the Swat River represented by those stations can be safely transposed to the Panjkora River as they share adjacent catchment area with identical climatic, topographic and geomorphologic conditions.

3.5.1 Extension of flow series

In this regard- based on the location of the relevant stations in the Panjkora and Swat River, their data available period, the reliability and plausibility of their data and the correlation of the data between the stations - all as explained already- the daily flow series at the Sharmai station is extended to determine the long-term flows available at the project site by accomplishing the following steps sequentially.

Step 1:
The data from Chakdara station (1961-2015) needs to be corrected for the period from 2004 onwards for which its correlation/dependency with the data from Kalam station located upstream in the same Swat River can be utilized. But for Kalam station the data series is available only for the period of 1961-2009, which means only six years can be corrected. But the two other stations located at the tributaries just upstream of Kalam station, namely Gulshanabad in Gabral and Jildat in Ushu, has recorded daily flow series for the period of 2005-2015 which can be utilized first to extend the data series of Kalam station beyond 2009.

From the estimated catchment areas it can be noticed that the catchment area of these two stations constitute about 74% of the catchment area of the Kalam station. Moreover, the concurrent daily flow series between that of Kalam station and the combined flow series of Gulshanabad and Jildat stations shows very high correlative dependency with correlation coefficient as high as 0.98 and quite dispersion of data points as shown in the figure below.

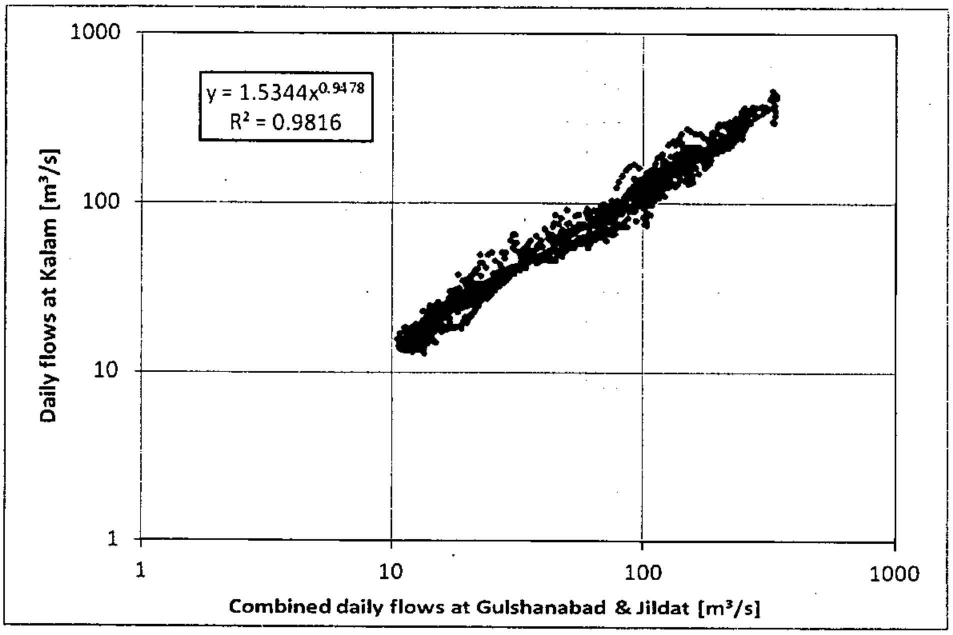


Figure 3-13: Correlative dependency between daily flow series of Kalam station and that of Gulshanabad and Jildat stations combined (2005-2009)

Using this correlative dependency the daily flow series at the Kalam station is extended as shown in the figure below.

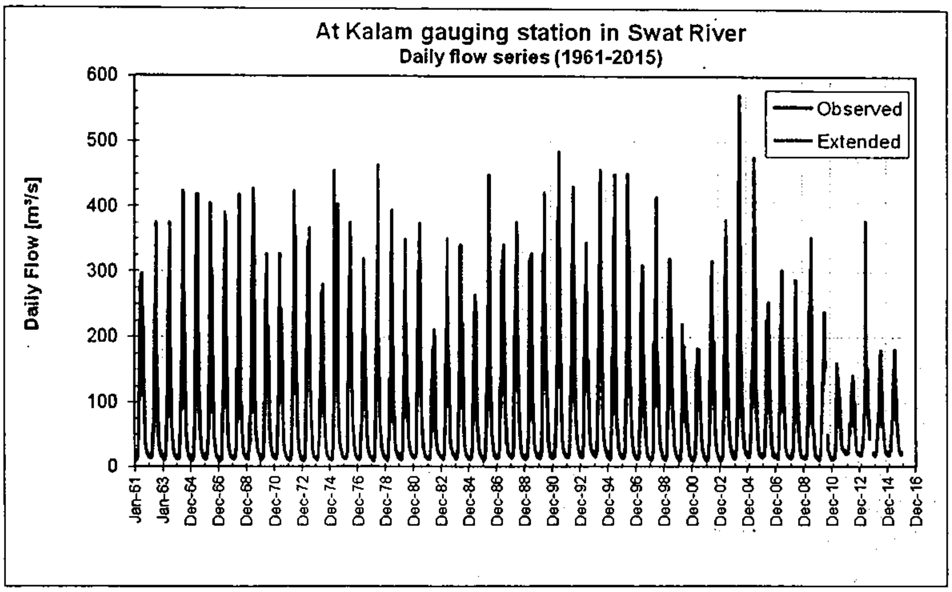


Figure 3-14: Observed and extended daily flow series at Kalam gauging station

<u>Step 2</u>:

The data from Chakdara station (1961-2015) needs to be corrected now for the period from 2004 onwards for which the extended data series from the Kalam station can be utilized. The 43 years long concurrent daily flow series (1961-2003) between the two stations show quite good correlation coefficient of 0.91 but their correlative dependency indicates quite dispersed data points as shown in the figure below. This dispersion is due to the fact that the dependency between Chakdara and Kalam flows are different in different period or flow regime.

The effect of snow melt runoff, which is in high flow period and comes from concentrated area at higher elevation, is much more pronounced in the higher Kalam station than at the lower Chakdara station, where as the low flow period is more or less uniformly distributed in the entire catchmen tarae of the Chakdara which encompasses that of the Kalam station as well.

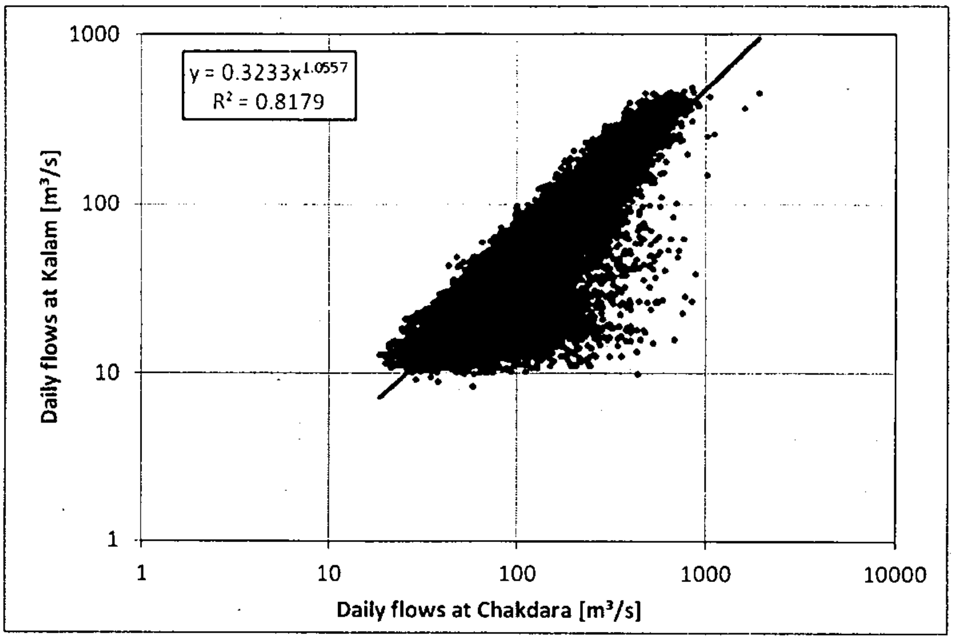


Figure 3-15: Correlative dependency between daily flow series of Kalam station and Chakdara station (1961-2003)

Due to such excessive dispersion of the data points, the resulting regression equation cannot be used to correct the Chakdara station data using Kalam station data because with the regression line the dispersion/variation will be averaged out and the higher values will be underestimated and the lower values will be overestimated.

This is exactly adverse for the HPP project because energy will be overestimated and the flood will be underestimated. This dispersion occurs mainly because the dependency between the two flow series is different during different period of the year like during low flows and during high flows as can be seen in the following figure.

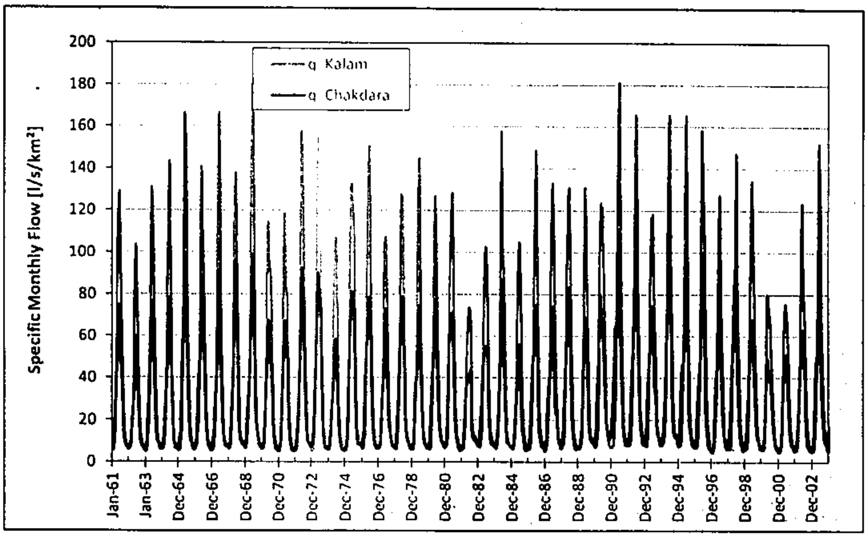


Figure 3-16: Specific monthly flow series observed at Chakdara and Kalam stations (1961-2003)

It can be observed that the difference in the specific flows between Kalam and Chakdara is quite low during low flow period and quite high during high flow period. To account for this variation, the average ratio of the specific flows (or flows) between the two stations is calculated for each month. These month-wise ratios are then used to extend/correct the daily flow series of Chakdara for the period from 2004 onwards using the data series of that period from the Kalam station.

The cross-check of the flow series estimated with such month-wise ratios for the period of 1961-2003 with the observed series of that period at the Chakdara station shows good matching which could not be obtained with the regression equation due to over and under estimation in different period of a year. The resulting daily flow series at the Chakdara station is shown in the figure below.

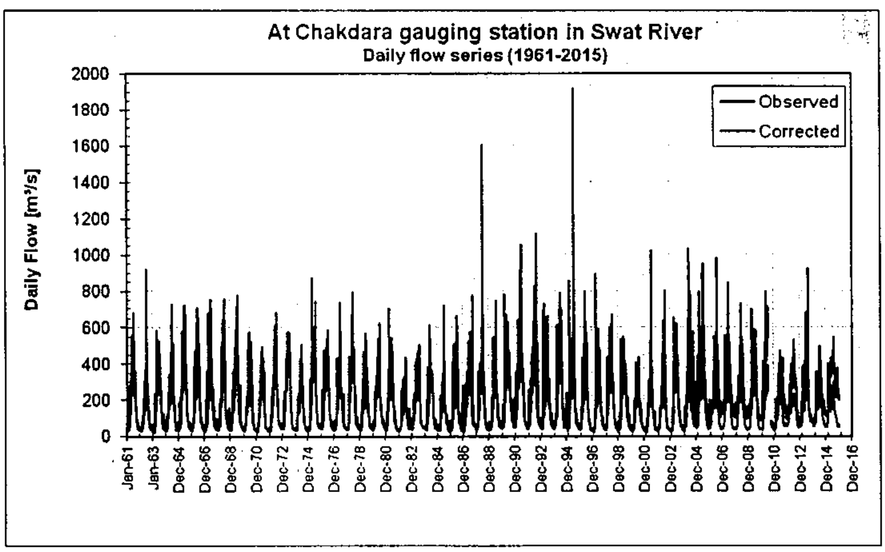


Figure 3-17: Observed and corrected daily flow series at Chakdara gauging station

Step 3:

The corrected long-term flows from Chakdara station in Swat River are then transposed to the Koto station in Panjkora River. The Koto station has more recent and longer time series of daily flows of Panjkora River as in Sharmai and is also closer to the Chakdara station in terms of location, altitude, catchment area than Sharmai (as already shown in the respective tables above). As for transposing flows between Kalam and Chakdara station, transposing here is also done by estimating the average ratio of the specific flows (or flows) between the two stations for each month from the concurrent flow series of 2005-2015.

These month-wise ratios are then used to extend the daily flow series of Koto for the period of 1961-2004 using the data series of that period from the Chakdara station. The cross-check of the flow series estimated with such month-wise ratios for the period of 2005-2015 with the observed series of that period at the Koto station shows good matching. The resulting extended daily flow series at the Koto station is shown in the figure below.

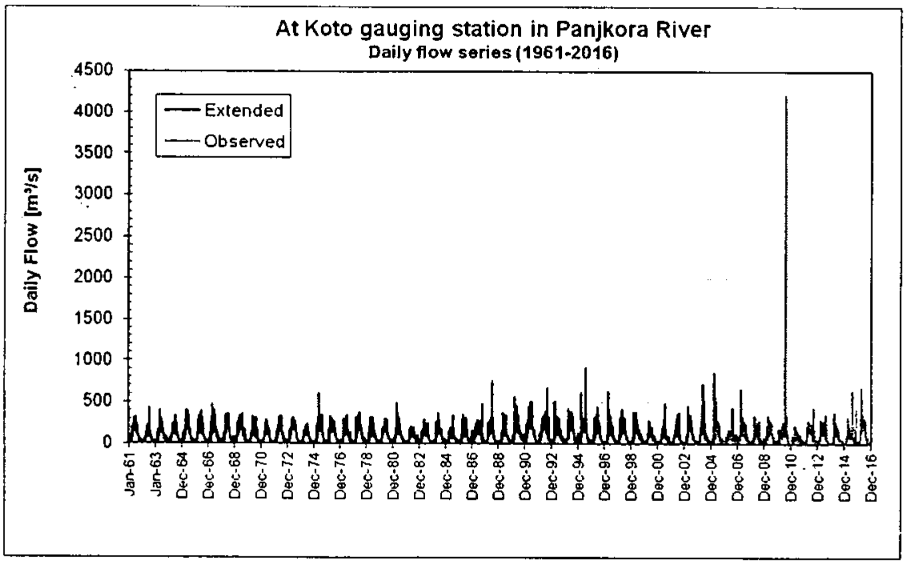


Figure 3-18: Observed and extended daily flow series at Koto gauging station

Step 4:

The extended daily flow series at Koto gauging station in the Panjkora River is then finally transposed to the Sharmai station based on the concurrent daily flow series between the two in the period of 2008-2016. The two stations in different altitude (Koto at 771 masl and Sharmai at 1212 masl) here have quite different specific runoff (higher at Sharmai than at Koto) and therefore the flows cannot be transposed simply by using their catchment area ratio which assumes the constant specific runoff throughout the catchment area.

The two concurrent flow series are quite good correlated (R=0.8) but the data points are dispersed along the regression line and hence the transposing using the regression equation will not capture the variability of flows. Further, the difference of flows or specific flows between the two stations is different in different seasons or months of the year. The difference is higher during high flows and negligible during low flows. Therefore a constant ratio of flows or specific flows also cannot be used for the transposing.

Hence, as used earlier, here also transposing the flows from Koto station to Sharmai in the Panjkora River is done by estimating the average ratio of the specific flows (or flows) between the two stations for each month from the concurrent flow series of 2008-2016. These month-wise ratios are then used to extend the daily flow series of Sharmai for the period of 1961-2007 using the data series of that period from the Koto station.

The cross-check of the flow series estimated with such month-wise ratios for the period of 2008-2016 with the observed series of that period at the Sharmai station shows good matching both during high and low flow period. The resulting extended daily flow series at the Sharmai station is shown in the figure below.

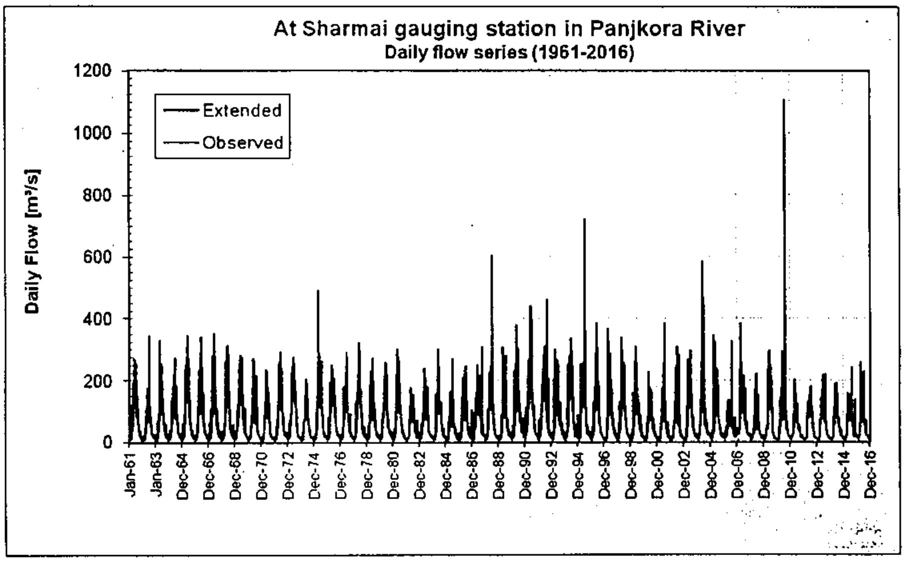


Figure 3-19: Observed and extended daily flow series at Sharmai gauging station

This long-term daily flow series (1961-2016) estimated at the Sharmai gauging station forms the hydrological data basis for the project.

3.5.2 Assessment of estimated flow series

The daily flow series (1961-2016) estimated at the Sharmai gauging station shows that the long-term mean flow at the site is 71.4 m³/s corresponding to the specific runoff of 38 l/s/km². These flows are estimated using the flows from Gulshanabad in Gabral River, Jildat in Ushu River, Kalam in Swat River, Chakdara in Swat River, Koto in Panjkora River and the Sharmai in Panjkora River.

As a cross-check, the estimation directly from the Kalam station (which has similar catchment area as that of Sharmai) shows that the long-term average flow at Sharmai is 75.5 m³/s if the catchment area ratio method is used and it will be 74.1 m³/s if the month-wise ratios of specific flows are used. It indicates that the estimated long-term flows seem to be plausible on average.

Further, the graphical check through the single mass curve, i.e. the plot of cumulative annual flow, as shown in figure below, indicates that the long-term flow records estimated at Sharmai station are consistent, homogenous and steady and hence they are plausible for using it in the hydrological analysis to determine different hydrological design parameter for the Sharmai HPP project site.

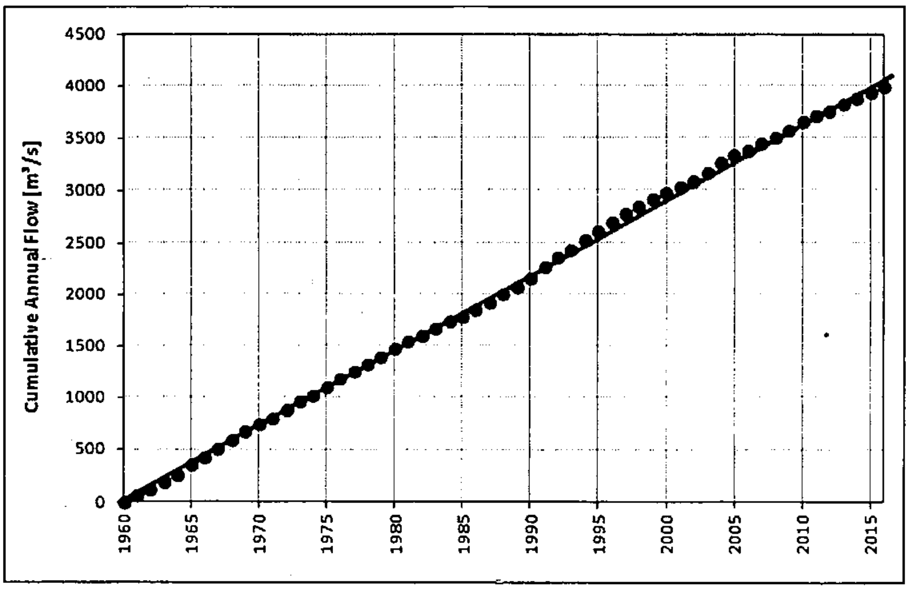


Figure 3-20: Plausibility check of the long-term flow records estimated at Sharmai station

3.5.3 Flows available at site

As mentioned earlier, the flows at the Sharmai station can be directly considered as the flows available at the project damsite due to its proximity to the proposed damsite. The distance between the damsite and station is only about 100 m with no tributaries in-between and the catchment area differs only by 4 km² i.e. 0.2%- therefore, any increase of flow between the damsite and the station is negligible. Hence the flows at the Sharmai station estimated and assessed as explained above represents the flows available for the project for the energy generation, which are presented below.

3.5.3.1 Daily flows

The daily flow series estimated at the project site is shown in figures below. During the estimated period (1961-2016), the daily flows at the project site vary from 5.9 m³/s to 1109 m³/s with the average value of 71.4 m³/s. The power and energy output of the project are calculated based on these daily flow series.

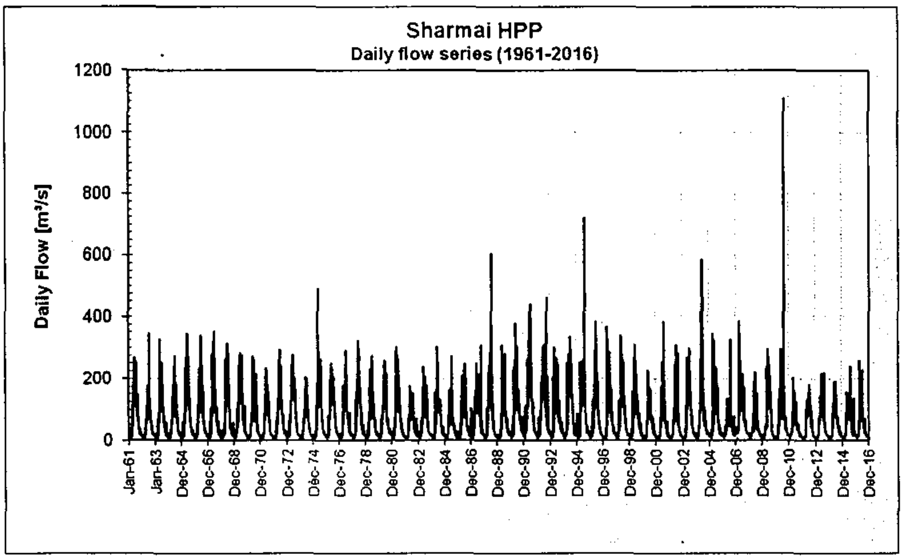


Figure 3-21: Daily flow series at the project site

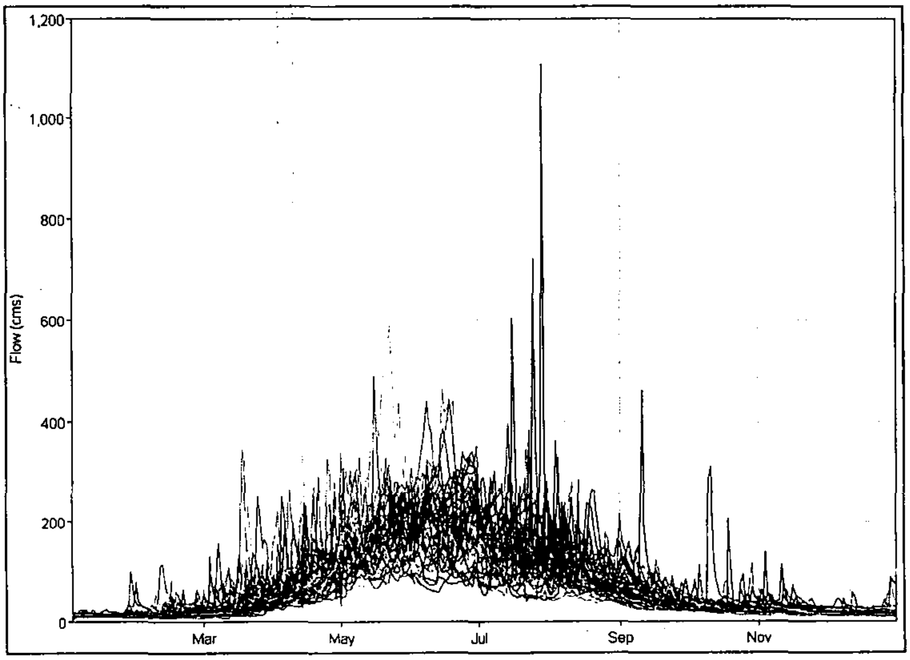


Figure 3-22: Daily flows of different years at the project site

These daily flows are averaged over each 10 days and the mean of theses 10 days average daily flow series is also calculated as shown in table below.

Table 3-4: Mean of ten days average daily flows at project site

Month	1st 10 days	2nd 10 days	3rd 10 days
Jan	11.6	11.3	12.0
Feb	13.5	16.6	17.3
Mar	21.7	33.6	43.3
Apr	67.0	86.0	103.0
May	146.6	162.2	179.0
Jun	178.4	191.2	198.1
Jul	154.3	146.3	134.6
Aug	115.0	96.9	75.5
Sep	65.4	50.5	38.6
Oct	30.2	27.1	23.7
Nov	24.7	23.0	20.7
Dec	16.2	15.8	15.7

3.5.3.2 Monthly flows

The daily flow data series at the project site are aggregated month-wise to analyze the monthly pattern. The resulting monthly flow series at the project site is shown in the figure below. The series along with their simple statistics are also listed in table below. The variation of monthly discharges over a year on average can be seen in the figure below.

As shown, the monthly flow at the project site varies from 8 m^3/s to 311.7 m^3/s during the period. The average of mean monthly flows at the site varies from 11.6 m^3/s on January to 189.2 m^3/s on June.

Monthly pattern of the average flow shows that the wettest period or major high flows occur during May to July while the period during November to February is comparatively a low flow or drier period.

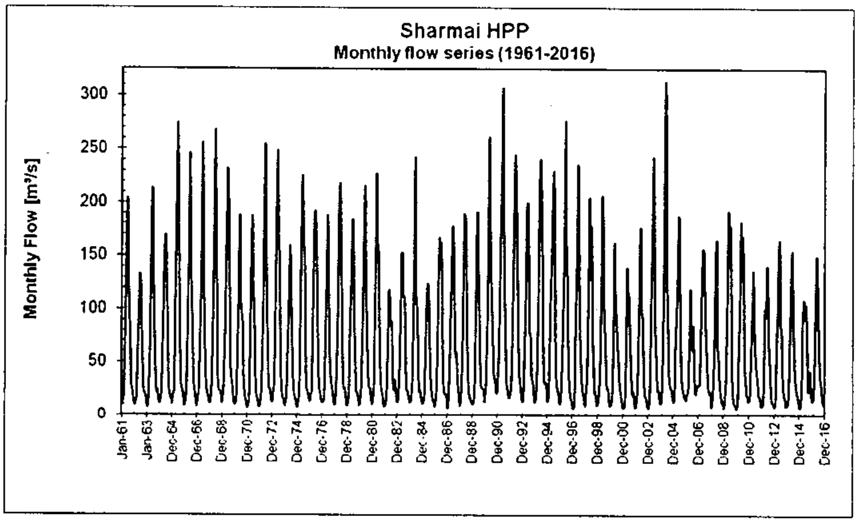


Figure 3-23: Monthly flow series at the project site

Table 3-5: Monthly and annual flow series and their statistics at the project site

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1961	11.3	12.8	17.8	65.1	154.4	204.2	161.8	95.7	81.7	31.1	23.8	15.2	**72.9**
1962	10.1	11.3	15.5	48.3	93.9	133.3	129.3	84.0	48.3	22.5	20.8	14.9	**52.7**
1963	8.9	8.9	27.8	77.0	182.6	212.9	142.5	88.3	47.5	22.2	24.8	13.5	**71.4**
1964	11.7	14.7	25.2	71.9	132.7	169.5	169.4	113.3	63.6	26.0	17.8	15.4	**69.3**
1965	11.2	20.0	23.9	108.7	209.6	275.1	198.3	104.5	51.7	26.6	23.9	14.4	**89.0**
1966	9.7	14.9	32.0	96.7	168.9	246.2	152.6	107.0	69.5	31.6	22.3	13.4	**80.4**
1967	9.3	16.3	25.4	72.4	132.7	254.3	197.3	100.3	59.1	28.6	21.2	17.6	**77.9**
1968	11.7	12.9	24.0	69.0	147.6	266.9	206.5	123.8	42.5	24.7	23.1	20.6	**81.1**
1969	11.8	17.8	41.5	81.8	159.7	231.7	213.9	140.4	56.6	36.5	28.1	15.3	**86.3**
1970	10.7	11.3	19.4	60.9	143.8	187.8	110.2	94.1	84.9	31.4	20.0	12.6	**65.6**
1971	8.1	9.2	15.4	67.3	163.2	187.3	120.8	94.9	42.5	19.2	16.2	10.7	**62.9**
1972	8.7	13.6	29.3	69.8	174.2	255.5	170.6	106.0	62.2	26.7	25.6	17.0	**79.9**
1973	12.7	18.4	32.6	87.3	192.8	248.7	173.1	142.5	72.0	28.3	18.6	13.9	**86.7**
1974	9.9	13.0	23.4	64.6	111.7	159.4	126.4	77.6	36.5	21.7	16.4	12.9	**56.1**
1975	8.3	11.4	24.2	86.6	199.9	225.6	165.3	148.1	69.6	27.7	23.0	17.7	**83.9**
1976	13.6	18.7	26.1	96.6	180.4	192.7	170.0	119.6	61.6	28.1	20.9	13.8	**78.5**
1977	12.9	13.2	17.8	69.6	129.0	187.6	157.4	80.4	46.2	31.6	24.9	15.9	**65.5**
1978	11.1	11.8	36.4	80.1	181.2	218.7	170.3	110.8	45.9	25.5	26.9	15.1	**77.8**
1979	10.1	13.2	22.7	86.6	131.9	183.8	160.7	91.8	55.7	23.2	21.6	14.0	**67.9**
1980	10.9	16.0	39.2	89.8	174.6	215.1	135.2	82.6	47.4	28.0	26.2	16.6	**73.5**
1981	11.3	15.8	35.8	123.8	227.7	164.0	153.5	85.6	43.5	25.1	20.5	11.6	**76.5**
1982	9.4	11.0	20.6	59.6	117.9	111.9	84.6	88.9	30.6	24.2	34.5	20.2	**51.1**
1983	12.8	15.4	34.0	70.8	152.4	153.6	112.4	112.2	67.1	26.8	23.5	17.6	**66.5**
1984	12.7	14.2	20.1	54.7	141.3	242.8	117.5	100.4	62.7	23.0	24.5	17.1	**69.3**
1985	12.1	12.7	16.0	48.1	104.6	124.2	121.3	77.6	38.4	24.5	14.6	14.2	**50.7**
1986	9.5	15.5	31.7	88.4	165.4	146.0	162.3	113.6	37.7	23.7	19.9	20.1	**69.5**
1987	8.3	11.6	39.5	82.4	140.7	177.8	158.9	92.8	55.0	59.4	31.6	18.1	**73.0**
1988	10.4	14.3	37.9	94.6	189.8	188.6	179.8	95.3	43.8	22.4	17.1	13.3	**75.6**
1989	12.7	11.7	17.6	44.5	159.4	191.5	130.9	90.8	45.6	26.9	26.4	24.4	**65.2**
1990	13.5	23.3	53.7	113.8	259.9	192.0	137.7	100.3	69.3	39.2	33.2	23.8	**88.3**
1991	21.6	43.6	65.8	153.0	214.4	307.3	203.0	119.5	80.2	33.4	24.9	17.1	**107.0**
1992	17.5	22.6	41.1	101.5	186.6	244.2	221.8	148.2	95.5	44.9	31.1	18.5	**97.8**
1993	13.9	14.7	48.8	99.4	186.4	199.5	160.9	79.4	74.4	35.3	33.3	17.9	**80.3**
1994	13.0	18.1	36.6	80.0	200.4	240.6	216.7	141.4	67.4	31.2	32.4	28.3	**92.2**
1995	14.3	17.4	53.7	110.8	148.6	206.5	228.0	143.2	48.2	30.3	23.7	16.8	**86.8**
1996	11.7	18.2	46.7	94.0	165.0	276.6	163.1	122.5	53.6	28.1	18.6	12.5	**84.2**
1997	8.3	8.4	18.1	93.0	149.4	235.9	159.9	89.4	50.4	29.1	21.4	13.5	**73.0**
1998	10.5	25.4	42.2	110.4	204.4	150.4	176.3	85.4	48.6	23.2	19.0	11.2	**75.6**
1999	12.8	19.4	38.5	86.0	204.9	189.9	113.3	80.3	54.4	26.9	26.9	14.0	**72.3**
2000	10.4	11.7	17.3	58.8	161.5	110.6	93.5	68.6	52.5	29.5	22.1	13.6	**54.2**
2001	8.9	8.5	11.4	44.7	137.8	118.1	111.3	69.5	45.0	20.2	23.6	12.6	**51.0**
2002	8.7	13.1	25.3	67.6	154.3	175.7	100.8	91.2	41.5	18.8	17.3	11.7	**60.5**
2003	8.3	9.6	33.2	110.2	177.2	243.2	166.1	76.6	49.8	23.9	25.2	14.0	**78.1**

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
2004	12.3	16.7	51.7	135.3	311.7	295.0	123.5	82.4	41.6	27.2	26.3	19.3	**95.3**
2005	13.8	18.1	63.8	94.8	133.0	187.7	137.5	69.6	34.4	25.6	19.7	16.6	**67.9**
2006	15.7	21.6	23.1	57.8	119.9	75.2	62.7	86.5	39.6	21.3	27.2	29.5	**48.3**
2007	29.3	31.4	65.3	128.7	157.0	154.4	109.1	71.3	51.3	27.8	21.4	23.2	**72.5**
2008	9.6	15.1	31.4	70.6	149.7	163.7	66.6	52.9	29.9	19.8	13.6	12.6	**53.0**
2009	8.7	18.6	37.5	91.3	191.1	176.3	178.4	103.1	31.0	14.7	11.5	9.7	**72.7**
2010	8.0	15.1	55.4	101.8	182.4	127.5	169.0	123.7	52.8	31.5	19.3	15.8	**75.2**
2011	14.8	21.8	52.3	94.0	137.1	95.9	61.2	44.3	35.7	20.0	19.5	12.1	**50.7**
2012	9.9	11.2	25.7	101.2	94.1	140.8	123.7	64.1	45.2	17.8	13.3	11.6	**54.9**
2013	9.5	13.2	50.9	92.8	150.7	163.5	87.7	77.6	33.6	21.4	16.1	10.3	**60.6**
2014	10.6	14.0	35.0	77.0	135.9	154.0	95.9	53.8	33.2	23.7	19.1	12.7	**55.4**
2015	8.6	11.4	30.2	93.6	109.5	92.2	105.3	83.2	23.2	24.5	43.6	20.4	**53.8**
2016	14.9	16.9	33.8	99.2	150.2	122.7	77.0	39.0	32.6	19.3	13.6	11.4	**52.5**
***Statistics*:**													
Mean	11.6	15.7	33.2	85.3	163.2	189.2	144.7	95.2	51.5	26.9	22.8	15.9	**71.4**
Min.	8.0	8.4	11.4	44.5	93.9	75.2	61.2	39.0	23.2	14.7	11.5	9.7	**48.3**
Max.	29.3	43.6	65.8	153.0	311.7	307.3	228.0	148.2	95.5	59.4	43.6	29.5	**107.0**

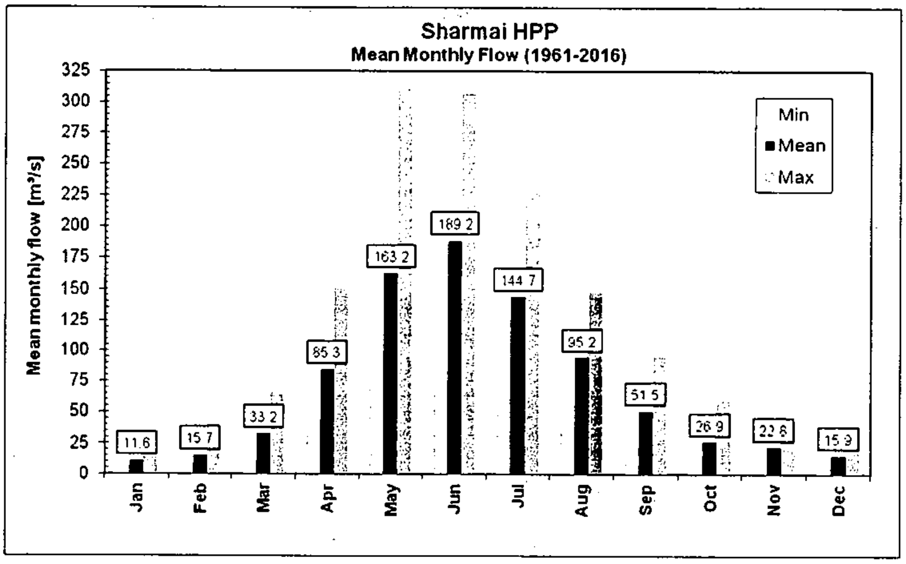


Figure 3-24: Mean monthly flow at the project site

3.5.3.3 Annual flows

The monthly flow data series are aggregated to annual flows which are also listed in table above. During the considered period (1961-2016) the mean annual flow at the project site varies from minimum of 43.8 m³/s in the year 2006 to the maximum of 107 m³/s in the year 1991.

The mean annual inflow (MQ) at the project site is 71.4 m³/s which corresponds to the specific runoff of 38 l/s/km². The annual flow series at the project site is shown in figure below.

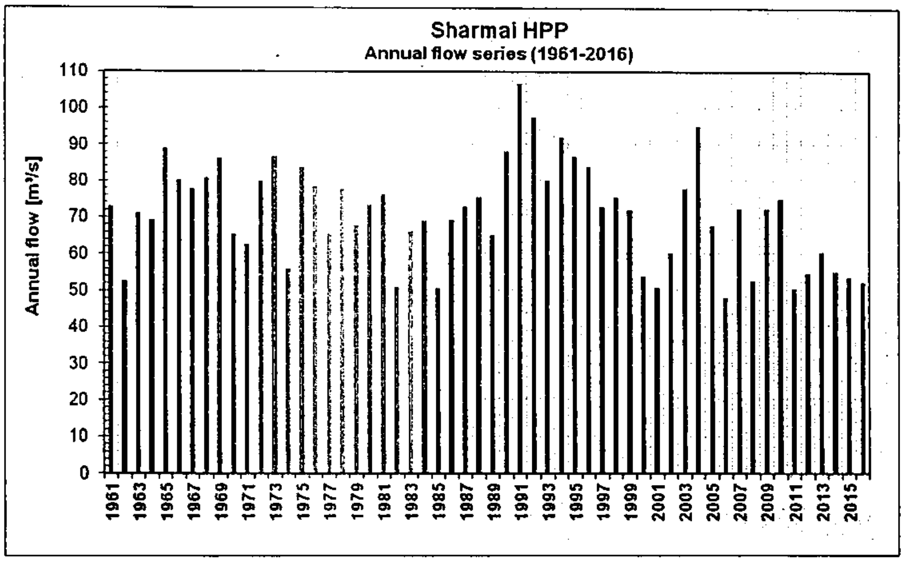


Figure 3-25: Annual flow series at the project site

3.5.3.4 Maximum and minimum flows

The yearly analysis of maximum and minimum daily flows for the considered period (1961-2016) at the project site is presented in table and shown in figures below. It shows that the annual maximum daily flow (HQ) varies from 175.9 m³/s in the year 1982 to the maximum of 1,109 m³/s in the year 2010 while the annual minimum daily flow (NQ) varies from 5.9 m³/s in the year 2001 to the maximum of 20.3 m³/s in the year 2007 during the period. While there are several distinct floods like in the year 1975, 1988, 1995, 2004, the devastating flood occurred in July 2010 in the region (including the Panjkora and Swat River valley) is one of the highest till now and is well reported and documented with severe consequences.

On average, the mean annual maximum daily flow at the site is 332 m³/s while the mean annual minimum daily flow is 9.2 m³/s. The summary of the relevant statistics is shown as well in table below.

Table 3-6: Average, maximum and minimum daily flows at the project site

Year	MQ [m³/s]	HQ		NQ	
		Q_{max} [m³/s]	day	Q_{min} [m³/s]	day
1961	72.9	268.5	8-Jun	9.4	15-Jan
1962	52.7	345.3	22-Jul	8.6	2-Mar
1963	71.4	328.1	9-May	6.4	1-Mar
1964	69.3	271.9	13-Jul	9.0	3-Jan
1965	89.0	346.4	14-Jun	8.5	15-Jan
1966	80.4	341.1	28-Jun	8.4	31-Jan
1967	77.9	352.2	30-Jun	7.8	31-Jan
1968	81.1	313.5	26-Jun	9.6	17-Feb
1969	86.3	282.0	20-Jun	10.0	24-Jan
1970	65.6	271.4	4-Jun	9.3	20-Feb
1971	62.9	235.3	31-May	7.2	24-Feb

Year	MQ [m³/s]	HQ		NQ	
		Q_{max} [m³/s]	day	Q_{min} [m³/s]	day
1972	79.9	293.2	26-Jun	7.5	11-Jan
1973	86.7	275.3	13-Jun	11.4	30-Jan
1974	56.1	205.6	5-Jun	9.5	14-Jan
1975	83.9	488.8	16-May	7.6	26-Jan
1976	78.5	251.7	25-May	11.7	6-Jan
1977	65.5	290.3	30-Jun	8.9	8-Jan
1978	77.8	323.2	8-Jun	10.4	10-Jan
1979	67.9	273.8	29-Jun	9.2	12-Feb
1980	73.5	258.8	12-Jun	8.9	14-Jan
1981	76.5	302.7	5-May	8.9	30-Jan
1982	51.1	175.9	10-May	8.1	21-Jan
1983	66.5	239.9	19-May	10.4	25-Jan
1984	69.3	302.7	31-May	9.4	7-Mar
1985	50.7	270.8	19-Jul	8.5	17-Jan
1986	69.5	248.2	18-Jul	7.9	21-Jan
1987	73.0	307.8	11-Oct	6.1	30-Jan
1988	75.6	603.2	15-Jul	8.2	17-Jan
1989	65.2	308.3	2-May	9.7	30-Jan
1990	88.3	378.4	16-May	11.5	10-Jan
1991	107.0	440.8	8-Jun	13.7	25-Jan
1992	97.8	461.8	10-Sep	10.5	2-Jan
1993	80.3	297.6	15-Apr	11.1	28-Jan
1994	92.2	336.7	24-Jun	11.1	26-Jan
1995	86.8	722.3	25-Jul	9.8	9-Feb
1996	84.2	384.9	14-Jun	10.2	8-Jan
1997	73.0	364.2	14-Apr	6.9	23-Feb
1998	75.6	338.6	1-May	8.8	31-Dec
1999	72.3	309.9	22-May	6.9	3-Jan
2000	54.2	228.2	11-May	8.1	10-Jan
2001	51.0	384.1	23-Jul	5.9	13-Feb
2002	60.5	308.9	24-Jun	7.4	14-Jan
2003	78.1	297.7	21-Jun	6.7	29-Jan
2004	95.3	583.2	22-May	11.1	25-Jan
2005	67.9	343.9	18-Mar	9.9	1-Feb
2006	48.3	327.3	5-Aug	13.1	1-Jan
2007	72.5	385.1	1-Apr	20.3	29-Nov
2008	53.0	222.7	9-Jun	6.9	1-Jan
2009	72.7	295.9	20-May	7.4	7-Jan
2010	75.2	1109.0	29-Jul	7.2	20-Jan
2011	50.7	201.8	18-Apr	9.3	31-Dec
2012	54.9	180.6	3-Jul	9.6	27-Jan
2013	60.6	220.2	14-Aug	8.9	17-Jan
2014	55.4	192.0	16-Jun	9.6	26-Jan
2015	53.8	239.9	1-Aug	7.9	5-Feb
2016	52.5	258.2	3-Apr	10.5	30-Dec
Statistics:					
Mean	**71.4**	**332.0**	-	**9.2**	-
Max.	**107.0**	**1109.0**	-	**20.3**	-
Min.	**48.3**	**175.9**	-	**5.9**	-
SDV	13.67	146.45	-	2.24	-
CV	0.19	0.44	-	0.24	-
Skew	0.19	3.29	-	2.29	-
Kur	-0.37	14.51	-	9.83	-

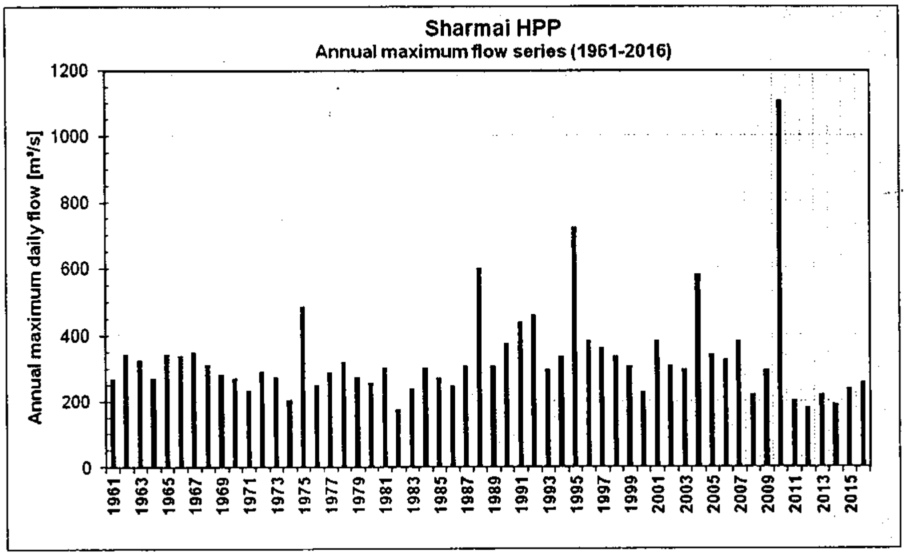


Figure 3-26: Annual maximum daily flow series at the project site

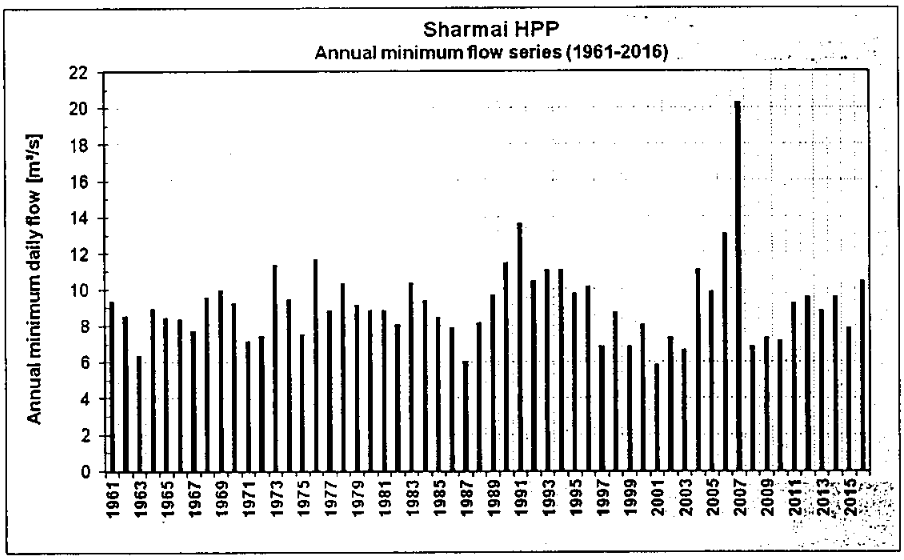


Figure 3-27: Annual minimum daily flow series at the project site

Table 3-7: Relevant statistics of daily flows at the project site

Parameters	Daily Flow Value
Absolute minimum daily flow (NNQ)	5.9 m³/s
Mean annual minimum daily flow (MNQ)	9.2 m³/s
Mean annual flow (MQ)	71.4 m³/s
Mean annual maximum daily flow (MHQ)	332 m³/s
Absolute maximum daily flow (HHQ)	1,109 m³/s

3.6 Flow Duration Curve

The flow availability at a site is generally expressed in terms of "Flow duration curve (FDC)" which shows the percentage of time that flow is likely to equal or exceed any specified value. It thus provides the discharge value at the site that occurs or is exceeded some percent of the time (for e.g., 95% of the time).

The flow duration curve is calculated using the daily flcws available at the project site continuously for the whole period (1961-2016). The resulting curve is shown in figure below. The obtained daily flow duration curve values are also presented in table below. For the purpose of more information about the variability, flow duration curves for driest year (2006, MQ= 48.3 m^3/s) and the wettest year (1991, MQ=107 m^3/s) are also calculated as shown in the figure and table below.

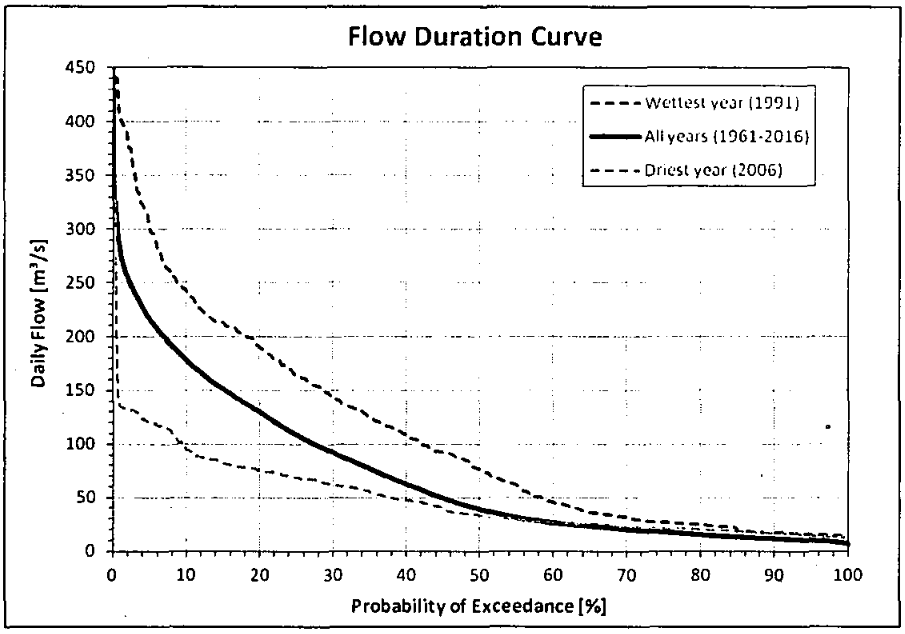


Figure 3-28: Daily flow duration curve at the project site

Table 3-8: Daily flow duration curve values at the project site

Exceedance Probability [%]	Daily Flow Duration Curve values [m³/s]		
	All years (1961-2016)	Wettest year (1991)	Driest year (2006)
5	217.7	301.8	120.1
10	178.8	243.5	95.7
15	152.3	212.9	82.7
20	130.5	190.1	75.5
25	109.4	164.3	69.0
30	92.3	145.6	62.8
35	77.0	125.7	56.4
40	62.5	109.3	47.7

Exceedance Probability [%]	Daily Flow Duration Curve values [m³/s]		
	All years (1961-2016)	Wettest year (1991)	Driest year (2006)
45	49.7	92.6	41.2
50	39.3	76.8	34.1
55	31.2	62.4	30.2
60	26.4	45.5	27.0
65	23.1	36.2	25.4
70	20.1	32.1	23.1
75	17.5	27.1	21.8
80	15.3	24.8	20.2
85	13.3	19.5	19.0
90	11.7	17.2	17.3
95	9.9	15.9	15.2
100	5.9	13.7	13.1
MQ =	71.4	107.0	48.3

Further the flow duration curves for each month and each year are also calculated as shown in figures below. Based on the flow duration curve of daily inflows calculated for each month, the set of seasonal variation curves with different occurrence probabilities are also generated for the project site and shown below.

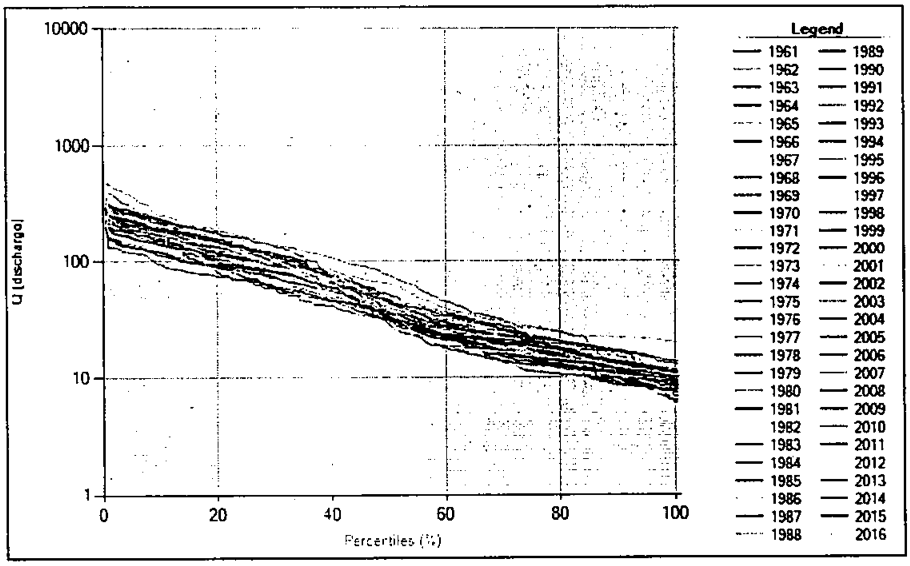


Figure 3-29: Daily flow duration curve for each year at the project site

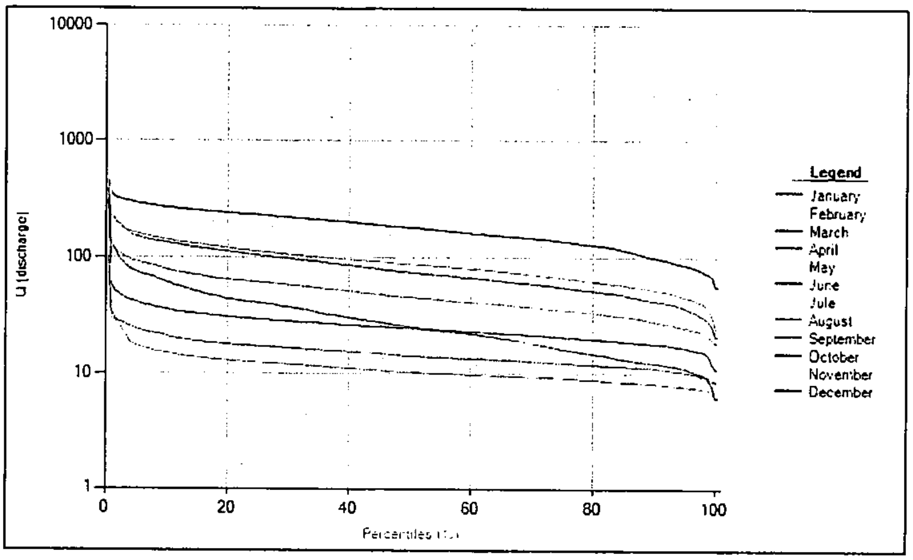


Figure 3-30: Daily flow duration curve for each month at the project site

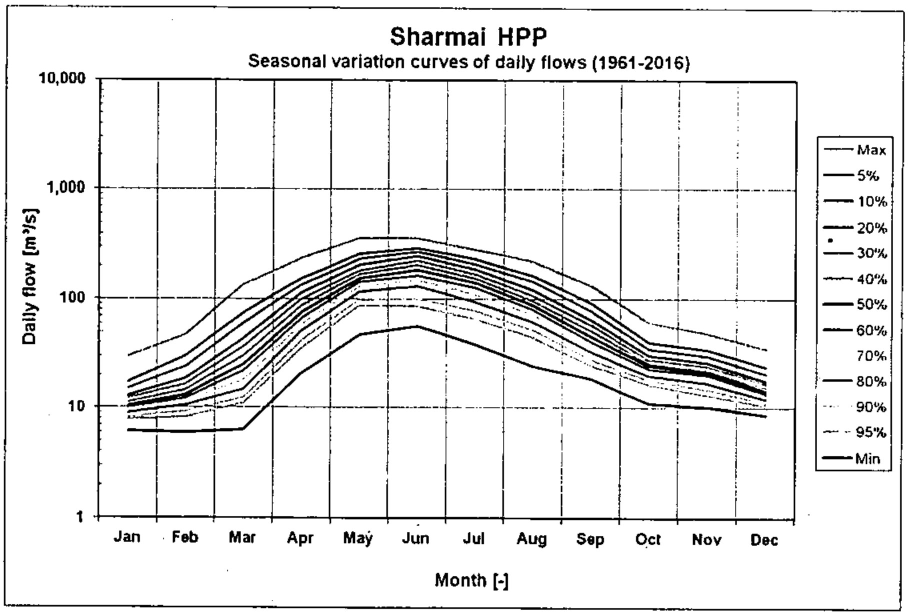


Figure 3-31: Seasonal variation curves of daily inflows with different occurrence probabilities at the project site

3.7 Design Flood

A design flood is a probabilistic or statistical estimate being generally based on some form of probability analysis of flood or rainfall data. An average recurrence interval or exceedance probability is attributed to the estimate. Hence, a design flood indicates the magnitude of the flood peak associated with different recurrence interval or return periods.

In a feasibility study of a hydropower project, estimation of the design flood is required for the design of different hydraulic structures and diversion works of the project. Method of 'flood frequency analysis' is commonly used for estimating the 'design flood' for a HPP project.

3.7.1 Data basis

The primary objective of the flood frequency analysis is to relate the magnitude of extreme events to their frequencies of occurrence through the use of probability distributions. Direct Measurements of extreme events or peak flows are not available at the reference hydrometric station (Sharmai hydrometric station).

However, annual series of maximum daily flow can be extracted from the available/estimated daily flow series at the site, which forms the basis for the design flood estimation. Such annual maximum daily flow series is shown in the figure below. As can be distinctively observed, the recent devastating flood of 29 July 2010 is also included in the considered database. While there are several distinct floods like in the year 1975, 1988, 1995, 2004, the devastating flood occurred in July 2010 in the region (including the Panjkora and Swat River valley) is one of the highest till now and is well reported and documented with severe consequences.

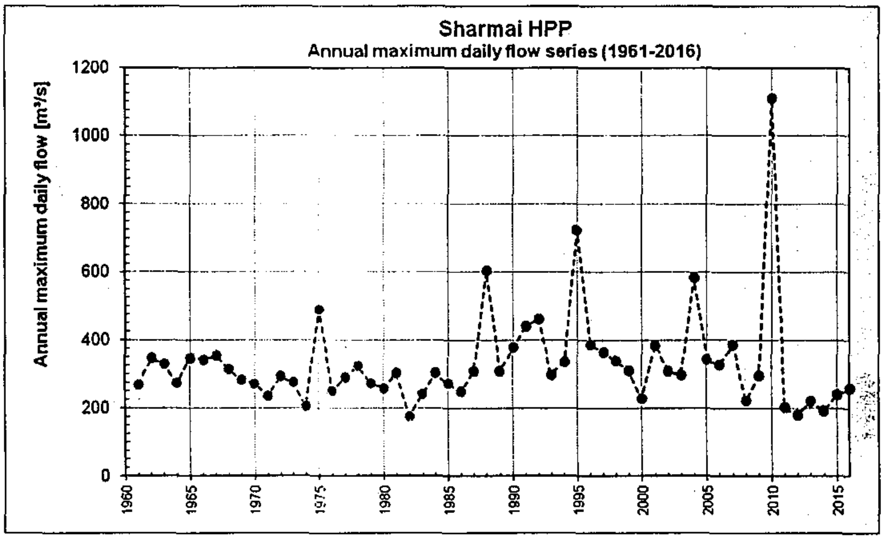


Figure 3-32: Annual series of maximum daily flow at project site

Further, it is obvious that the flow observed in mean daily time scale does not represent the actual flood peak which can be larger than the maximum mean daily value. Therefore, to account for this possible difference between the maximum daily flow value and the flood peak, the commonly used "Fuller's formula" is used to convert the maximum daily flow series into the corresponding flood peak series. The "Fuller's formula" is given as follows:

$$Q_{peak} = Q_{max}\ (1 + 2.66A^{-0.3})$$

where, Q_{peak} = predicted peak flow or flood (m^3/s), Q_{max} = maximum mean daily flow (m^3/s); and A=drainage area (km^2). For the project site, this factor (ratio of peak flood to maximum daily flow) is estimated to be 1.28 which matches well with other nearby gauging stations which has both the maximum daily flow and recorded maximum flood peak.

This means the peak flood record series considered for the flood frequency analysis is 28% more in magnitude than the estimated/observed annual maximum daily flow series, as shown in the table below.

Table 3-9: **Annual series of maximum daily flow and corresponding peak flood series at project site**

Year	Date	Max. Daily Flow [m³/s]	Flood Peak [m³/s]		Year	Date	Max. Daily Flow [m³/s]	Flood Peak [m³/s]
1961	8-Jun	269	343		1989	2-May	308	394
1962	22-Jul	345	441		1990	16-May	378	483
1963	9-May	328	419		1991	8-Jun	441	563
1964	13-Jul	272	347		1992	10-Sep	462	590
1965	14-Jun	346	442		1993	15-Apr	298	380
1966	28-Jun	341	436		1994	24-Jun	337	430
1967	30-Jun	352	450		1995	25-Jul	722	923
1968	26-Jun	314	400		1996	14-Jun	385	492
1969	20-Jun	282	360		1997	14-Apr	364	465
1970	4-Jun	271	347		1998	1-May	339	432
1971	31-May	235	301		1999	22-May	310	396
1972	26-Jun	293	374		2000	11-May	228	291
1973	13-Jun	275	352		2001	23-Jul	384	491
1974	5-Jun	206	263		2002	24-Jun	309	395
1975	16-May	489	624		2003	21-Jun	298	380
1976	25-May	252	321		2004	22-May	583	745
1977	30-Jun	290	371		2005	18-Mar	344	439
1978	8-Jun	323	413		2006	5-Aug	327	418
1979	29-Jun	274	350		2007	1-Apr	385	492
1980	12-Jun	259	331		2008	9-Jun	223	284
1981	5-May	303	387		2009	20-May	296	378
1982	10-May	176	225		2010	29-Jul	1109	1416
1983	19-May	240	306		2011	18-Apr	202	258
1984	31-May	303	387		2012	3-Jul	181	231
1985	19-Jul	271	346		2013	14-Aug	220	281
1986	18-Jul	248	317		2014	16-Jun	192	245
1987	11-Oct	308	393		2015	1-Aug	240	306
1988	15-Jul	603	770		2016	3-Apr	258	330

The occurrence of the peak flood is generally from May to July as shown in the figure below. This can be also observed in the another figure below showing daily flows of different years at the project site.

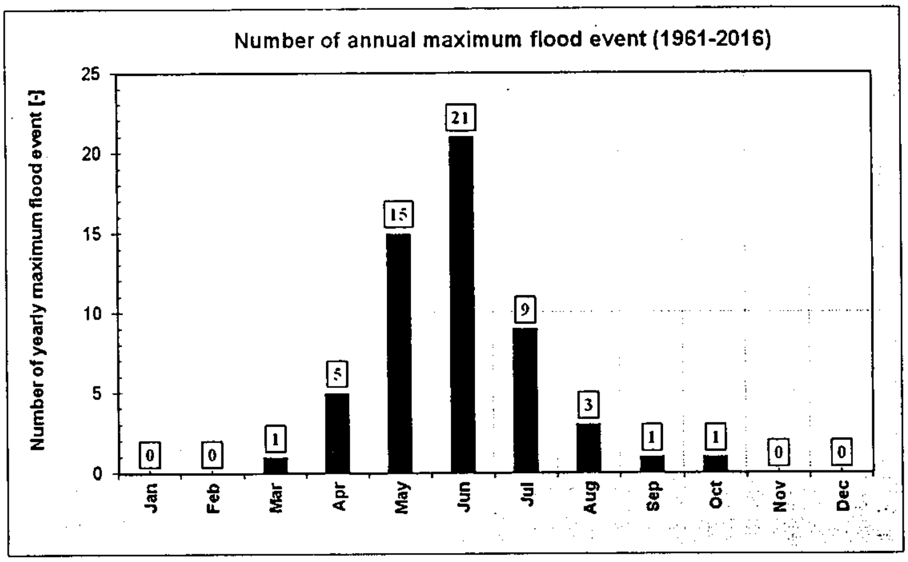


Figure 3-33: Monthly number of occurrence of peak flood during 1961-2016 at project site

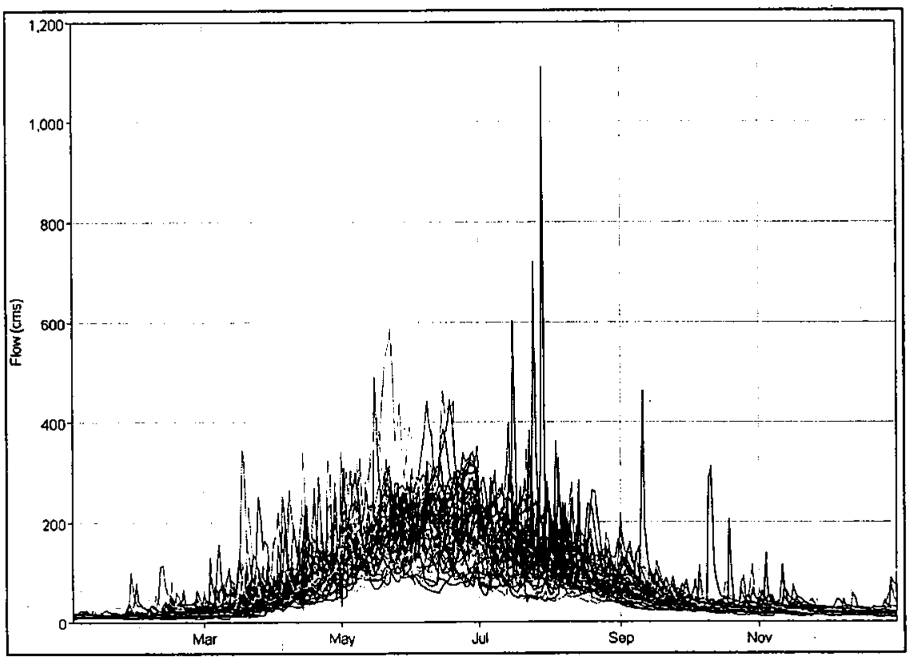


Figure 3-34: Daily flows of different years at the project site

It can be observed that during the data period (1961-2016) the most floods appear in month of June (21 times) followed by May (15 times) and then July (9 times). These months of the year are associated with higher temperature period (snow-melt). The preceding months of Feb-March-April are the highest rainfall period while Jul-Aug also experiences relatively higher rainfall (monsoon-rains).

Hence, the highest flood peaks at the site (which determines its design flood) are basically created by snow-melt phenomenon. However, some years when the maximum flood of the year is due to rainfall rather than snow-melt, then that maximum flood value appears accordingly in the considered database – e.g. the well documented rainfall-generated flood of Sep-1992. Hence, the prepared database of the highest flood peaks at the site, upon which the design-flood estimates are based, consists of both snow-melt generated and rainfall-runoff generated floods (monsoon-rains) as they occurred at the site during the period 1961-2016.

3.7.2 Plausibility check of data basis

Before using the prepared peak flood data series in the flood frequency analysis for the design flood estimation, the plausibility of the data series must be checked. Therefore, several statistical checks have been carried out to assess the plausibility of the peak flood data series at the project site, as summarized below.

Based on the peak flood data series at the site, the average annual peak flow (Ave.) is 424 m³/s. The minimum peak flow (Min.) is 225 m³/s dated 10th May 1982 and the maximum peak flow (Max.) is 1,416 m³/s dated 29th July 2010. The other statistical parameters namely, standard deviation (SDV), coefficient of variation (CV), coefficient of skewness (skew) and coefficient of kurtosis (Kur.) are listed in the table below.

Table 3-10: Statistical parameters of the peak flows estimated at the project site [m³/s]

Ave.	Min.	Max.	SDV	CV	Skew	Kur.
424	225	1,416	238	0.562	3.291	14.513

Ten different statistical tests, based on the "*WMO/UNESCO Expert Workshop on Trend/Change Detection and the CRC for Catchment Hydrology publication Hydrological Recipes*" have been carried out with the peak flood data series estimated at the project site, whose results are given below.

Table 3-11: Results of statistical checks of peak flow series estimated at the project site

Test for	Name of Test	Test statistic	Critical values			Result
			α=0.1	α=0.05	α=0.01	
Trend	Mann-Kendall	-0.191	1.645	1.96	2.576	Not Significant
	Spearman's Rho	-0.328	1.645	1.96	2.576	Not Significant
	Linear regression	0.915	1.676	2.006	2.672	Not Significant
Step change in mean/median	Cumulative deviation	1.011	1.144	1.272	1.524	Not Significant
	Worsley likelihood	2.127	2.87	3.16	3.79	Not Significant
difference in mean/median in two different data periods	Rank Sum	-1.401	1.645	1.96	2.576	Not Significant
	Student's t	-1.514	1.676	2.005	2.67	Not Significant

Test for	Name of Test	Test statistic	Critical values			Result
			α=0.1	α=0.05	α=0.01	
Randomness	Median Crossing	1.483	1.645	1.96	2.576	Not Significant
	Turning Point	-1.289	1.645	1.96	2.576	Not Significant
	Auto Correlation	0.048	1.645	1.96	2.576	Not Significant

The results of these statistical checks show that: there exist no statistically significant trend, no statistically significant step jump, no statistically significant different mean/median between splitted data periods and the data series come from a random process.

The results of the statistical checks presented above prove that the peak flood data series estimated at the site are random, independent, homogenous and steady. Hence, it is justified to use the complete peak flood data series, estimated at the site, for the flood frequency analysis to estimate the design flood for the project.

3.7.3 Design flood estimation

For the flood frequency analysis with those flood peak series seven probability distributions, three parameter estimation methods, and three statistical tests are used (using HQ-EX software tool developed by WASY-Germany) as listed below:

a) Probability distributions:
- Gumbel or extreme value distribution of Type I (E1)
- Generalized extreme value distribution (AE)
- Pearson Type III distribution (P3)
- Logarithmic Pearson Type III distribution (LP3)
- Wakeby distribution (WB3)
- Three-parameter logarithmic normal distribution (LN3)
- Two-component extreme value or Rossi distribution (TCEV or ME)

b) Parameter estimation methods:
- Moment method (MM)
- Maximum likelihood method (MLM)
- Probability weighted method (PWM of WGM)

c) Goodness-of-fit test:
- Kolgomorow-Smirnow KS-test
- Nw^2-Test
- the probability plot correlation coefficient test PPCC-Test.

The different distributions estimated with different parameter estimation methods fitted to the estimated peak flood series at the project site (1961-2016) is shown in the figure below and to read out the resulting design flood corresponding to each of these different distribution functions, the results are presented in the table as well.

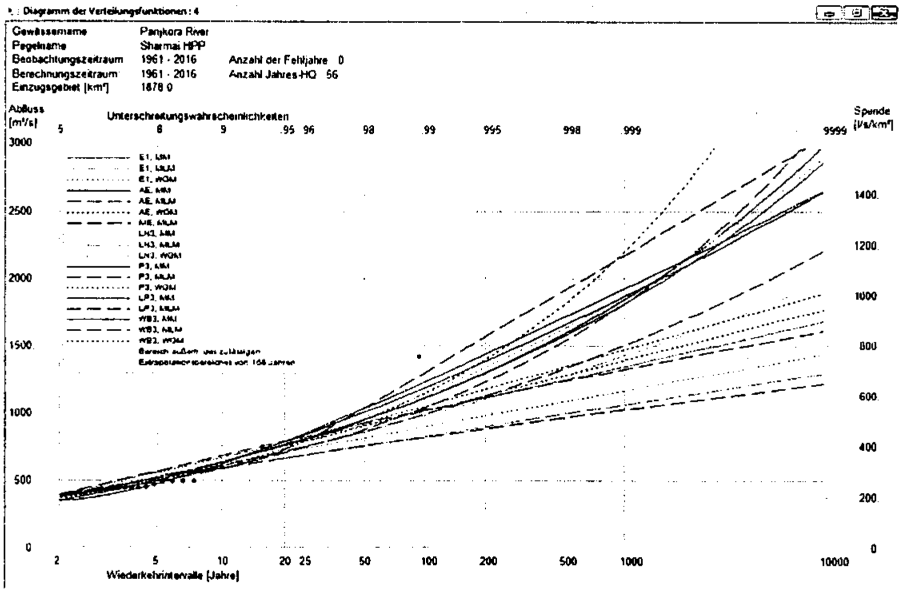


Figure 3-35: Fitting of different probability distributions for the flood frequency analysis at the project site (output of HQ-EX software)

Table 3-12: Design flood corresponding to different probability distribution functions

Probability Distribution	Parameter Estimation Method	Return Period [yrs]										
		2	5	10	20	25	50	100	200	500	1,000	10,000
E1	MM	393	559	668	773	806	909	1,011	1,112	1,246	1,347	1,683
E1	MLM	395	510	587	660	684	755	826	897	991	1,061	1,296
E1	WGM	399	532	621	705	732	815	897	979	1,087	1,168	1,439
AE	MM	379	520	632	756	799	945	1,110	1,299	1,590	1,847	2,997
AE	MLM	375	492	589	700	740	877	1,038	1,229	1,535	1,815	3,174
AE	WGM	375	498	605	733	780	945	1,147	1,395	1,810	2,208	4,304
ME	MLM	380	488	583	726	791	1,036	1,300	1,565	1,914	2,178	3,055
LN3	MM	371	512	632	765	812	967	1,142	1,337	1,629	1,878	2,904
LN3	MLM	383	514	612	715	749	860	978	1,103	1,282	1,427	1,981
LN3	WGM	372	505	619	748	794	946	1,119	1,313	1,607	1,859	2,913
P3	MM	347	474	619	788	845	1,033	1,230	1,434	1,712	1,926	2,658
P3	MLM	394	515	593	665	687	754	819	882	963	1,024	1,218
P3	WGM	368	519	639	761	801	925	1,051	1,177	1,344	1,472	1,897
LP3	MM	377	520	634	759	803	947	1,110	1,293	1,572	1,815	2,873
LP3	MLM	383	510	607	711	746	861	987	1,125	1,330	1,503	2,219
WB3	MM	356	498	632	783	834	1,004	1,187	1,380	1,652	1,869	2,650
WB3	MLM	399	567	681	788	821	922	1,020	1,114	1,236	1,325	1,610
WB3	WGM	374	523	636	750	786	900	1,014	1,128	1,278	1,393	1,772

The three statistical tests namely, KS-test, Nw^2-Test and PPCC-Test were carried out with all the fitted probability distribution functions for the flood frequency analysis. The results of these tests as the output from the HQ-EX software are presented in table below.

Table 3-13: Results of statistical checks of the fitting of different probability distribution functions in the flood frequency analysis

Probability Distribution	Parameter Estimation Method	Para 1	Para 2	Para 3	Para 4	KS-test	Nw^2-Test	PPCC-Test	Result
E1	MM	339.84	145.81			0.172	0.464	0.828	0.808
E1	MLM	357.59	101.91			0.122	0.135	0.828	0.429
E1	WGM	356.15	117.55			0.150	0.251	0.828	0.573
AE	MM	339.76	103.73	-0.19		0.115	0.189	0.924	0.380
AE	MLM	343.16	82.02	-0.24		0.099	0.099	0.944	0.255
AE	WGM	343.18	81.93	-0.30		0.099	0.105	0.961	0.243
ME	**MLM**	338.72	75.29	-450.46	380.59	**0.075**	**0.054**	**0.990**	**0.140**
LN3	MM	214.08	5.05	0.76		0.131	0.203	0.938	0.397
LN3	MLM	176.17	5.33	0.58		0.108	0.122	0.895	0.335
LN3	WGM	232.67	4.94	0.79		0.116	0.151	0.943	0.324
P3	MM	325.28	354.28	0.28		0.232	0.831	0.944	1.119
P3	MLM	172.99	70.09	3.49		0.127	0.153	0.813	0.467
P3	WGM	275.00	189.26	0.79		0.145	0.257	0.906	0.496
LP3	MM	5.16	0.15	5.47		0.119	0.199	0.925	0.393
LP3	MLM	5.17	0.12	6.71		0.103	0.108	0.906	0.305
WB3	MM	293.08	105.44	0.71		0.176	0.404	0.948	0.632
WB3	MLM	274.24	234.20	1.25		0.182	0.364	0.851	0.695
WB3	WGM	261.69	161.92	0.99		0.128	0.219	0.897	0.451

The results of the statistical checks of the fitting of different probability distribution functions in the flood frequency analysis indicates that the best distribution for the project site is the "Two-component extreme value or Rossi distribution (TCEV or ME)" with its parameters estimated through "Maximum likelihood method (MLM)".

The resulting best-fitting frequency curve is re-shown in the figure below. The resulting design flood magnitudes for the project damsite are listed in table below.

For the powerhouse site, the design flood estimated at the damsite is increased according to the increase in catchment area from the damsite to the powerhouse site (i.e. increased by the catchment area ratio of 44%) hence assuming that the specific flood from the catchment between damsite and the powerhouse site is same as that from the upstream of the damsite – a slightly conservative assumption here. The resulting design flood estimates for the powerhouse site are also presented in the table below.

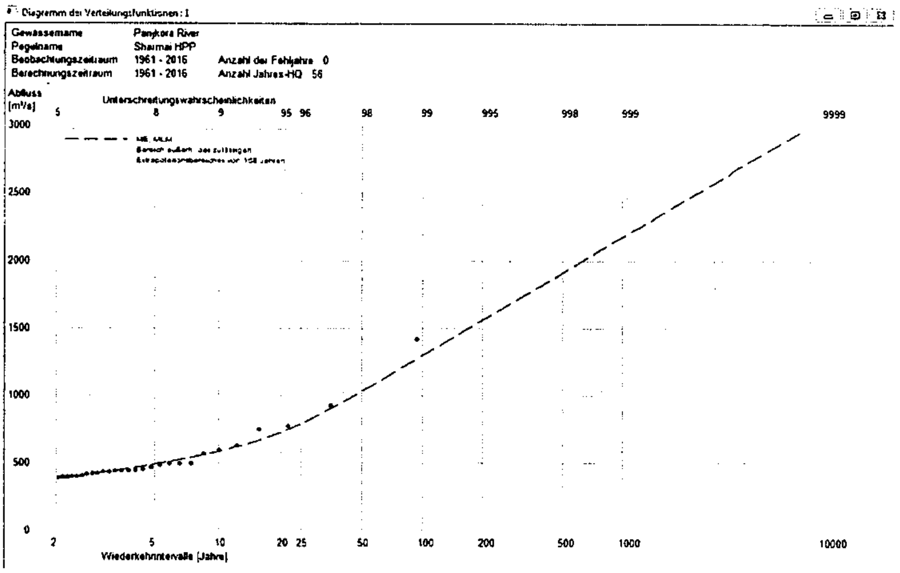


Figure 3-36: Best-fitting probability plot of design flood at the project site

Table 3-14: Estimated design flood with different return period at the project damsite and the powerhouse site

Return Period [yrs]	Probability of non-exceedance [-]	Design Flood at Damsite [m³/s]	Design Flood at Powerhouse site [m³/s]
2	0.5	380	550
5	0.8	490	705
10	0.9	580	835
20	0.95	730	1,050
25	0.96	790	1,140
50	0.98	1,035	1,490
100	0.99	1,300	1,875
200	0.995	1,565	2,255
500	0.998	1,910	2,750
1,000	0.999	2,180	2,140
10,000	0.9999	3,055	4,400

The design floods calculated in the above analysis matches well and are at slightly safer side when compared with the design flood estimated and transposed from the flood peak series recorded at Chakdara station and also when checked similarly with Kalam station.

Further, the design flood estimated at the project site is compared and checked with that from the other stations and the hydropower projects in the neighborhood. The comparison is shown in the following two tables indicating the respective design flood and the corresponding specific flood respectively as published/reported in the available studies.

Table 3-15: Design flood at different locations for comparison

Return Period [yrs]	Design Flood [m³/s]					
	Sharmai Damsite *(Area = 1878 km²)*	Madian HPP *(Area = 2403 km²)*	Gabral Kalam HPP *(Area = 962 km²)*	Chitral station *(Area = 12495 km²)*	Mastuj Bridge station *(Area = 520 km²)*	Arkari Gol HPP *(Area = 1036 km²)*
2	**380**	445	161	1,146	53	211
5	**490**	530	210	1,384	94	253
10	**580**	587	243	1,541	122	284
20	**730**	656	274	1,692	148	-
25	**790**	-	284	-	-	-
50	**1,035**	712	314	1,888	182	365
100	**1,300**	860	384	2,035	208	404
200	**1,565**	-	-	-	-	-
500	**1,910**	-	-	-	-	509
1,000	**2,180**	1,450	854	2,519	292	562
10,000	**3,055**	2,002	1,375	3,003	376	777

Table 3-16: Specific design flood at different locations for comparison

Return Period [yrs]	Specific Design Flood [lit/s/km²]					
	Sharmai Damsite *(Area = 1878 km²)*	Madian HPP *(Area = 2403 km²)*	Gabral Kalam HPP *(Area = 962 km²)*	Chitral station *(Area = 12495 km²)*	Mastuj Bridge station *(Area = 520 km²)*	Arkari Gol HPP *(Area = 1036 km²)*
2	**202**	185	167	92	102	204
5	**261**	221	218	111	182	244
10	**309**	244	253	123	234	274
20	**389**	273	285	135	285	-
25	**421**	-	295	-	-	-
50	**551**	296	326	151	350	352
100	**692**	358	399	163	399	390
200	**833**	-	-	-	-	-
500	**1,017**	-	-	-	-	491
1,000	**1,161**	603	888	202	561	542
10,000	**1,626**	833	1,429	240	722	750

It can be noted that the design flood estimated here for the project site fits well and are at the safer side when compared with the design flood estimated for the other stations and the hydropower projects as published/reported in the available studies and shown in the table above.

The comparatively higher values estimated for the project site here can be accounted for the extremely high devastating flood of July 2010 which is considered in the analysis here unlike the presented previous studies (except for the Mastuj Bridge station where the flood of July 2010 is also considered).

As a further check of the design flood estimation, the results from a regional analysis approach developed by GTZ is compared. GTZ had collected instantaneous maximum discharge data of various stream gauging stations of Northern areas of Pakistan including snowmelt runoff and glacier melt runoff watersheds (which has hydro-metereological similarity with the project area) and had developed correlations for the peak floods as a function of watershed areas for various return periods. According to those GTZ regression equations, the 1,000 and 10,000 years flood at the Sharami damsite will be 2,200 m³/s and 3,150 m³/s respectively- which matches quite well with the design flood estimations presented above.

All these different cross-checks conclude that the design flood estimated here for the project site can be considered as plausible, reliable and adequate.

Further, for the check flood, the "Probable Maximum Flood" (PMF) at the project site cannot be estimated directly/deterministically with adequate reliability given the required and available data conditions. However, the PMF-equivalent check flood for the Sharmai HPP is proposed considering the 65% increase in the estimated 10,000 years flood which results to 5,040 m³/s at the project damsite. This estimation is in line with or rather in the safer side when compared to the some international guidelines for the design flood for dams which suggest to consider the PMF or safety check flood to be 1.5 times the design flood and the design flood as 1,000 years flood (for e.g. Norwegiam guidelines).

Among the available previous studies, the estimation of PMF is shown only for the Naran HPP (through Storm maximization PMP-PMF approach) which shows the PMF to be 2,300 m³/s that corresponds to the specific flood of 2,542 lit/s/km² (catchment area = 905 km²). When compared to this, our consideration of PMF-equivalent check flood of 5,040 m³/s corresponds to the specific flood of 2,684 lit/s/km² matches well and is at slightly safer side. Further, the estimated magnitude of the PMF-equivalent check flood corresponds to "Francou Rodier" coefficient "*K*" of more than five which means the estimated value matches well with the regional maximum possible flood in the region as proposed by the well known Francou-Roudier method.

In this regard, the estimated design flood along with the considered safety check flood (PMF equivalent) can be considered as plausible and adequate.

3.8 Impact of Climate Change

Changes in precipitation and temperature lead to changes in runoff and water availability. Climate change impacts on the available river flows and the incoming floods, which are relevant in context to the HPP projects, will occur through the changes in precipitation, temperature/evapotranspiration of the area caused by the anthropogenic climate change.

The impact of climate change on a specific hydropower plant will depend on the local change of these hydrological characteristics, as well as on the type of hydropower plant and on the (seasonal) variation in energy demand, which will itself be affected by climate change. Run-of-river projects are more susceptible to increased flow variability than projects with large storage capacity. Projections of future hydropower generation are subject to the uncertainty of projected temperature, precipitation and streamflow.

Making a site-specific independent analysis and direct estimation of the impact of climate change on the viability of the specific HPP projects, is very much data-intensive (both data quality & quantity) and also computation-intensive. The constraint and the limitation in the quality and quantity of available data relevant for the appraisal of the possible climate change impacts to the project do not allow making such analysis in local scale. Therefore, such site-specific independent analysis of climate change is generally not within the scope of the HPP projects. Accordingly, none of the available previous studies has considered or reported on the impact of climate change for the hydropower projects in the region.

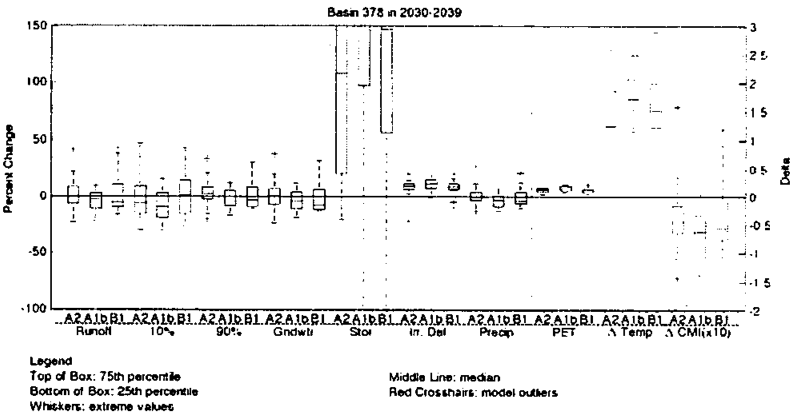
However, certain level of the assessment of the impact of climate change in the project area can be done through several published/available relevant research projects and studies, which is done here as well.

As one of such possibility, the impacts of the climate change in HPP projects in a region can be fairly assessed through the published and accessible results of the several General Circulation Models (GCMs) that have been checked and accepted by the relevant communities (IPCC). One of such reliable source is provided by the "World Bank Group" as a "*Climate Change Knowledge Portal*". The World Bank has recently developed a concept for the assessment of the impact of climate change on six hydrological indicators for more than 8,000 river basins across the world. Strzepek et al (2011) provide a thorough description of the developed basin scale indicator approach. Based on the future precipitation and temperature changes, one of the assessed future hydrological indicators is mean annual runoff along with flood and drought indicator which are relevant for the HPP projects. The portal can be accessed through: http://sdwebx.worldbank.org/climateportal/

This portal provides directly the expected future change in runoff for the following options:

- Emission Scenarios (Future Climate Scenario) - i) A1b[1], ii) A2[2] and iii) B1[3]
- Ground Circulation Models (GCM) - The portal utilizes results of 22 GCMs in order to perform an analysis that leads to the percentage changes of runoff.
- Time Period - i) 2030-2039 and ii) 2050-2059.

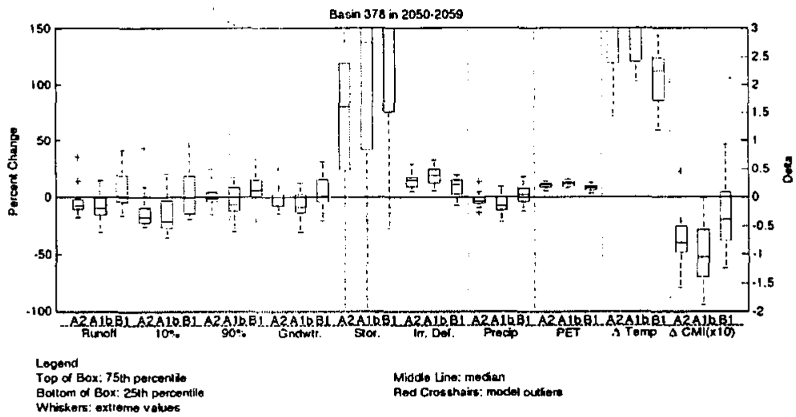
The retrieved results for the project site in Northern Pakistan are summarized in the following figures.



[1] The A1 storyline and scenario family describes a future world of very rapid economic growth, global population that peaks in mid-century and declines thereafter, and the rapid introduction of new and more efficient technologies. Major underlying themes are convergence among regions, capacity building and increased cultural and social interactions, with a substantial reduction in regional differences in per capita income. The A1b scenario is distinguished by its technological emphasis assuming a balance across all energy sources (where balanced is defined as not relying too heavily on one particular energy source, on the assumption that similar improvement rates apply to all energy supply and end-use technologies).

[2] The A2 storyline and scenario family describes a very heterogeneous world. The underlying theme is self-reliance and preservation of local identities. Fertility patterns across regions converge very slowly, which results in continuously increasing population. Economic development is primarily regionally oriented and per capita economic growth and technological change more fragmented and slower than other storylines.

[3] The B1 storyline and scenario family describes a convergent world with the same global population, that peaks in mid-century and declines thereafter, as in the A1 storyline, but with rapid change in economic structures toward a service and information economy, with reductions in material intensity and the introduction of clean and resource-efficient technologies. The emphasis is on global solutions to economic, social and environmental sustainability, including improved equity, but without additional climate initiatives.



where,

A2, A1b, B1	:	Future Climate Scenarios
Runoff	:	Mean Annual Water Runoff
10%	:	Flood Indicator (Annual High Flow)
90%	:	Drought Indicator (Annual Low Flow)
Gndwtr	:	Groundwater (Baseflow)
Stor	:	Water available to basin (Basin Yield)
Irr. Def	:	Irrigation Deficit (Crop water deficit for crops)
Precip	:	Mean Annual Precipitation
PET	:	Annual Potential Evapotranspiration
Delta Temp	:	Average Change of Mean Temperature
Delta CMIX (x10)	:	Climate Moisture Index for all

Figure 3-37: Box plot of climate change impact in the project area for future time period of 2030-2039 and 2050-2059

The results of the 56 GCM models as retrieved by the World Bank Climate Change Knowledge Portal with regards to the % change of mean annual runoff, for example, are shown in the figure below.

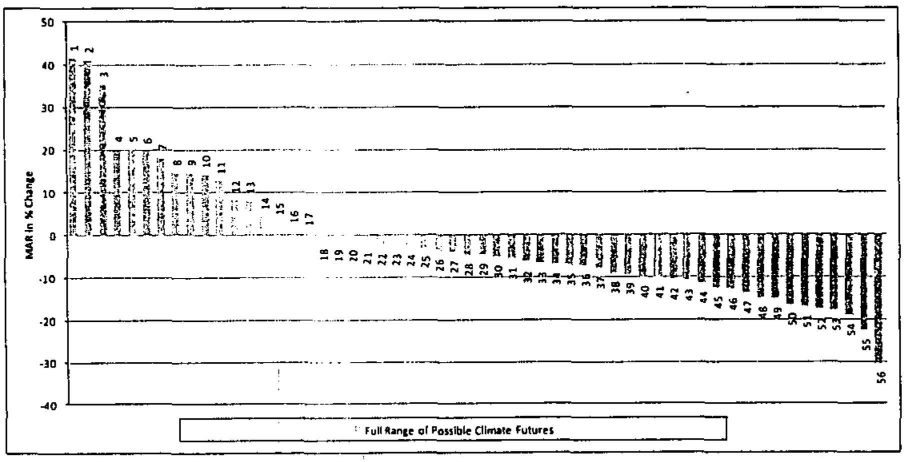


Figure 3-38: GCM results for three different CO2 emission scenarios regarding % change of MAR in water basin of the project (2005-2059)

As indicated, the models results for the future time period varied within a range from 42% increase of runoff to 30% decrease. Similarly the absolute increase of mean annual temperature is expected to range between 1.19 and 4.13 °C. For different possible scenarios, following different results are obtained.

Scenario A1b
For this scenario, the in-house hydrological analysis of the World Bank Climate Change Knowledge Portal indicates that the median value of the percentage runoff change predictions of the individual GCMs is 5% reduction of the mean annual runoff. The median value of the predictions of absolute temperature change is 2.8°C

Scenario A2
For this scenario, the median value of the percentage runoff change predictions of the individual GCMs is 0%. The median value of the predictions of absolute temperature change is 2.6°C

Scenario B1
For this scenario, the median value of the percentage runoff change predictions of the individual GCMs is 0.5% increase of the mean annual runoff. The median value of the predictions of absolute temperature change is 2.1°C

There is however no consensus between the climate change experts/models and therefore it is not possible to infer in a deterministic manner what exact scenario is more likely to occur or which GCM is more reliable. However, these results are overall in agreement with the forecast on future climate change impacts in the project area as included in the most recent and relevant report from a research project launched by the Ministry of Climate Change (MoCC) and the United Nations Development Programme (UNDP) in July 2015 - "*The Vulnerability of Pakistan's Water Sector to the Impacts of Climate Change: Identification of gaps and recommendations for action*".

The project's goal was to analyze how climate change could adversely affect the availability of water resources in the Indus basin (to which the Sharami project site also belongs), and therefore limit the country's future economic and social development. The relevant observations from this report are as follows.

- There remains uncertainty regarding how changing climatic conditions are or could adversely affect the country's critical water resources. Large knowledge gaps remain regarding the impact of climate change on the area's hydrological regime. The systematic review also made evident the continuing lack of knowledge regarding the physical processes that shape the hydrological regime of the area. There is no clear understanding of the relationship between the glacial and nival (snow-melt) regimes, how they respond to current climate conditions and how they might change in the future.
- Climate projections suggest that temperatures in the area will continue to rise in the coming decades—in both the summer and winter—and that there remains significant uncertainty regarding changes in precipitation, particularly with respect to winter precipitation, as available projections are inconsistent with respect to changes on a seasonal and spatial basis. How each of the glacial, nival and rainfall regimes respond to these

climatic changes will determine the future hydrology of the area. As runoff in the glacial regime is positively correlated to summer temperatures, higher summer temperatures are expected to lead to higher rates of runoff. However, it is difficult to predict the extent or timing of this increase over the long-term given the uncertainties associated with climate projections, the topographical complexity of the area, and the absence of comprehensive inventory of glaciers within in the basin.

- Nival regime changes are of particular interest when trying to assess the potential consequences of climate change for future hydrological patterns in the area as it comprises the greater portion of consistent downstream water flow. A higher rate of winter precipitation, which would be consistent with historical trends, could lead to higher levels of summer runoff. However, as winter precipitation projections from different models contradict one another and are associated with great uncertainty, it is challenging to anticipate how runoff rates from this regime might change in the future.
- Overall, significant uncertainty remains regarding not only how each of the glacial, nival and rainfall regimes will be affected by climate change but also with respect to the interplay between them and therefore the collective outcome of these changes.
- The reviewed studies all concluded that the volume of water flow in the area will not change significantly prior to 2050 as decreases in glacial melt runoff will be compensated by runoff generated from increasing monsoon rainfall. While no significant change in total flow volume is projected, modeling results show that the main impact of climate change prior to 2050 will be a shift in the timing of peak flow to slightly earlier in the year. It is also projected that there will be a shift in the timing of peak flows in the area, with modeling results suggesting that peak flow will occur three to four weeks earlier compared to a baseline of recorded flows between 2001 and 2005 in which peak runoff occurred in weeks 26 and 27 of the calendar year (June/July).
- In the long-term, available models suggest that water flow in the area will decline with the loss of glaciers. For example, a model found that a 50 percent decrease in glacial cover due to the temperature increases would result in a decrease in glacier runoff of only 22 percent. However, the total runoff in the area would be offset by increases (by 53 percent) in rainfall concentrated in the period between June to August. In contrast, the discharge drops significantly for the scenario in which there is complete loss of glaciers in the area. Another model also found increases in discharge for models that assume 50 to 100 percent glacier coverage, but substantial reductions when there is a complete loss of glacier coverage.
- Climate change impacts on water flows in the area will be limited in the near term. The systematic review of existing but limited researches on how climate change may affect the hydrological regime of the area shows almost consistent agreement that the climate change will only have a limited impact on total annual volumes of water flow stemming from the glacial and nival regimes in the near and medium terms. The impact of climate change in the period before 2050 is more likely to be reflected in changes in the timing of peak flows and increased variability in flow

levels, primarily due to greater unpredictability in the rainfall (monsoon) regime.
- The run-of-the-river hydroelectric power installations may be affected by greater variability in water flows, while large-scale hydropower plants are unlikely to be significantly affected in the near to medium terms.

Hence, as shown by the different information, studies and analysis presented above, increase in temperature is a common finding but the uncertainty and regional scale of the analysis doesn't allow to infer or quantify changes in water availability at the Sharmai project site although it is demonstrated not to be significantly high.

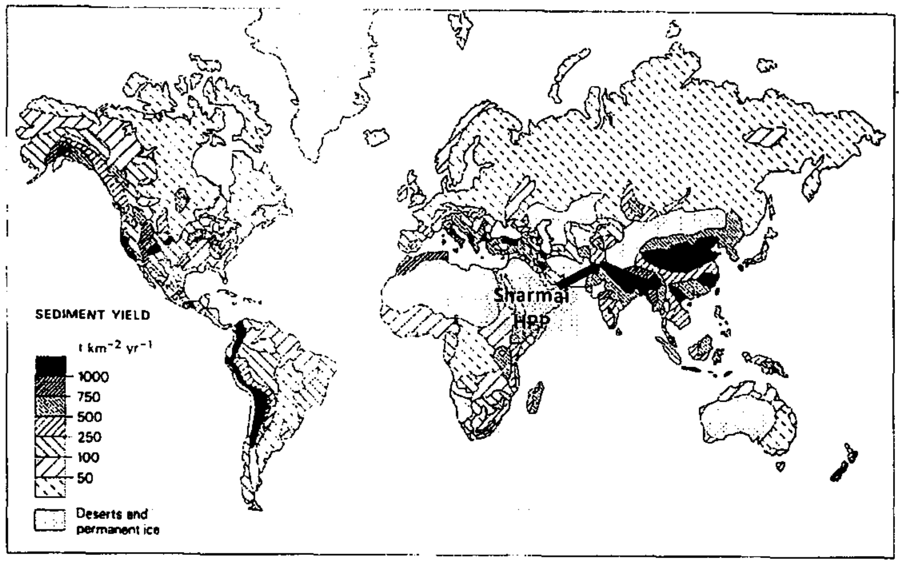
Further, the sensitivity analyses applied in design and sizing of the plant (different water availability scenarios) and the variation in the appraisal of the financial performance (i.e. different energy generation scenarios) of Sharmai project assess the robustness of the technical and financial performance of the project against the impacts attributable to climate changes.

Therefore, it can be concluded that the climate change impact on the viability of Sharmai project can be considered as a certain hydrological risk, which shall be taken into account in the project risk analysis and in conclusion on the financial viability of the project.

3.9 Sedimentation Study

3.9.1 Introduction

Sharmai HPP is located in northern Pakistan, in the upstream reach of Panjkora River. The latter originates from a mountainous region (>4000 m) and confluences into Swat River. The project is located on the western border of one of the most prone to surface erosion regions in the world, with specific sediment loads exceeding the value of 1000 tons/km²/a, as indicated in the global map of specific sediment yield which was published by Walling & Webb (1996).



Source: Walling & Webb (1996)

Figure 3-39: Global map of specific sediment yield including the project location

The project area is strongly affected by the glacial processes and the monsoon seasonal precipitation pattern. It is characterized by the high energy potential stemming from the high terrain elevation and steep slopes, the rocky terrain conditions with high content in quartz minerals as well as the presence of snow and ice. The interrelation of the aforementioned features has as result the realization of the highest sediment concentrations worldwide, characterized by relatively high hardness.

The objective of the sedimentation study is to provide an assessment of:

- the sediment inflow into the Sharmai headpond, its intra-annual variability and its grain size distribution
- the lifetime of the headpond if no sediment management is applied
- the risks associated with sedimentation of the headpond
- the necessary mitigation measures against sedimentation.

3.9.2 Morphology of Panjkora River

Panjkora River is a typical steep mountainous gravel bed river. The river morphology and the alluvial deposits on the river banks in the proposed reservoir area give an indication of the capability of Panjkora River to transport sediments.

The topographical survey indicated that the mean annual river bed gradient at the project site is approximately 1%. The river bed width varies between 25m and 50 m. The side slope of the river banks ranges between 1:1.5 and 1:2.5 (V:H). As expected, the river morphology is characterized by the following two distinct morphological features.

- A surface armor layer which is coarser than the underlying finer subsurface material. Hence, morphological changes occur only after the armor layer is broken during high flow events, since the sub-surface material is protected against erosion.
- Riffle-pool sequences which have as result increased energy losses and bed stability.

The aforementioned morphological features are shown in the following picture which was taken during the site visit.

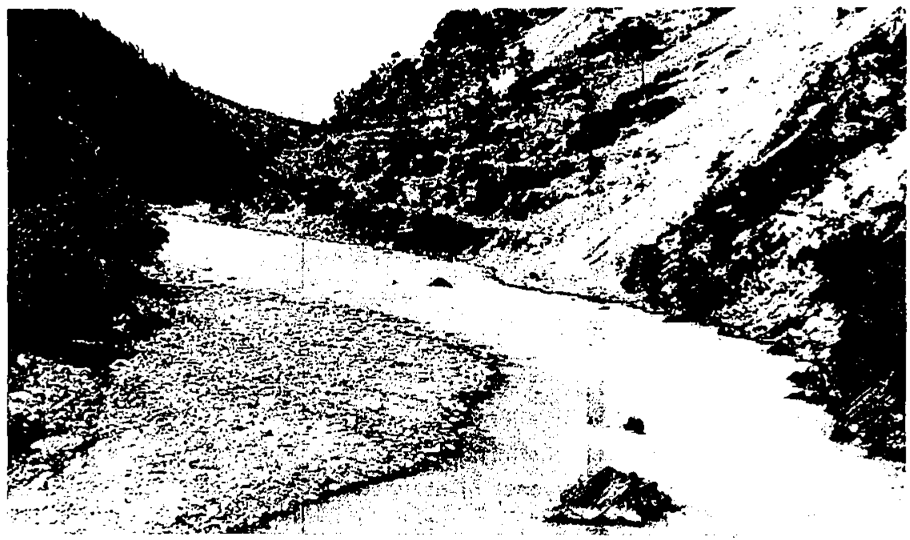


Figure 3-40: River morphology of Punkjkora River at project site. Armor layer on gravel bank (left side) and riffle-pool morphology in the main channel

Panjkora River is the natural drainage of glaciated areas in North-Pakistan, which are the source of sediments of different gradation. Approximately 10% of the catchment area of Sharmai HPP extends at elevations higher than 4200 masl. The morainic deposits in the project area are an indication of presence of glaciers in ancient times that have retreated with time.

3.9.3 Bed material

The Panjkora River is characterized by a distinct armor layer which protects the underlying subsurface material from erosion. The grain size distribution of the bed material was determined by means of the line by number method. The application of the latter was based on photos of the surface armor layer taken by the Consultant during the site visit. The line-by-number analysis was applied in three samples, each one of which comprised approximately 350 particles along the transect line, following the respective guidelines given in the technical literature.

The grain size distribution of the subsurface bed material was derived on basis of the line-by-number grain size distribution of surface material. The analysis consisted of the steps given below, following Fehr (1987).

- Identification of the gravel and cobble particles on the plan view photos and measurement of the b-axis diameter of particles which were beneath a transect line as presented in the figure below. Particles with a b-axis of less than 5 mm are neglected in the field because it is difficult to include them all and measure them correctly.
- Numerical sieving for classification of the measured particle diameters into grain classes and determination of the respective frequency of occurrence distribution.
- Conversion of the derived line-by-number grain size distribution to the equivalent volume-by-weight grain size distribution. The conversion is done as follows.

$$\Delta p_i = \frac{\Delta q_i \, d_{mi}^{0.8}}{\sum_1^n \Delta q_i \, d_{mi}^{0.8}}$$

Where:

Δpi	Volume by weight fractional content of the bed material (Weight of the grain class / weight of the hole sample)
Δqi	Grain size occurrence frequency Count of the particles of grain class i / total count of sampled particles (Line by number analysis of the armor layer)
dmi	mean diameter of the grain class
n	total count of the grain classes
0.80	Exponent for the conversion of a line by number grain size distribution of surface bed material into a volume by weight grain size distribution of the subsurface bed material.

The calculated volume-by-weight grain size distribution is further corrected with the following relationship

$$p_{ic} = 0.25 + 0.75 \sum_1^i \Delta p_i$$

- Fitting of a Fuller grain size distribution for the representation of the sand and silt grain classes existing in the bed material.

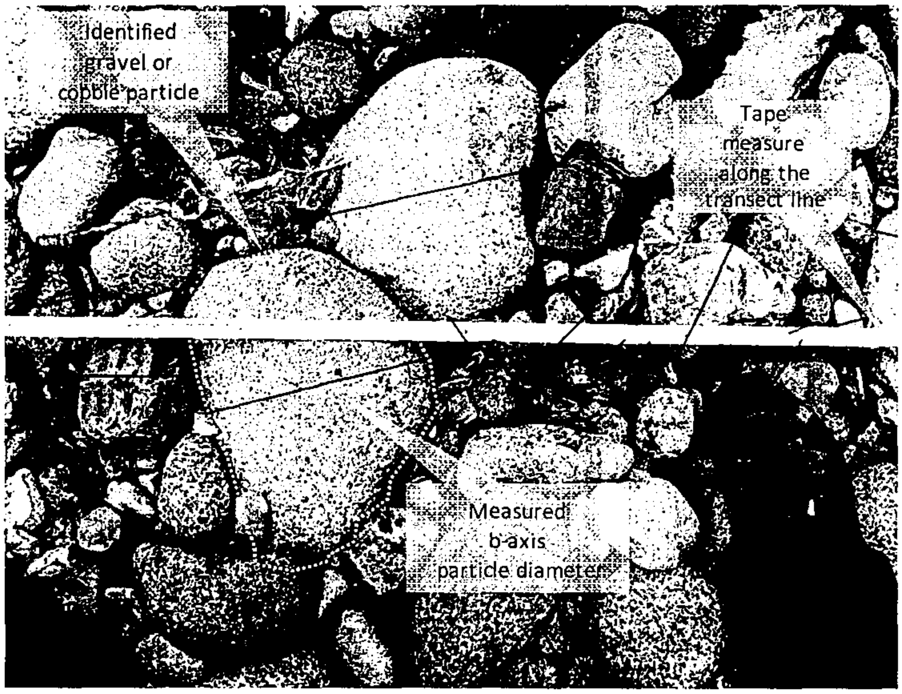


Figure 3-41: Line-by-Number analysis for determination of armor layer composition (Sample extract)

The derived bed material grain size distribution for the three samples together with the mean gradation curve is shown in the figure below. The same figure contains the characteristic grain sizes that are used for determination of the skin and total friction losses coefficients as well as for the assessment of the bedload transport rates.

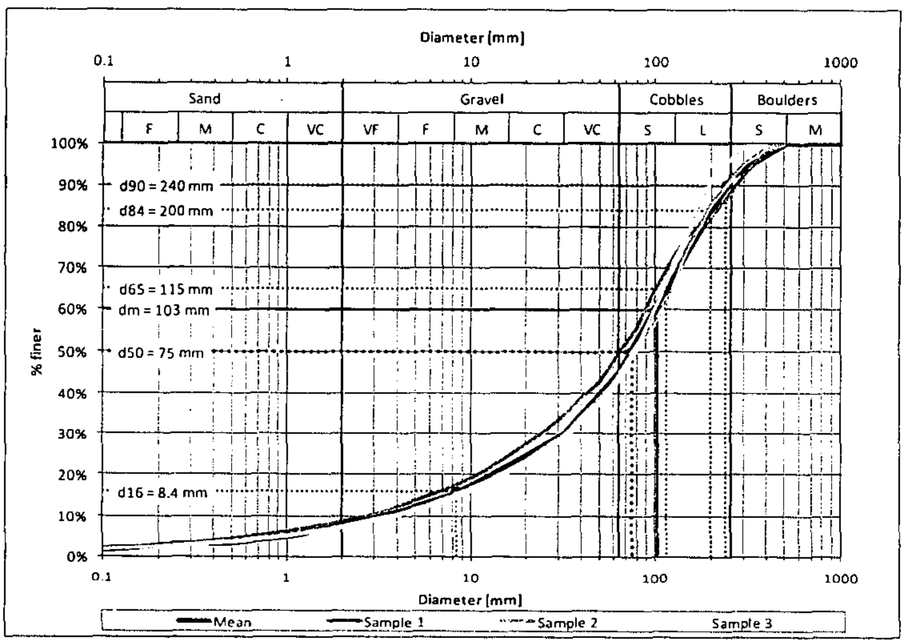


Figure 3-42: Grain Size Distribution of surface armor layer of Panjkora River at project site

3.9.4 Bedload

The bedload annual yield at the project site is computed with application of appropriate empirical predictors on basis of the site specific composition of bed material and flow conditions.

3.9.4.1 Flow conditions

The finally observed total energy losses depend on the grain size distribution of the surface material, the river slope and the relative submergence of the coarser particles of the armor layer. The aforementioned relationship is given by the following equation, which has been developed from measurements in mountainous rivers in northern Pakistan (Palt 2001).

$$\frac{k_{st}}{k_r} = 0.13I^{-0.28}\left(\frac{h}{d_{90}}\right)^{0.21}$$

Where:

k_{st} Strickler's coefficient for total losses including both skin friction and bedform losses

k_r Strickler's coefficient for skin (particle) losses (calculated with Meyer-Peter Müller (1949) equation.

I Average longitudinal river gradient at the project site

h Water depth

d_{90} 90% by weight of surface bed material is finer than this grain size

Under consideration of the aforementioned representative geometrical and morphological features of Panjkora River at the project site, the following water depth -discharge rating curve is calculated for a representative cross-section with application of the Manning-Strickler equation.

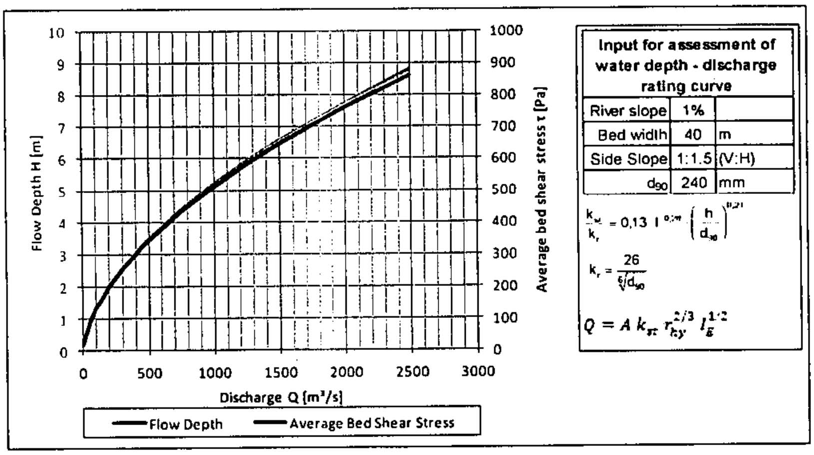


Figure 3-43: Flow depth - discharge rating curve for representative cross-section of Panjkora River at project site

The figure above includes also the calculated tractive shear stress exerted by the flow on the river bed as a function of the flow discharge. The calculated shear stresses are used for the assessment of the bedload transport rates which are presented in the subsequent section below.

3.9.4.2 Bedload computation

The assessment of bedload is performed with application of the following empirical predictors:

Meyer-Peter & Müller (1948)	**Application range** Bed material: sediment mixtures (6.4 mm - 30 mm) Slope: 0.04% - 2.3% **Equation** $$Q_B = \frac{8\sqrt{g}\rho_s B}{s-1}\left[\left(\frac{k_{st}}{k_r}\right)^{1.5} R\,I - \tau_{*cr}(s-1)d_m\right]^{1.5}$$ $\tau_{*cr} = 0.049$ $$\frac{k_{st}}{k_r} = \begin{cases} 1.0 & for\ flat\ river\ beds\ (no\ bed\ forms) \\ 0.5 & for\ step-pool\ morpology \\ 0.7 & selected\ value\ for\ Panjkora\ River \end{cases}$$
Smart & Jäggi (1983)	**Application range** Bed material: gravel (uniform material and sediment mixtures) Slope: 3% - 20% **Equation** $$Q_B = B\frac{4\rho_s\left(\frac{d_{90}}{d_{30}}\right)}{s-1} R\,v_m\,I^{1.6}\left(1-\frac{\tau_{*cr}(s-1)d_m}{R\,I}\right)$$ $\tau_{*cr} = 0.049$
Palt (2001)	**Application range** Based on bedload field measurements in northern Pakistan Bed material: gravel-boulders (sediment mixtures) Rivers with developed step-pool and riffle/pool morphology Slope: 0.6% - 12% **Equation** $$\varphi = \begin{cases} 9067\left[\left(\frac{k_{st}}{k_r}\right)^2\tau_*\right]^{5.23} & for\ \left(\frac{k_{st}}{k_r}\right)^2\tau_* < 0.22\ and\ q > q_c \\ 30.34\left[\left(\frac{k_{st}}{k_r}\right)^2\tau_*\right]^{1.45} & for\ \left(\frac{k_{st}}{k_r}\right)^2\tau_* > 0.22\ and\ q > q_c \end{cases}$$ $$q_{*c} = \frac{q_c}{\sqrt{g(s-1)d_{65}^3}} = 0.039\,I^{-1.16}$$ $$\varphi = \frac{q_B}{\sqrt{(s-1)g d_{50}^3}}$$ $$\frac{k_{st}}{k_r} = 0.13 I^{-0.28}\left(\frac{h}{d_{90}}\right)^{0.21}$$ $$\tau_* = \frac{R\,I}{(s-1)d_{50}}$$

Annotation	
Q_B	bedload transport rate [kg/s]
q_B	Volumetric bedload transport rate per river bed width unit [m³/s/m]
B	Average width of morphologically active river bed [m]
R	Hydraulic radius [m]
v_m	Average flow velocity [m/s]
h	Average flow depth [m]
I	Average longitudinal river bed gradient
ρ_s	Sediment density [kg/m³] (ρ_s = 2650 kg/m³)
s	Ratio of sediment to water density [-] (s = 2.65)
k_{st}	Strickler's coefficient for total losses including both skin friction and bedform losses
k_r	Strickler's coefficient for skin (particle) losses
k_{st}/k_r	Correction factor for increased energy losses due to bed forms [-]
τ^*_{cr}	Critical Shields parameter for entrainment into motion referring to d_m grain size of bed material [-]
d_m	Median grain size of bed material
d_{30}, d_{50}, d_{65}, d_{90}	30%, 50%, 65%, 95% by weight of bed material is finer than the d_{30}, d_{50}, d_{65}, d_{90} grain size respectively
φ	Dimensionless Einstein´s parameter for bedload transport rate
τ^*	Shields's parameters refering to d_m of bed material
q_c	Critical specific discharge for entrainment of fine gravel particles into motion [m³/s/m]

The aforementioned equations are selected among a plethora of available bedload predictors because they have been developed especially for gravel-bed rivers in mountainous regions.

- Meyer-Peter & Müller (MPM) (1948) formula is selected because it is the classical approach for gravel bed rivers with mild slopes. Its derivation was based on laboratory experiments. Panjkora River's slope at the project site (1%) is within the range of the tested river bed slopes (0.04% - 2.3%). The bed material of Panjkora River at the project site is however substantially coarser than the sand-gravel sediment mixtures used in the laboratory experiments.
- Smart & Jäggi (1983) formula is selected because it is the development of MPM equation for steep mountainous rivers. Its derivation was based on laboratory experiments. Panjkora River's slope at the project site (1%) is slightly out of the range of the tested river bed slopes (3% - 20%). The bed material of Panjkora River at the project site is substantially coarser than the sand-gravel sediment mixtures used in the laboratory experiments.
- Palt (2001) formula has been selected because it was developed for mountainous rivers with step pool and riffle pool morphological features. Its derivation was based on extensive field bedload measurements on 16 rivers in the mountainous Karakoroum region in Northern Pakistan. This region is characterized by similar geomorphological and climatic conditions as the project area. Panjkora River's slope at the project site (1%) is within the range of the river slopes (0.6% - 12%), where measurements were performed.

The long term mean annual bedload yield as calculated with application of the three aforementioned empirical predictors on the project site flow time series (1961-2016) is shown in the table below. The same table includes the corresponding bedload yields expressed as percent of the calculated mean annul suspended load. The latter is estimated to be 7.5 million tons/a, corresponding to a mean annual suspended concentration of 3300 ppm. Its computation is presented in the subsequent section below.

Table 3-17: Mean annual bedload yield as calculated with three different predictors

Predictor	Mean annual bedload yield	Mean annual bedload as percent of mean annual suspended load
Meyer-Peter & Müller (1948)	1.8 million tons/a	24%
Smart & Jäggi (1983)	25 million tons/a	330%
Palt (2001)	0.85 million tons/a	11%

Smart & Jäggi (1983) delivers an unrealistically high bedload yield (3 times the mean annual suspended load) even though it is one of the most recent developments of the classical MPM (1948) equation. For this reason is excluded from any further considerations. MPM (1948) and Palt (2001) result in comparable bedload yields. MPM (1948) however delivers practically double so much compared to Palt (2001). A practical method for evaluating the feasibility of calculated bedload transport rates has been presented by Lane & Borland (1951) in terms of guidelines which are shown below.

Table 3-18: Partitioning between bedload and suspended load according to Lane & Borland (1951) [Retrieved from Turowski et al. (2010)

Suspended load concentration	Bed material	Texture of suspended load	% bedload in terms of suspended load
Low < 1000 ppm	Sand	Similar to bed material	25% - 150%
	Gravel, rock or consolidated clay	Small amount of sand	5% - 12%
Medium 1000-7500 ppm	Sand	Similar to bed material	10% - 35%
	Gravel, rock or consolidated clay	25% sand or less	**5% - 12%**
High > 7500 ppm	Sand	Similar to bed material	5% - 12%
	Gravel, rock or consolidated clay	25% sand or less	2% - 8%

The above guidelines indicate that the bedload yield at the project site shall range between 5% and 12% of the mean annual suspended load, since the suspended concentration is 3300 ppm which can be classified as medium and the bed material consist predominantly from gravel, cobbles and boulders. On these grounds it is concluded that the most reliable prognosis of the bedload is delivered by the Palt (2001) empirical predictor which calculates a **mean annual bedload yield equal to 0.85 million tons/a** corresponding approximately to 10% of the mean annual suspended load. The calculated annual bedload yields for the time period 1961 - 2016 ranged from 0.07 million tons/a during dry years up to 6 million tons/a during extreme wet years.

The comparison of the daily flow values at the project site and the critical discharge for entrainment of fine gravel particles indicate that the bedload transport will occur during a time window of approximately 4 months extending from May until September. The bulk of bedload transport will take place however during a substantially shorter time period comprising sporadic days during which the armor layer is broken and the finer underlying material is exposed to the flow.

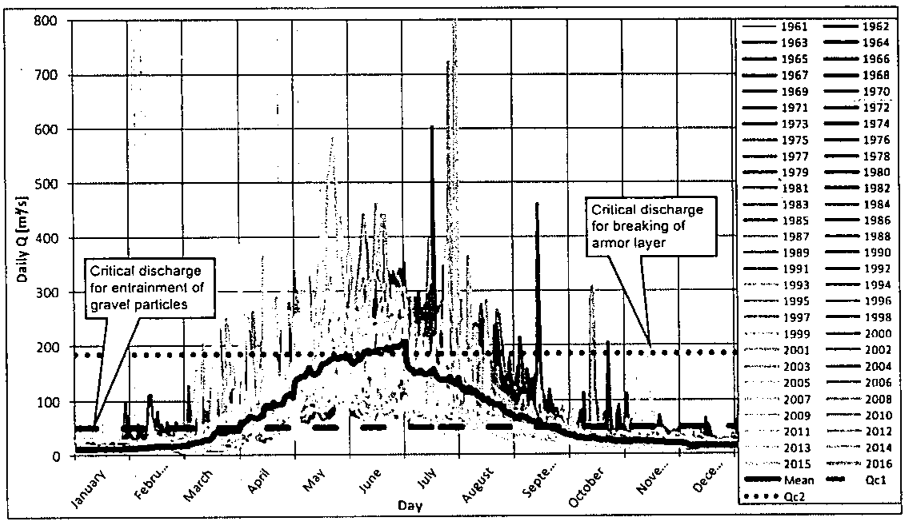


Figure 3-44: Comparison of mean annual hydrograph with critical discharge for entrainment of fine gravel and breaking of the armor layer

The calculated intra-annual distribution of bedload with application of the Palt (2001) predictor is shown in the figure below.

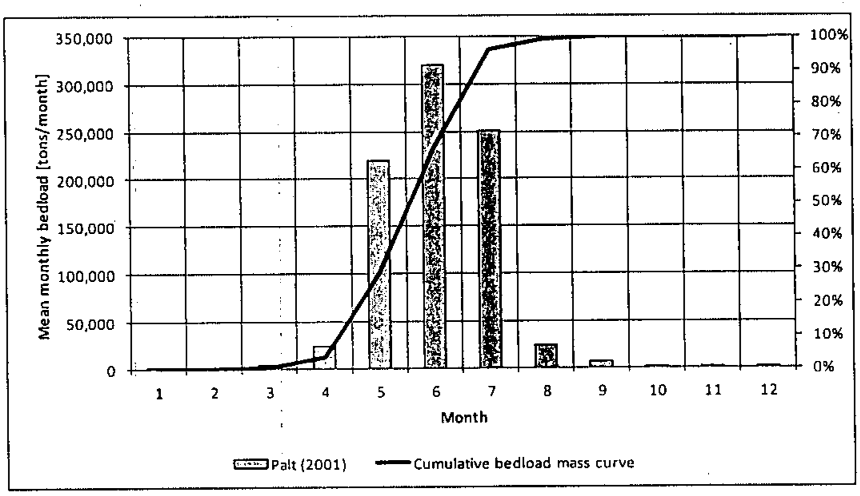


Figure 3-45: Mean monthly bedload yield at the project site

3.9.5 Suspended load

3.9.5.1 Data basis

The assessment of suspended sediment yield at the project site is based on an exceptionally sound data basis of suspended sediment field measurements. The measurements were conducted by WAPDA, close to the project site during a time period of 7.5 years.

Table 3-19: Key parameters of suspended sediment measuring station

Location	Zulam bridge on Panjkora River 65 km downstream of Sharmai proposed dam site 34°47'24.78"N 71°47'48.46"E
Catchment area	4285 km²
Time span of records	March 1999 till September 2006
Frequency of measurements	Once every 10 days (with intermediate measurements at pronounced flood events)
Measured data	• Concentration of suspended solids • Discharge at Zulam Bridge station • Grain size distribution of suspended sediment
Number of measurements	397 (suspended concentration) 80 (Grain size distribution of suspended load)

During pronounced flood events WAPDA measured also the gradation of the suspended matter. The grain size distribution of the suspended samples was determined whenever the measured sediment concentration was higher than 1000 ppm.

The sediment discharge data are presented in Annex. The location of the measuring station and the project site are shown below.

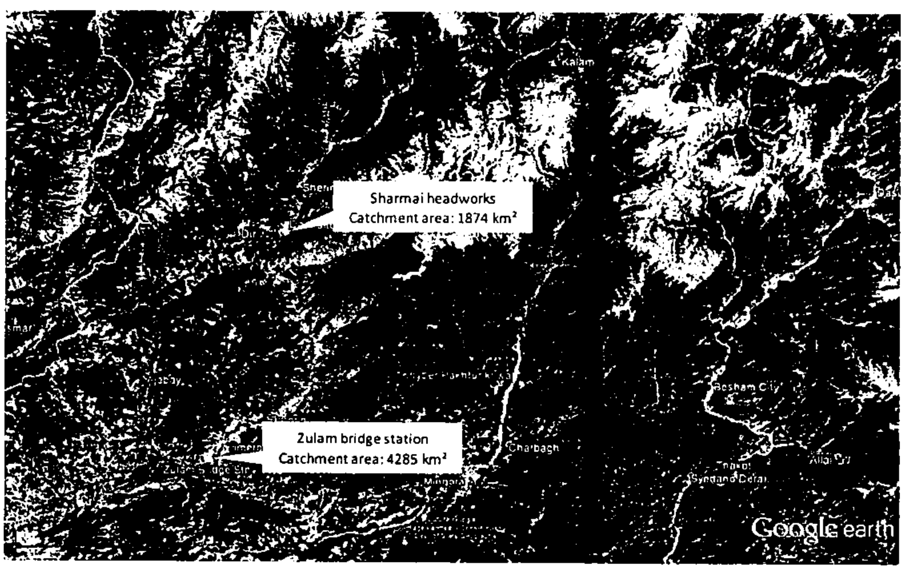


Figure 3-46: Project site and location of Zulam bridge suspended measurement station

3.9.5.2 Suspended load computation

3.9.5.2.1 Empirical regional assessment

In a first approach, the sediment yield of Panjkora river at the project site of Sharmai HPP is estimated with application of the empirical regional equation BQART, Syvitski and Milliman (2007), which is incorporated in the software RESCON 2. This equation was selected among a plethora of available empirical regional equations for the following reasons:

- To the knowledge of the Consultant it was developed from the most extensive river data base available. The derivation of the equation was based on measured sediment yields of 488 rivers, the catchment area of which covers 63% of the global land surface and is highly representative of global geology, climate and socioeconomic conditions.
- The sediment yield estimate is based on a large number of parameters including basin area, relief, lithology, precipitation, temperature and anthropogenic factors such as reservoir trapping in the catchment area, population density and gross domestic product. Finally, and most important it takes into account the effect of the ice cover.

The data used for estimation of the sediment yield of Panjkora river at the location of Sharmai HPP with the BQART equation are shown below.

Table 3-20: Data input for calculation of suspended load with BQART equation

Parameter		Unit	Value
Drainage Area	A	[km²]	1874
Mean Annual Discharge	Q	[km³/a]	2.25
Maximum Basin Relief	R	[masl]	5700
Average Basin Temperature	T	[°C]	11
Basin averaged lithology class		Sedimentary rocks, unconsolidated sedimentary cover, or alluvial deposits	
Ice cover as percentage of total drainage area	A_g	[%]	10%
Basin Trap Efficiency	T_E	[%]	0%
Basin human-influenced soil erosion class		Low human footprint (Density population (PD)<50/km²)	

The resulted sediment yield expressed in different units is shown below:

Table 3-21: Suspended yield according to BQART empirical regional approach

Parameter		Unit	Value
Long term mean annual suspended sediment load	Q_s	[kg/s]	265
		[Mt/a]	8.0
Specific suspended sediment Yield	q_s	[t/a/km²]	4245
Suspended Solids Concentration	C_s	[ppm]	3550

3.9.5.2.2 Sediment rating curve based computation

As a second approach, the suspended sediment yield at the project site is estimated by developing a sediment rating curve which correlates the available daily flow to the daily suspended sediment load expressed in tons/day. The rating curve is developed as follows:

- Calculation of daily loads at the project site based on the measured suspended concentrations at Zulam bridge and the calculated daily flows at Sharmai intake location.
- Plot of calculated daily sediment loads against corresponding water flows at the project site in log-log diagram and least square regression analysis for fitting a power function.

The derived sediment rating curve is shown in the figure below:

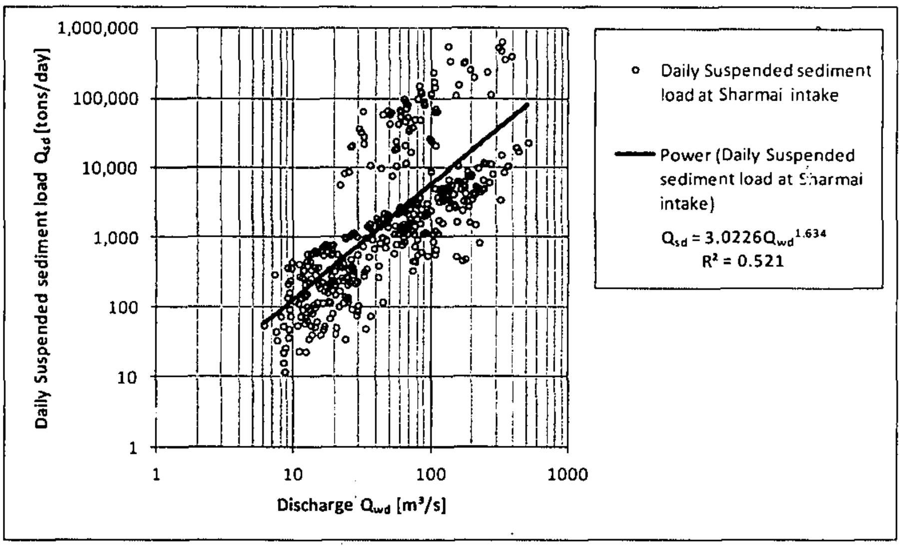


Figure 3-47: Suspended sediment rating curve at Sharmai intake (based on measurements taken at Zulam bridge the time period 1999-2006)

By applying the above presented sediment rating curve on the time series of daily flows at the Sharmai intake location it is calculated that the long term (time period 1961-2016) mean annual suspended sediment inflow at the project site will be 1.75 million tons/a.

The mean annual suspended load in the time period 1999-2006, i.e. the time period of the sediment measurement at Zulam bridge is calculated 1.6 million tons/a. That means that the long term mean annual suspended yield is approximately 10% higher than the mean annual yield of the measurement time period. This difference is justified by the fact that the time period of sediment measurement (1999-2006) was slightly drier than the long term (1961-2016) average hydrological year.

3.9.5.2.3 Temporal integration of sediment discharge records

As a third approach, the mean annual sediment load at Sharmai was determined as follows:

- Calculation of the daily sediment load at Sharmai as the product of the daily flow at the intake and the daily measured concentration at Zulam.
- Integration of the calculated daily sediment loads over time with the trapezoidal rule in order to determine the total suspended sediment yield during the time period of sediment measurements (03.1999 - 09.2006).
- Determination of the mean annual suspended yield by dividing the total sediment yield with the duration of the observation period (7.5 years).
- Correction of the calculated mean annual suspended yield by 10% in order to account for the fact that the measurement time period was slightly drier than the long term average hydrological year.

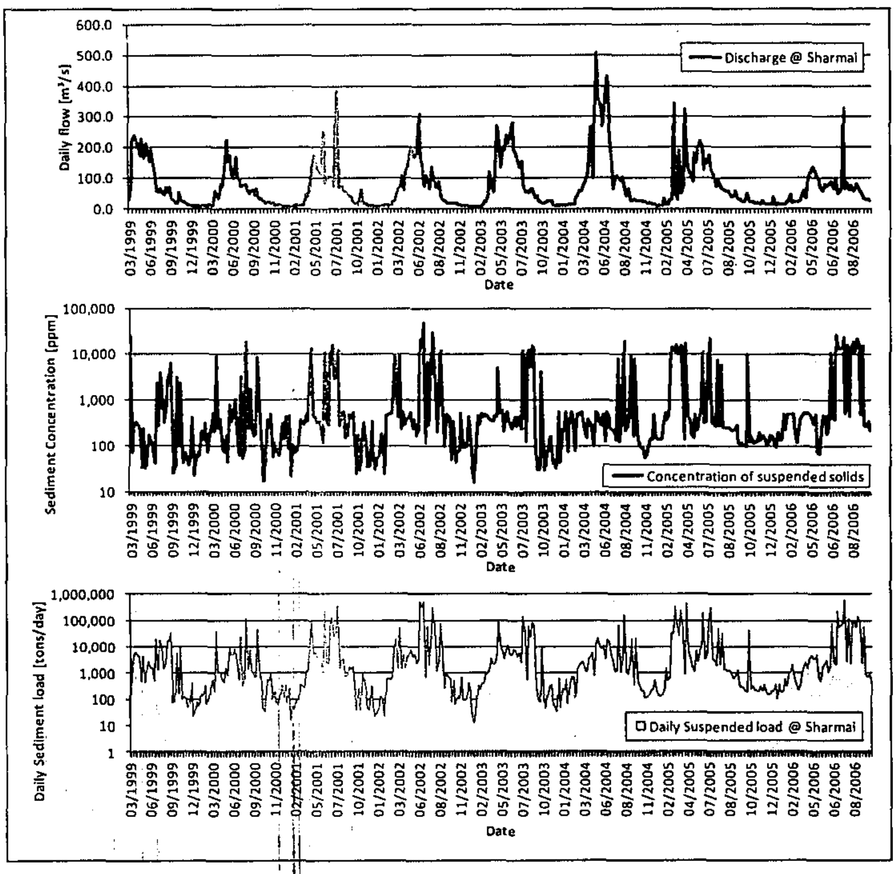


Figure 3-48: Assessment of mean annual suspended yield at Sharmai project site

Based on the above described methodology it has been calculated that the mean annual suspended yield at the Sharmai project site during the observation time period April 1999 - September 20006 was 6.8 million tons/a. In order to account for the fact that this time period was slightly drier than the average hydrological year, the aforementioned value of 6.8 million tons/a shall be increased by 10%. Hence, the **long term mean annual suspended sediment inflow at the project site is 7.5 million tons/a.**

3.9.5.2.4 Conclusion on mean annual suspended sediment load

The mean annual suspended sediment load at the project site has been calculated with three different approaches, the results of which are summarized below.

Table 3-22: Summary of suspended sediment computations at Sharmai with empirical regional, sediment rating curve and pragmatic approach

	Mean annual sediment load	Specific sediment yield	Mean annual concentration
Empirical regional	8.0 million tons/a	4250 tons/km²/a	3550 ppm
Sediment rating curve	1.8 million tons/a	950 tons/km²/a	780 ppm
Temporal integration	7.5 million tons/a	4000 tons/km²/a	3300 ppm

The above presented results indicate that the empirical regional approach and the temporal integration approach deliver comparable results. Contrary, the sediment rating curve approach delivers a substantially lower (approximately 4 times lower) mean annual sediment load. According to Ferguson (1986), statistical considerations have shown that the sediment load of a river is likely to be underestimated by methods in which unmeasured concentrations are estimated from discharge using a least squares regression for the logarithm of concentration.

On these grounds it is considered that the 3rd approach delivers the most reliable assessment, since it is based solely on the available site specific field measurements without introducing any over- or under-prediction bias. Therefore it is concluded that **the mean annual suspended sediment load at the project site will be 7.5 million tons/a, corresponding to a specific sediment yield of 4000 tons/km²/a.** The calculated annual suspended sediment yields for the time period 1999 - 2006 ranged from 2 million tons/a to 14 million tons/a.

The bulk of suspended load transport is observed in the time period March till September. Contrary during the winter months October till February practically no sediment transport takes place. The plot of the calculated mean monthly bedload and suspended sediment loads indicates that during this time period will occur approximately 90% of the mean annual suspended sediment inflow.

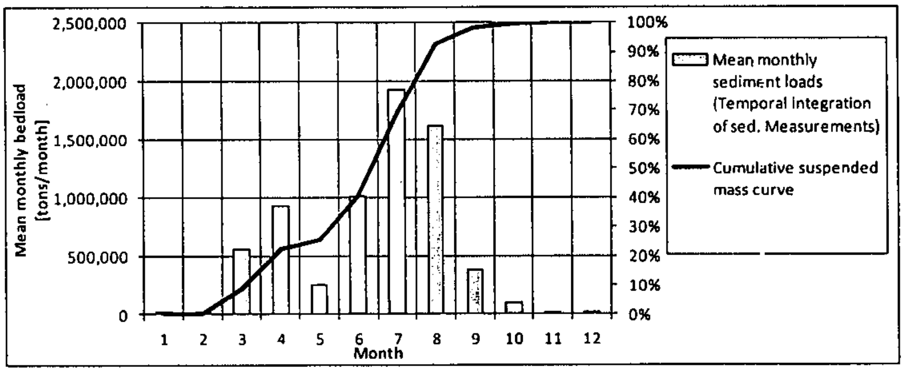


Figure 3-49: Mean monthly bedload yield at the project site

3.9.5.3 Grain size distribution of suspended load

During pronounced flood events, WAPDA measured also the sand, silt and clay fractional content in the suspended sediment samples. The grain size distribution of the suspended samples was determined whenever the measured sediment concentration was higher than 1000 ppm.

The variation of the sand content as well as the variation of the content of silt and clay in suspended load with the flow discharge is shown in the following figure.

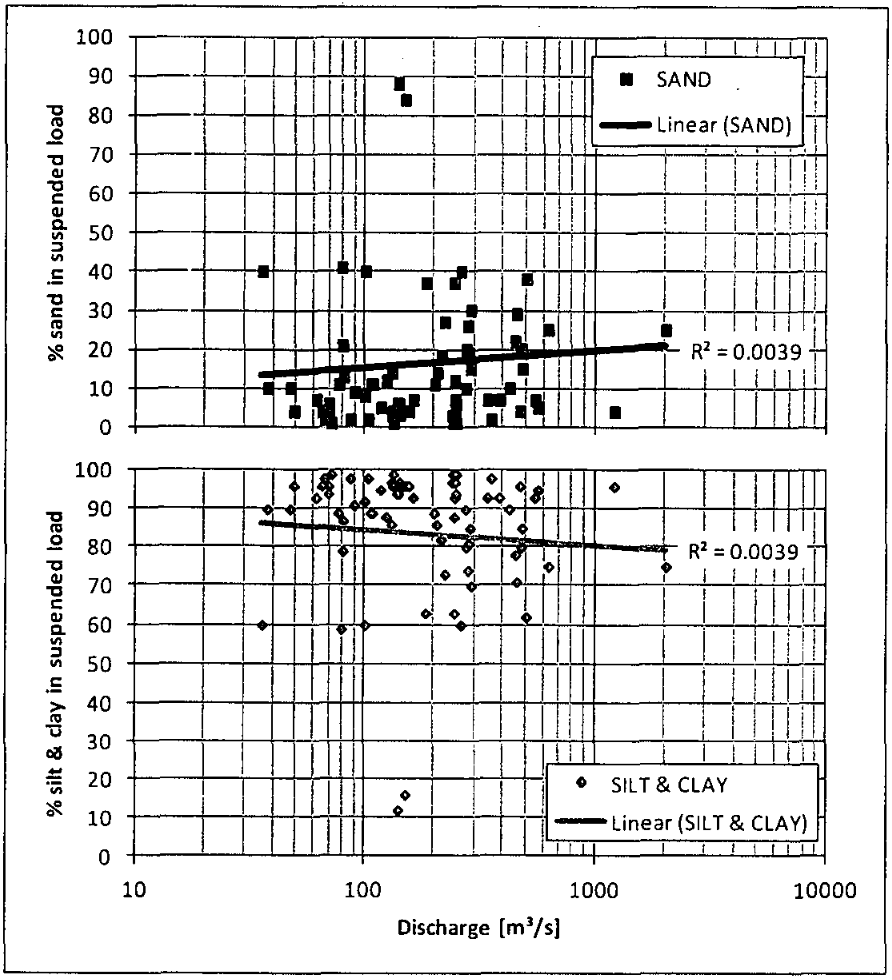


Figure 3-50: Variation of sand content and silt and clay content in suspended load with discharge

The plot above indicates that as expected there is a tendency of increase of sand and reduction of silt and clay in the suspended load as the discharge is increased. The correlation however is very week and therefore it is not possible to define the relationship between sand content and flow intensity. Following a conservative approach it is considered that the sand content in the suspended load will be constantly 25%. This is in agreement with the rule of thumb guidelines of Lane & Borland (1951), presented in chapter 9.4.2.

3.9.6 Total sediment load and its intra-annual distribution

The results of the above presented computations are herein summarized in order to conclude on the total sediment load expected at the project site. The following table presents the calculated mean monthly suspended, bedload and total sediment loads.

Table 3-23: Mean monthly and annual sediment loads at the project site

Month	Bedload [tons]	Suspended load [tons]	Total sediment load [tons]
January	7	5,906	5,912
February	24	18,288	18,312
March	4,059	606,008	610,066
April	23,992	1,018,428	1,042,420
May	220,085	270,822	490,907
June	319,612	1,113,515	1,433,127
July	250,470	2,123,515	2,373,986
August	24,218	1,779,997	1,804,215
September	6,371	422,027	428,399
October	1,503	104,867	106,370
November	41	21,069	21,111
December	16	6,788	6,805
Mean annual	**850,400**	7,491,231	8,341,631
Min (calculated)	71,729	1,937,898	2,009,627
Max (calculated)	5,984,924	15,316,730	21,301,654

The mean annual total sediment (including both bedload and suspended load) is expected to be approximately 8.3 million tons/a. According to the historic flow records the total sediment load might range between 2 million tons/a during dry years and 21 million tons/a during wet years.

The bulk of sediment transport is observed in the time period March till September. Contrary during the winter months October till February practically no sediment transport takes place. The plot of the calculated sediment loads indicates that during this time period will occur approximately 90% of the total mean annual sediment inflow.

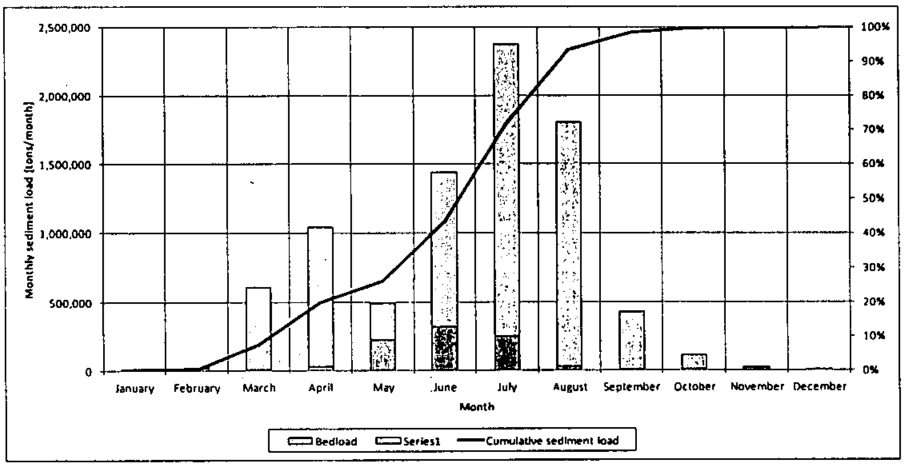


Figure 3-51: Intra-annual distribution of sediment load

3.9.7 Lifetime of reservoir

3.9.7.1 No sediment management

In this report section, the reservoir service lifetime for the case of no sediment management is assessed. The trap efficiency is calculated with the Churchil (1948) equation. This method is selected because according to Morris & Fan (1998) it is more appropriate for regularly sluiced reservoirs of relatively small size than the Brune (1950) method. The service lifetime is assessed for two dam site alternatives with different dam heights. The key geometrical features of the two reservoir alternatives as well as the involved hydrological and sedimentological parameters are summarized in the following table. The storage-elevation curves of the two reservoir alternatives are plotted in Figure 3-52.

Table 3-24: Reservoir geometries and hydrological key parameters

Parameter		Alternative 1 U/S	Alternative 2 D/S
Dam height	m	45 m	65 m
Normal operation water level	EL_{NWL}	1260 masl	
Minimum operation water level	EL_{MWL}	1255 masl	
River bed elevation at dam site	EL_{bmin}	1235 masl	1220 masl
Reservoir gross storage capacity	So_gr	3,200,000 m³	11,300,000 m³
Reservoir active storage capacity	So_a	1,300,000 m³	3,100,000 m³
Reservoir inactive storage capacity	So_d	1,900,000 m³	8,200,000 m³
Reservoir length	Lres	3675 m	5200 m
Mean annual water inflow	MAR	2,250 million m³/a	
Mean annual suspended load	MASs	7.5 million tonnes/a	
Mean annual bedload	p_b	0.85 million tons/a	
In situ density of sediment deposits	ρ_d	1.1 tons/m³	

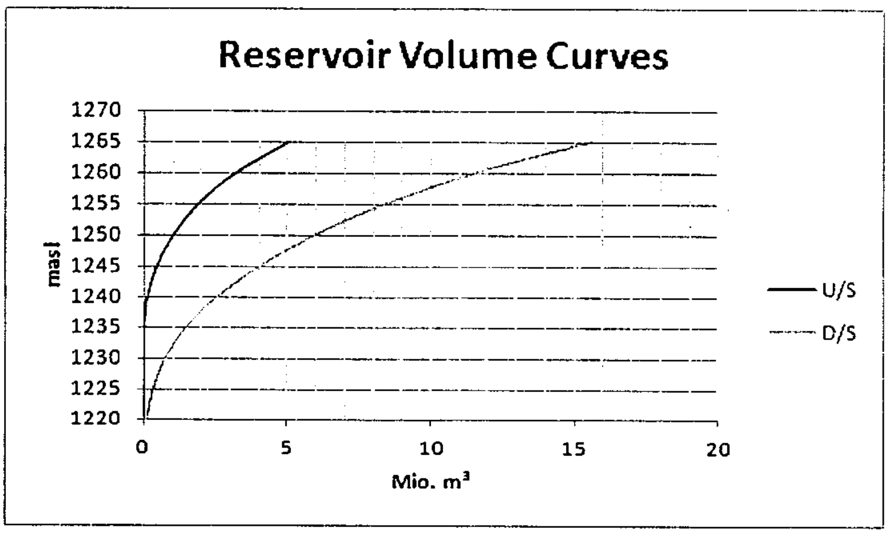


Figure 3-52: Storage - Elevation curves of two alternative reservoirs

The mean annual total sediment inflow amounts 8.3 million tons/a, which corresponds to 7.5 million m³/a assuming the above mentioned in situ density of sediment deposits of 1.1 tons/m³. The calculation of the trap efficiency with the Churchill (1948) method is shown in the figure below.

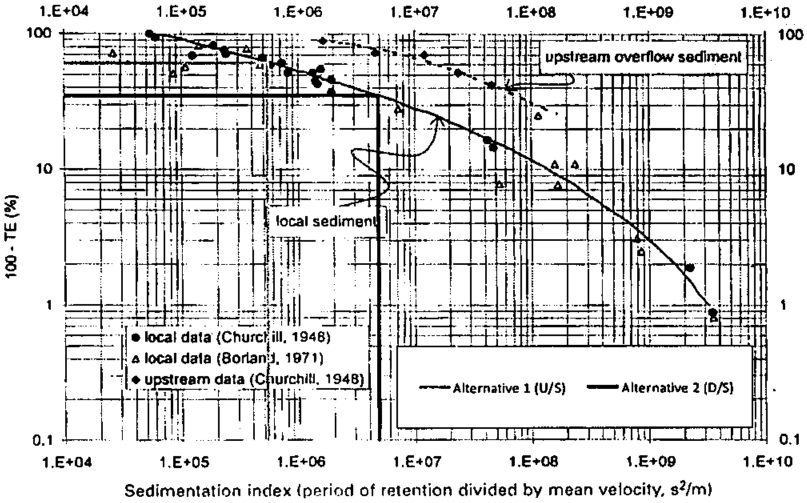


Figure 3-53: Assessment of suspended load trap efficiency for two reservoir alternatives

For the first alternative with the small reservoir, the release efficiency of suspended sediment is calculated to be 60%. Hence the trap efficiency is 40%. Similarly for the second alternative, the release efficiency is calculated 35% corresponding to a trap efficiency of 65%. For both alternatives it is considered that the bedload trap efficiency will be 100%. Based on these trap efficiencies, the mean annual deposits are calculated as follows:

- 1st alternative (Reservoir gross storage capacity = 3.2 million m³)
  Mean annual deposits = (100% x 0.85 + 40% x 7.5) /1.1
  Mean annual deposits = 3.5 million m³/a
- 2nd alternative (Reservoir gross storage capacity = 8 million m³)
  Mean annual deposits = (100% x 0.85 + 65% x 7.5) /1.1
  Mean annual deposits = 5.2 million m³/a.

It is deducted that if no sediment management is performed, the reservoir of the first alternative will be presumably fully sedimentated within the first year, while the reservoir of the second alternative after less than 2.5 years operation. In that case, the Sharmai HPP will become a mere Run-of-River scheme without the possibility of daily power peaking during the winter season.

The above presented argues that the second alternative will impound an unnecessary large reservoir, when compared to the first option. Therefore from the reservoir sedimentation point of view, the first alternative is clearly preferable since it requires the same sedimentation countermeasures as the second, with the possibility of cost savings because of the smaller dam height.

3.9.7.2 Sediment management

3.9.7.2.1 Selection of sediment management technique

In order to manage the extremely high sediment loads and protect the reservoir's active storage for to allow for the possibility to produce peak power during the winter months in a sustainable manner, sediment management has to be applied. Different approaches can be applied for the management of sedimentation (Annandale et al. 2016, Morris & Fan 1998).

The technical feasibility, economic performance and environmental impact of the state-of-the-art sediment management alternatives were evaluated with purpose the determination of the optimum method that can extend the lifetime of Sharmai reservoir active storage. The evaluation of the different sediment management techniques is summarized in the table below.

The assessment was performed with the RESCON 2 software which was developed by the Consultant for the World Bank Group (Efthymiou et al. 2017). The assessment is presented in detail in Annex.

Table 3-25: Evaluation of the state-of-the-art sediment management techniques

Sed. Mgmt. Technique	Evaluation and reasoning
Dredging	**EXCLUDED** • Technically feasible with mechanical dredger • NOT ECONOMIC VIABLE: Very high installation and annual operation costs • Adverse environmental impact associated with disposal of removed deposits
Hydrosuction Removal	**EXCLUDED** NOT TECHNICALLY FEASIBLE: Small head, long reservoir
Flushing	**EXCLUDED** • The flushing efficiency is questionable because of the coarse sediment deposits. • high water and energy generation losses due to water level drawdown, • IMPORTANT ADVERSE ENVIRONMENTAL IMPACTS: unnatural increase of sediment concentrations downstream of the reservoir during flushing operation
Sluicing	**SELECTED.** • As soon as the reservoir is filled up to the spillway crest level (within the first year of operation) this technique will become technically feasible. • Positive environmental impact since it restores the sediment transport continuity
By-Pass	**EXCLUDED** • The implementation of this method requires the construction of a 4 km long by-pass tunnel and a diversion weir. The technical feasibility depends on the geological conditions in the project area. • NOT ECONOMIC VIABLE: Very high implementation and maintenance cost
Density Current Venting	**EXCLUDED.** NOT TECHNICALLY FEASIBLE: The geometry of the reservoir does not favour the formation of density currents.

Sed. Mgmt. Technique	Evaluation and reasoning
Reduction of sediment inflow	SELECTED • Catchment management would require the implementation of a very large number of check dams upstream of the reservoir in very unfavourable terrain conditions and difficult to access. Annual maintenance under such difficult conditions would not be possible limiting thus the efficiency. Therefore, EXCLUDED • It is suggested however the implementation of a boulder trap with purpose the retention of cobbles and boulders before they enter the reservoir.

Flushing is considered as less reliable than sluicing with regards to protection of reservoir of active storage because of the high bedload transport rates. Generally, it is more difficult to remobilize a coarse particle that has already settled down (flushing) than to maintain the same particle in transport in order to route it through the reservoir (sluicing). A big part of the annual sediment deposits are expected to be gravel and cobbles and therefore the efficiency of flushing might be limited only in the vicinity of the outlet structure.

Experiences in HPPs of similar type in mountainous regions (e.g. Kali Gandaki, Nepal) indicate that sluicing was able to maintain in a sustainable manner the required active storage. Furthermore flushing will be associated with adverse environmental impacts downstream of the reservoir, contrary to sluicing. In any case a bottom outlet should be foreseen in the weir structure close to the intake in order to allow the removal of any deposits might accumulate there and prevent thus the entrance of coarse particles to the waterway.

During high flood events the river might transport boulders which will be retained in the reservoir even during sluicing. For this reason a boulder trap should be implemented upstream of the reservoir headwaters and should be regularly cleared by mechanical means in order to maintain its functionality for the case of extreme flood events. It is assumed that the boulder trap will retain approximately 400,000 m³ of the incoming bedload (approximately the coarsest 50%). For the removal of the retained material, an additional yearly maintenance cost has to be foreseen, to be estimated depending on the specific cost in USD/m³. The annual cost may range between 0.5 - 1.0 USD/a as a first rough indication. This needs to be further evaluated and confirmed.

Finally during high flow events in the time period June till August the suspended sand concentration in the river and on the flow diverted into the turbines will spike. For this reason when the sand concentration exceeds a critical value, the energy generation shall be ceased in order to protect the turbines from hydro-abrasive wear. It might be considered to shut down the power production when the flow discharge exceeds the value of 220 m³/s which, according to the presented Flow Duration Curve (FDC) corresponds to approximately 5% exceedance probability. This should be tested in a physical model, if deemed appropriate.

Based on the above presented evaluation of the available sediment management techniques it is suggested to apply the combination of the following sedimentation countermeasures:

- seasonal reservoir drawdown and sluicing during the summer period
- implementation of a boulder trap upstream of the reservoir
- shut down of power production during extreme flood events
- preventive coating of parts of generating equipment exposed to abrasion.

3.9.7.2.2 Seasonal sluicing

The time path of the reservoir storage development for the scenario of implementation of seasonal sluicing is shown in the following figure. The calculation is performed with RESCON 2 software. The calculation is based on the geometrical and hydrological key parameters included in Table 3-24 for the first reservoir alternative (U/S) as well as the intra-annual distribution of sediment load presented in section 3.9.6. As a first approximation, the following seasonal operational mode for sluicing has been specified.

Table 3-26: Sharmai HPP operational mode incorporating sediment management by means of seasonal sluicing

Time season	Operational mode	Reservoir Water level
Monsoon April - September	Sluicing mode	Minimum Operating Water Level 1255 masl
Winter October - March	Impounding mode	Full Supply Water Level 1260 masl
Flood Q > 220 m³/s	Cease power production	

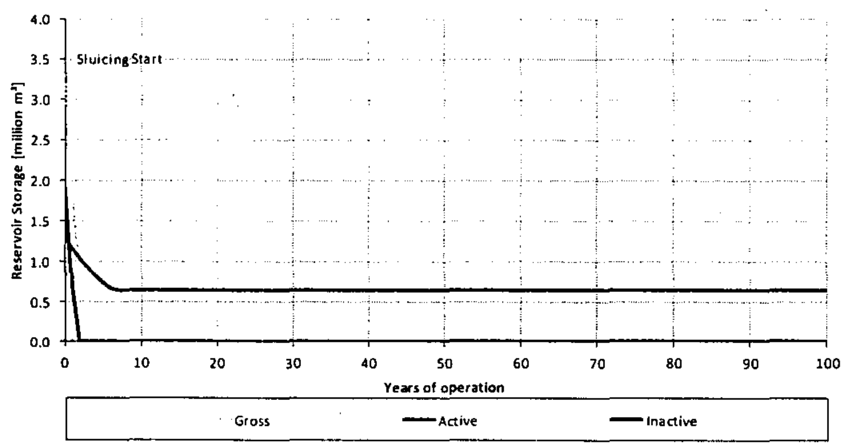


Figure 3-54: Time path of reservoir storage if drawdown sluicing is performed

The calculation indicates the aforementioned sluicing procedure will stabilize sustainably the reservoir active storage to approximately 650,000 m³. A preliminary power production analysis presented in the Inception

Report indicated that the minimum required active volume to allow the utilization of Q90 (net turbine flow with the availability of 90% amounts to about 635,000 m³. A further water level drawdown would improve the performance of sluicing and a larger active storage would be sustained.

3.9.8 Numerical modeling of flow conditions in reservoir

The mean annual inflow hydrograph, which is plotted in Figure 3-44 with black solid line, indicates that the bulk of sediment transport during the average hydrological year is expected to occur when the water inflow ranges between 25 m³/s and 200 m³/s.

The flow velocity in Sharmai reservoir has been calculated with the widely used one-dimensional numerical model HEC-RAS v. 5.03. The simulation was based on the recently measured topography of the reservoir (pre-impoundment conditions). As downstream boundary condition, the water level was fixed at 1260 masl, i.e. the normal operating water level. The simulated flow velocities are plotted in the following figure.

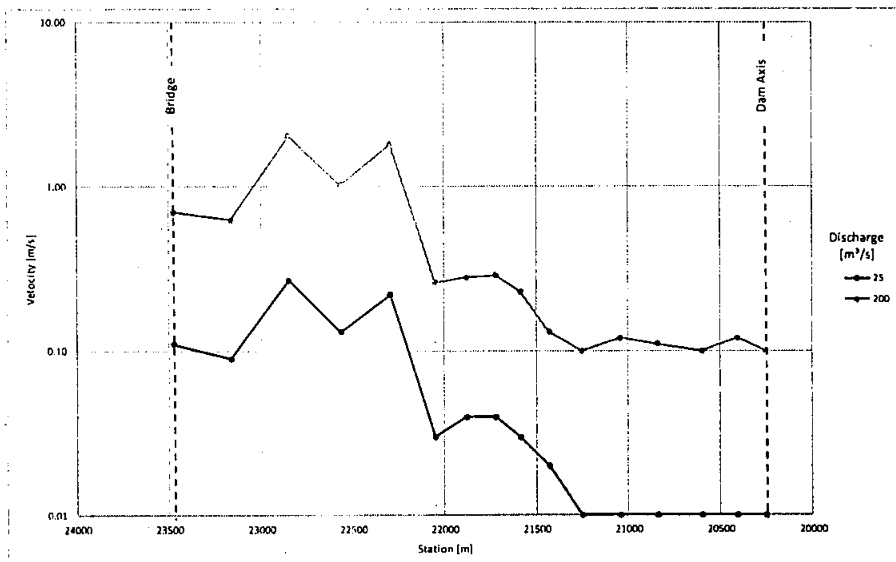


Figure 3-55: Longitudinal profile of flow velocity in Sharmai reservoir (HEC-RAS simulation)

The numerical simulation shows that the flow velocity drops as the dam axis and accordingly the deeper reservoir section is approached.

- In the upper part of the reservoir, which extends 2 km downstream of the bridge, the velocity ranges between 0.1 m/s for the low water inflow of 25 m³/s and 2 m/s for the water inflow of 200 m³/s.
- In the lower and deeper part of the reservoir extending 1 km upstream of the dam, where the bulk of sediment accumulation is expected to occur, the flow velocity ranges between 0.01 m/s for the flow water inflow of 25 m³/s and 0.1 m/s for the water inflow of 200 m³/s.

The application of the classical Hjulstrom diagram for the above presented range of reservoir flow velocities allows a preliminary assessment of the grain sizes that will be deposited in the reservoir or entrained into motion (re-suspension). Its application is presented in the figure below.

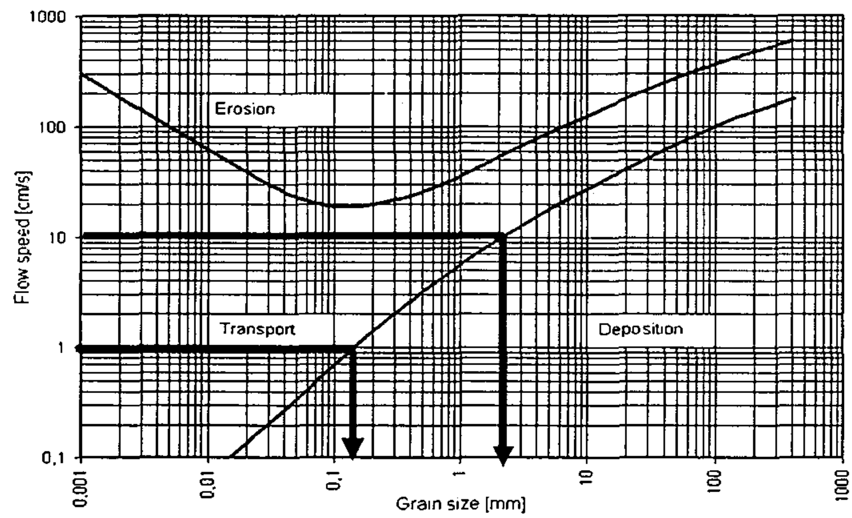


Figure 3-56: Application of Hjulstrom diagram for the Sharmai project specific flow velocities (pre-impoundment geometry and normal operation)

The major conclusions of the application of Hjulstrom diagram for the calculated flow velocities in the lower part of the Sharmai reservoir (extending one km upstream of the dam) are summarized below.

- The grain size of the particles that will be deposited will depend on the flow discharge. For low flow conditions (25 m^3/s) the particles coarser than 0.15 mm will be deposited. For high flows of 200 m^3/s, the particles coarser than 2 mm will be deposited. It is pointed out that the Hjulstrom diagram was developed for normal flow conditions that are usually attained in long open channel and not for gradually varied flow conditions imposed by man made barriers that slow down the flow. Therefore, the threshold between transport and deposition conditions as indicated by this nomograph is not realistic for the case of reservoir sedimentation conditions and in the reality the amount that will be retained in the reservoir will be substantially higher.
- bedload, i.e. gravel material will be fully retained in the reservoir.
- Sand will be either retained in the reservoir or will enter the waterway
- One part of the silt will be retained in the reservoir or will enter the waterway and one part will be routed out of the reservoir through the spillway.

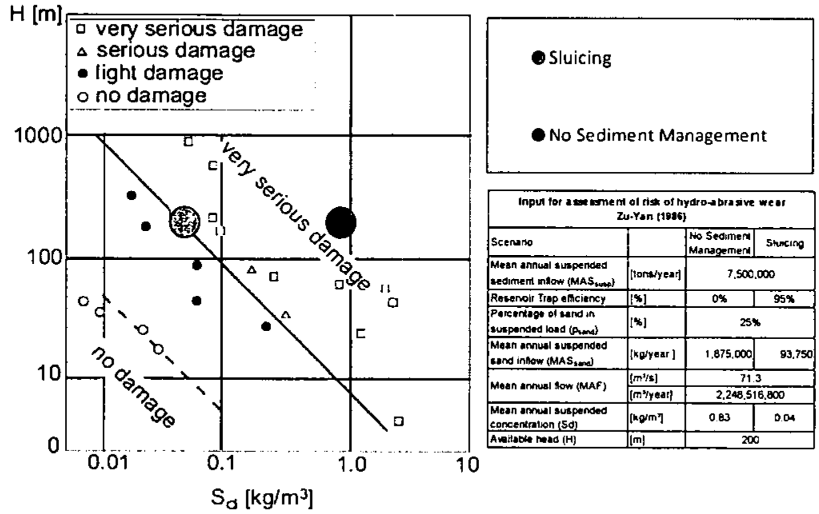
Based on the aforementioned theoretical considerations it is concluded that the assessment of the trap efficiency with the empirical method of Churchill is realistic. Therefore, the lifetime of the reservoir if no sediment management is applied will be less than one year. It appears that the water level drawdown can provide a valuable means for manipulating the deposition spatial pattern since it will have as result an increase of the flow velocities in the reservoir. This sediment management activity can protect the active storage and consequently allow the daily peaking operation in a sustainable manner. The water level drawdown can be achieved through the bottom outlet and the spillway crest.

3.9.9 Necessity for desander

If no sediment management is applied. the Sharmai headpond will be fully sedimentated within the first year of its operation and its trap efficiency will drop to zero. In that case, the mean annual concentration of sand material entering the waterway will be approximately 0.83 kg/m³, considering that 25% of the suspended load will consist of sand. This assessment is based on the measured sand concentrations at Zulan bridge. The petrographic analysis of suspended sediment samples indicated that the content of highly abrasive quartz minerals in the transported sand exceeds 30%. The results of the petrographic analysis are included in Annex.

If sluicing is performed, the concentration of suspended sand on the water diverted into the power waterway will be reduced because the reservoir storage will not be eliminated rather it will be stabilized sustainably to approximately 0.65 million m³. Therefore, the biggest part of suspended sand will be sluiced out of the reservoir. Assuming that the reservoir will act as a desilting facility with trap efficiency of 95%, the mean annual concentration of suspended sand in the diverted water will drop to 0.04 kg/m³.

In both cases, the calculated concentrations of abrasive sand in the turbined water together with the head of approximately 200 m pose a high risk of hydro-abrasive wear of the generating equipment. This assessment is based on the empirical criterion of Zu-Yan (1996) which is illustrated in the following figure. Therefore, the construction of a desander is of uppermost importance in order to reduce the downtimes and the costs for overhaul of the generating equipment and to protect its efficiency.



Input for assessment of risk of hydro-abrasive wear Zu-Yan (1986)			
Scenario		No Sediment Management	Sluicing
Mean annual suspended sediment inflow (MAS_{susp})	[tons/year]	7,500,000	
Reservoir Trap efficiency	[%]	0%	95%
Percentage of sand in suspended load (p_{sand})	[%]	25%	
Mean annual suspended sand inflow (MAS_{sand})	[kg/year]	1,875,000	93,750
Mean annual flow (MAF)	[m³/s]	71.3	
	[m³/year]	2,248,516,800	
Mean annual suspended concentration (Sd)	[kg/m³]	0.83	0.04
Available head (H)	[m]	200	

Figure 3-57: Assessment of risk of hydro-abrasive wear of equipment for the case of no sediment management and eliminated reservoir trap efficiency

3.9.10 Conclusions

Panjkora River is a steep mountainous river. Its river morphology is characterized by the distinct surface armor layer and the riffle-pool sequences. The latter have an important impact on the energy losses and the river bed stability. This has been considered in the herein presented assessment through application of carefully selected equations that were developed on basis of field measurements on river of comparable character in northern Pakistan.

The mean annual suspended sediment yield has been calculated to be 7.5 million tons/year. The assessment was based on regular suspended load measurements at Zulan bridge during the time period 1999 - 2006.
The mean annual bedload yield has been calculated to be 0.85 million tons/year. The assessment was based on the application of an empirical predictor that was derived from bedload field measurements in the region of Karakorum, north Pakistan. Bedload corresponds to 11% of the mean annual suspended sediment inflow in the project site. 90% of sediment transport occurs in the time window end March - mid September.

If no sediment management is applied, the headpond of Sharmai HPP will be fully silted within the first year of operation, since the sediment inflow exceeds by far the available storage capacity. In that case Sharmai HPP will be a pure Run-of-River scheme without the possibility of daily power peaking during the winter time.

In order to prevent the active storage loss due to sedimentation and sustain thus the daily power peaking operation it is recommended to apply seasonal sluicing with water level drawdown to the level of the minimum operating water level during the high flow season. Preliminary calculations performed with the software RESCON 2 indicate that the active storage can be stabilized to approximately 0.65 million m^3. It is our strong recommendation to confirm the results presented in this sense by means of physical model.

It is also recommended to implement a boulder trap upstream of the reservoir headwaters in order to retain boulders that might enter the reservoir during extreme flood events. The functionality of the boulder trap shall be regularly maintained through mechanical excavation. It is roughly estimated that the maintenance of the boulder trap will cost approximately 1 million US$/a.

In order to protect the generating equipment against abrasion it is recommended to install a desander at the beginning of the headworks. Furthermore, it could be beneficial to apply protective coating on the exposed parts of the turbine. Finally, it is recommended to cease the energy generation during high flood events when the suspended concentration spikes. As a preliminary assessment it is suggested to shutdown the HPP operation when the discharge exceeds the threshold of 220 m^3/s, which corresponds in an exceedance probability of 5%.

4. Topography and Site Accessibility

4.1 Topographic and Bathymetric Survey

4.1.1 Introduction

Topographic planning data are fundamental geographic components for the execution of various planning projects.

Based on the scope of work and the terms of reference concerning the tunnel project area for the Sharmai hydropower project in Pakistan, the Consultant decided to create the topographic planning data for the subject project on the basis of satellite data and the remote sensing technology.

Technological progress concerning remote sensing techniques made it possible to extract reliable topographic data for planning purposes with sufficient accuracy within a short period of time. Therefore, the use of satellite data for topographic information has become a major and popular factor for extensive project areas.

The satellite data was used to meet the accuracy requirements of the project and are suited for the creation of topographic planning data for this project.

4.1.2 Scope of Work

The main activity related to this project area is to create topographic planning data on the basis of satellite data and the remote sensing technology. Preparation of the topographic data base for Sharmai HPP was performed for the following areas of interests:

- tunnel area
- dam and powerhouse sites
- particular sites of interest: roads, bridges, man-made structures.

Topographic data base for the tunnel area was based on the following scope of work:

- acquisition of Pleiades satellite data with a resolution of 50 cm for the area of approx. 110 km^2
- processing of satellite imagery
- geo-referencing
- ortho-rectification
- mosaicking
- development of a Digital Elevation Model (DEM) for the area of approx. 250 km^2
- preparation of contour lines with an equidistance of 5 m and 25 m
- ArcReader GIS by ESRI.

For the dam and powerhouse areas, more accurate topographic data base was created, including the following tasks:

- densification of the terrain points
- construction of break lines such as river shore lines and road edges
- development of a Digital Terrain Model (DTM) for the DAM project area of 0,56 km^2 and creation of contour lines with an equidistance of 1 m and 5 m
- development of a Digital Terrain Model (DTM) for the PH project area of 1,43 km^2 and creation of contour lines with an equidistance of 1 m and 5 m
- ArcReader GIS, Digital topographic map with layout.

The completion of the topographic planning data takes place on the basis of satellite data and the remote sensing technology.

The stereoscopic satellite data of the sensor WorldView-2 (Digital Globe, USA) with a resolution of 50 cm is used for the photogrammetric processing and development of the DTM. The geo-referencing of the satellite data is based on the terrestrially measured points of Geomatics and Engineering Services (Pvt.) Limited.

4.1.3 Reference System and Map Projection

A consistent, clearly defined geodetic reference system is a decisive and essential issue concerning the creation of topographic planning data. The following reference system was specified to create the topographic planning data and implement the project:

Horizontal:
- UNIVERSAL TRANSVERSE MERCARTOR (UTM)
- ZONE 42 NORTH
- GCS WGS 1984/ DATUM WGS 1984
- SPHEROID WGS 1984
- FALSE EASTING 500000
- FALSE NORTHING 0
- CENTRAL MERIDIAN -69 / SCALE 0,9996
- Authority: 32642 (EPSG)
- The reference system for elevation: EGM 96.

Vertical:
- Orthometric height (EGM 2008 /WGS84).

The use of this geodetic reference system, which is used all over the world, simplifies the process of merging. Further processing of the various spatial data from that can be required during the course of the project from different sources.

4.1.4 Tunnel area

4.1.4.1 Satellite data

For this project satellite data of the sensor Pleiades was acquired. Pleiades belongs to AIRBUS Defense and Space (France). It is currently one of the best commercial satellites. The satellite data was processed with a resolution of 50 cm and is suitable up to a scale 1:2,000. The satellite image data was recorded in September 2016. The multi-spectral, radiometric resolution enables a detailed interpretation of topographic elements and land use.

To transform a satellite image into a map-like projection ortho-rectification must be applied. Ortho-rectification is the process of correcting imagery for distortion using elevation data and camera model information.

The following ortho-photo was created for the project area:

- ortho-photo from sensor Pleiades
- resolution 50 cm, RGB
- area approx. 110 km^2
- formats ECW, TIF.

4.1.4.2 Relief Data Digital Elevation Model (DEM)

The free-to-reach digital elevation model (DEM) ALOS World 3D (AW3D30) was used to create the relief data. The AW3D30 was released in 2015 by the Japan Aerospace Exploration.

Agency (JAXA). AW3D30 with 30m grid size was generated using the traditional optical stereo matching technique as applied to images acquired by the Panchromatic Remote-sensing Instrument for Stereo Mapping (PRISM) sensor onboard the Advanced Land Observing Satellite (ALOS). The actuality of the ALOS PRISM output satellite image data lies between years 2001 and 2006.

The accuracy analysis of an independent institution using 274 GCPs has determined a statistical error value of -4.04 m to +16.80 m. The accuracy depends on different vegetation, terrain, building and terrain characteristics. AW3D30 is used to derive contour lines with an equidistance of 5 m and 25 m.

The relief data was processed for an area of approx.250 km^2.

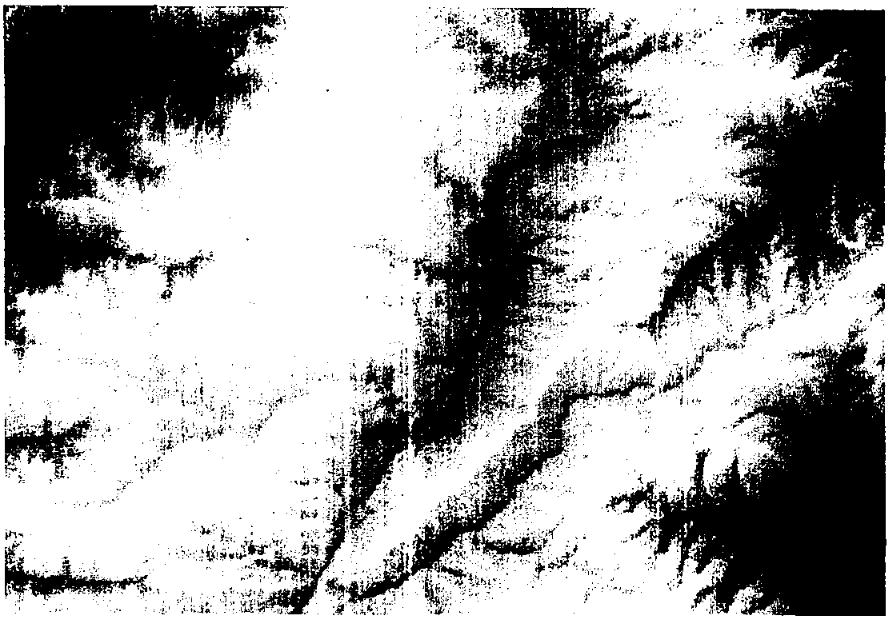


Figure 4-1: Example of DEM

4.1.5 Dam and powerhouse areas

4.1.5.1 Terrestrial survey on the site

The terrestrial and bathymetric survey work was carried out by Geomatics and Engineering Services (Pvt.) Limited, who delivered the raw results for further processing.

Surveying results were used for the completion of the topography using the stereoscopic satellite data. Satellite data orientation was performed based on survey data. Using this process, it was ascertained that the survey data for Dam and PH were not measured in a uniform reference system and showed a positional offset (X = approx. 20m, Y = approx. 2m).

The survey data seem to be consistent for each location, but the horizontal location of the sites is not consistent. The vertical readings seem to be plausible and no height offset could be detected.

Thus, it was necessary to orientate the stereoscopic satellite data for the Dam project area and the PH project area separately with respective survey points.

During the photogrammetric processing of the stereoscopic satellite data the survey points, which indicated a gross deviation or were not DTM conform, were excluded. These points were marked as "not used" in the final database.

4.1.5.2 Satellite data

The stereoscopic satellite data of the sensor WorldView-2 (Digital Globe, USA) was used for the photogrammetric processing. The satellite data for the dam and powerhouse areas was processed with a resolution of 50 cm and recorded in October 2017. This data is suited for the creation of a high-accuracy digital terrain model and topographic planning data.

4.1.5.3 Digital Terrain Model (DTM)

During the processing of the Digital Terrain Model (DTM) the obtained stereo images were analyzed using manual photogrammetric methods. Additional terrain points and break lines were measured during photogrammetric 3D processing, which are necessary for the creation of the DTM.

The project areas, which were not included in the surveying campaign on site, are measured photogrammetrically.

The resulting DTM has a pixel spacing of 1 m. DTM is used to derive contour lines describing the terrain with an equidistance of 1 m and 5 m.

Figure 4-2: Example of DEM at Dam area

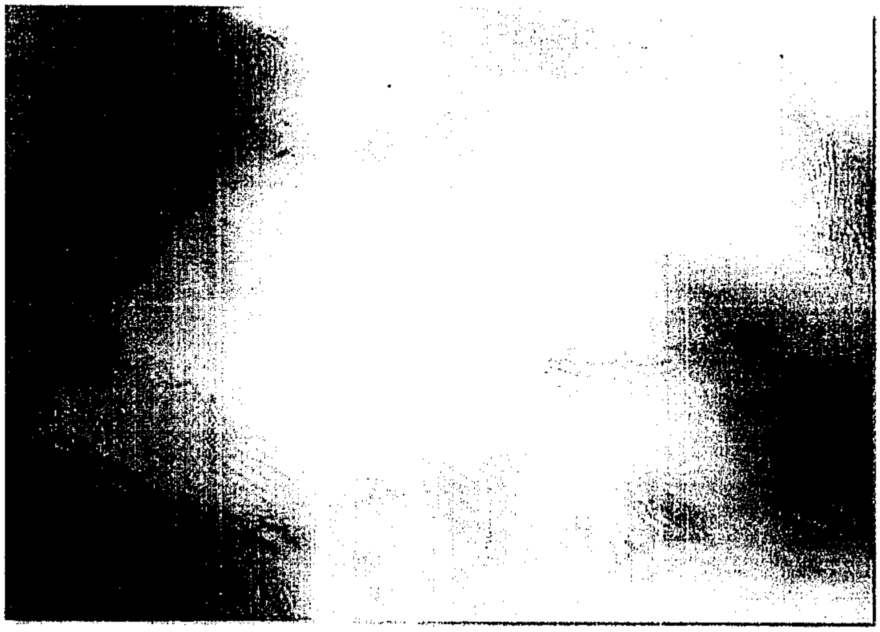


Figure 4-3: Example of DEM at Powerhouse area

4.1.6 ArcReader GIS

The topographic planning data is provided to the project team in ArcReader GIS by ESRI. ArcReader is a free, easy to use desktop mapping application that allows users to display, analyze, graphically draw and plot maps in different scales and with selected content. The layout has been adjusted in accordance with the Client's requirements.

This GIS offline solution includes the spatial data that are generated during the course of the project. The GIS is a professional planning platform for all people involved in this project.

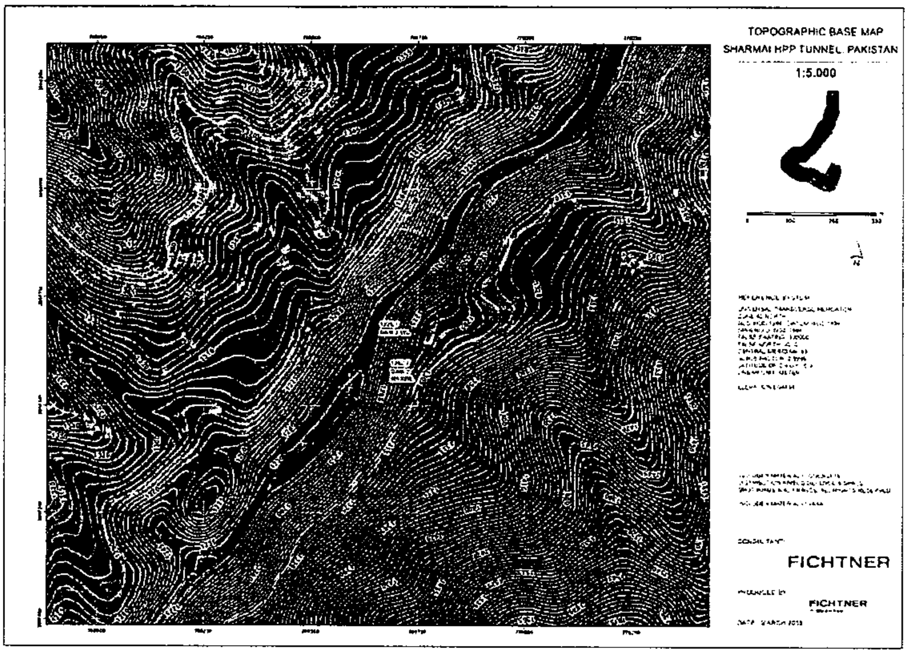


Figure 4-4: Example of ArcReader (1)

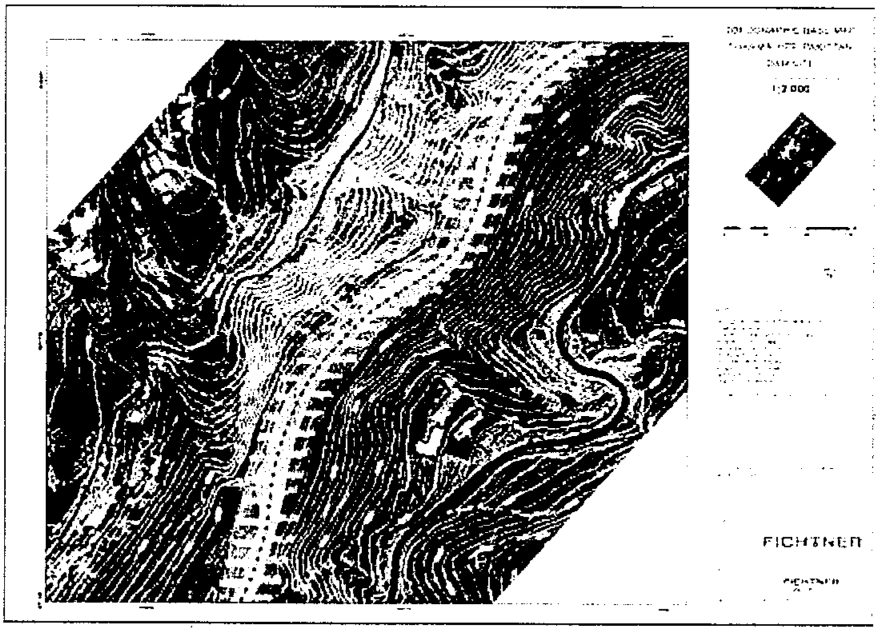


Figure 4-5: Example of ArcReader (2)

4.2 Site Communication, Transportation and Access

4.2.1 Introduction

Sapphire Electric Power Company Limited (SEPCL) is planning to implement a hydropower project Sharmai near Darora. SEPCL intends to use the existing berths of Port Qasim to unload the heavy equipment for its power plant. After unloading, the equipment will be transported using existing PQA road – National Highway – Motorway – Mardan to Buttkhela road – Buttkhela to Chakdara road – Chakdara to Dir, Chitral road – SEPCL Hydropower Plant Site at Darora. Total length of the roads from the Port Qasim to the project site amounts to about 1,700 km.

Sites have been visited and tentative routes have been physically surveyed to identify the most feasible route for transportation of heavy equipment. This report presents the capability of existing road and structures on which the trailers will move for transportation of heavy equipment.

4.2.2 General data

4.2.2.1 Details of equipment to be transported

The following information regarding equipment dimension and loads were assumed based on the preliminary analysis for confirming the route for transportation of heavy equipment of Hydropower Project.

Table 4-1: Details of Equipment

Description	Weight Approx. (tons)	Size of Package		
		Length (m)	Width (m)	Height (m)
Generator	80 (rotor)	3,5	3,5	3
Runner	2,75	3,5	3,5	2,5
Gantry Crane	35	10	2	2

4.2.2.2 Port Qasim Berths for Unloading Equipment from Ships

Port Qasim Authority (PQA) has four (04) Marginal Wharf Berths as shown on Figure 4-6. Each berth is 200 meter in length and having a draught of 9.5 meters. The area available behind quay wall is (200 x 222) Square meters.

Heavy equipment can be unloaded at Marginal Wharf Berth # 1 & 3. Cranes of required capacity are to be arranged and deployed for unloading the heavy equipment at berths because no such arrangement for unloading the heavy equipment is available at the berths.

4.2.3 Proposed route for transportation of heavy equipment

Equipment will be transported from Marginal Wharf Berths to the proposed Sapphire Hydro Power Project Site through existing Port Qasim road network. The proposed route is shown in various Figures. For the description of the proposed route, it has been divided in the following sections.

4.2.3.1 SECTION 1: From Port Qasim to National Highway N5

4.2.3.1.1 Segment I: Marginal Wharf Berths to Fire Station

This section is dual carriageway having three (03) lanes in each direction. The width of each lane is 3.5 meters. There is one right turn in this section. The equipment will be easily transported through this section.

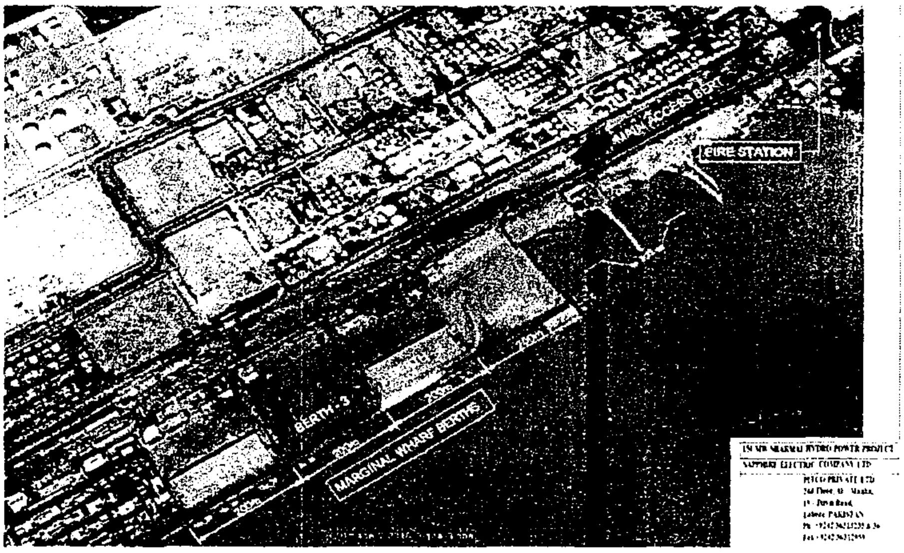


Figure 4-6: Berths for unloading equipment at Port Qasim

4.2.3.1.2 Segment II: Fire Station to PQA Custom Gate House

This section of route is single carriage way having two (02) lanes. Each lane is 3.5 meters. There are three (03) turnings on this section. The trailer carrying heavy equipment can maneuver easily along these turnings by doing minor earthwork to be assessed keeping in view the trailer.

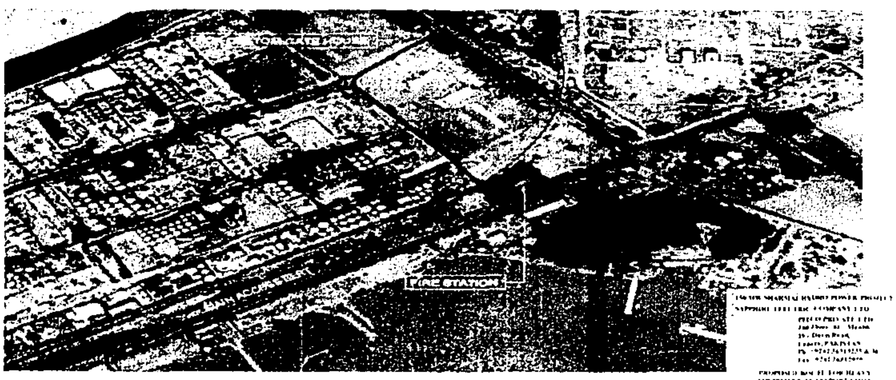


Figure 4-7: Carriage way from Fire Station to PQA Custom Gate House

4.2.3.1.3 Segment III: PQA Custom Gate to Pakistan Steel Mill Intersection

This section of route is a dual carriage way having three (03) lanes in each direction. Each lane is 3.75 meters wide. Along this section Pakistan Steel Mill conveyor is crossing the existing PQA road. The height of the conveyor is about 7.0 meters from the top of the road.

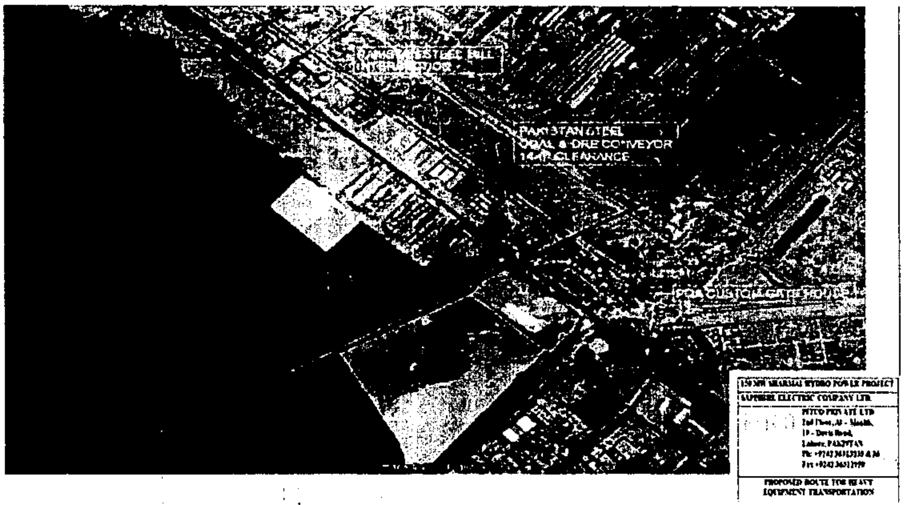


Figure 4-8: PQA Custom Gate to Pakistan Steel Mill Intersection

4.2.3.1.4 Segment IV: Pakistan Steel Mill intersection to FOTCO Installations

This section of route is a single carriage way having two (02) lanes of 3.75 meters wide and 3.0 meters paved shoulders on either side of carriage way. The High Tension (HT) lines are crossing near FOTCO installations. The H.T lines are significantly high. This section of route is dual carriage way having two (2) lanes of 3.75 meters wide in each direction. There is a RCC Culvert over the storm water drain.

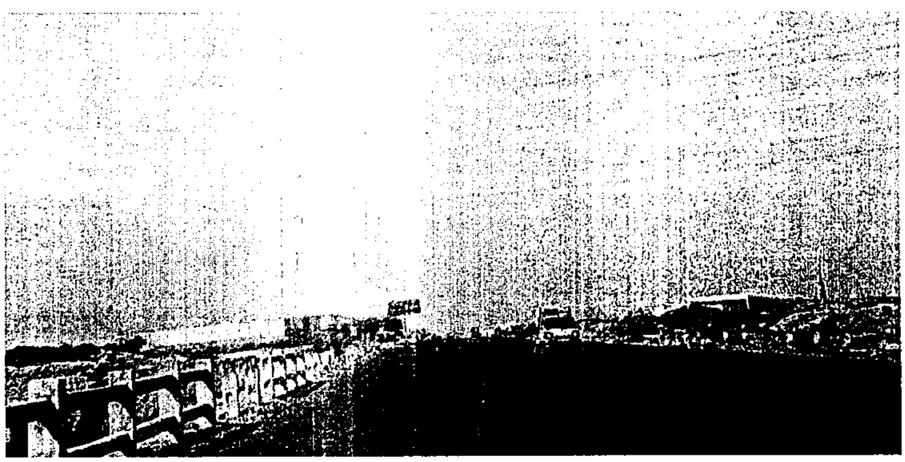


Figure 4-9: Culvert and H.T Lines crossing

4.2.3.1.5 Segment V: PQA North Western Industrial Zone Boundary to the Intersection of PQA road and National Highway

This section of route has single carriage way having two (02) lanes of 3.5 meters wide in each direction.

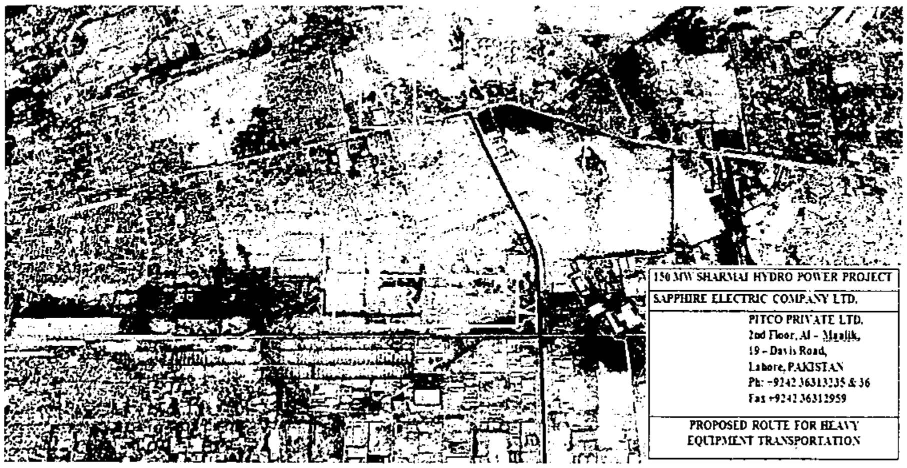


Figure 4-10: Intersection of PQA road and National Highway

4.2.3.1.6 Segment VI: Obstructions & Clearances

There are two (02) major obstructions along the proposed route up to Intersection of PQA road and National Highway.

Pakistan Steel Mill Conveyor Belt

Pakistan Steel Mill conveyor crosses the proposed route for the transportation of heavy equipment. The clearance available under the conveyor is about 7.0 meters and equipment proposed by M/s Fichtner can safely cross underneath the Pakistan Steel Mill conveyor belt structure considering the trailer bed height 1.2 meters and equipment height 5.0 meters.

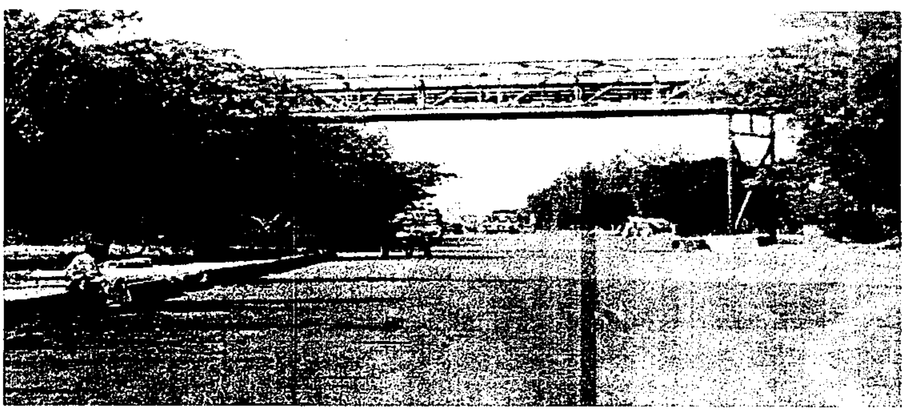


Figure 4-11: Pakistan steel mill conveyor belt crosses existing PQA road

Existing Culverts

The culverts and bridges are designed for class "A", loading as per West Pakistan code of practice for highway bridges 1967.

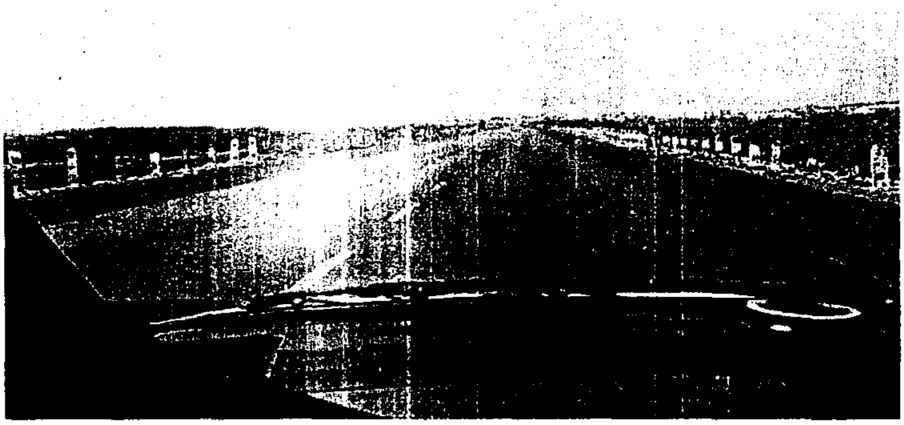


Figure 4-12: Culvert over existing nullah

4.2.3.2 SECTION 2: National Highway to Motorway up to Nowshehra (Rashkai)

National Highway and Motorways are commonly used by dumper trucks, gas bowser multiple-axels trailers without any problem for transportation of various equipment, materials, containers, etc. to and from the ports and oil tankers from various refineries in Pakistan.

As such, the requisite rout survey will be focused on the Section -3, i.e. following the route from Mardan Bypass to Dir-Chitral Road intended to be used for transportation of Sharmai Hydropower Project equipment to Power House Site at Darora and material transportation to Dam Site at Sharmai.

4.2.3.3 SECTION 3: Motorway Rashkai Interchange to Mardan Bypass to Dir Chitral Road

Along this section of the route 3 meter wide unpaved access is available. The topography of this section is undulated and natural nullah also exists in this section of route (Refer Figure 4-14).

Figure 4-13: Off-Motorway equipment transportation route

4.2.3.3.1 Segment I:

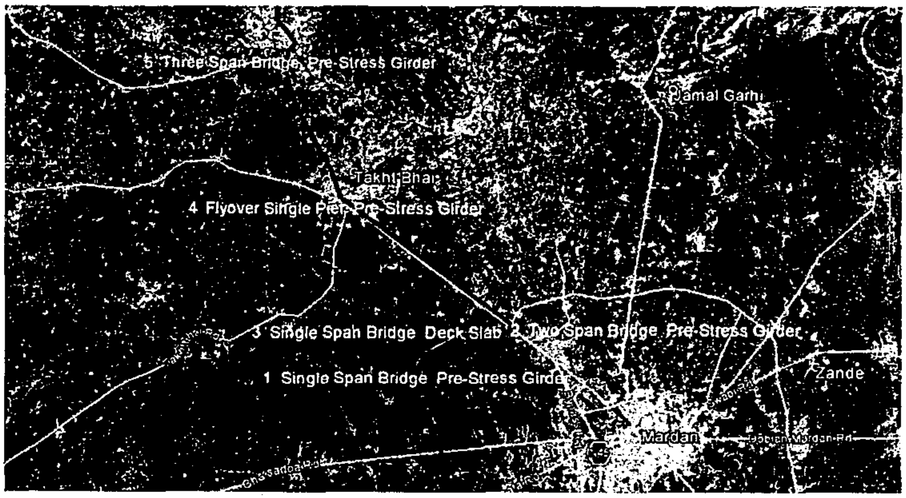


Figure 4-14: Part of section 3 transportation route

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
1	Two Lanes (25 Feet)	Single Span Bridge	Pre-Stress Girder	15	Very Good Condition
2	Two Lanes (25 Feet)	Two Spans Bridge	Pre-Stress Girder	30	Very Good Condition
3	Two Lanes (25 Feet)	Single Span Bridge	Deck Slab supported on Abutments	15	Very Good Condition
4	Two Lanes (25 Feet)	Two Spans Bridge	Pre-Stress Girder	20	Very Good Condition
5	Two Lanes (25 Feet)	Three Spans Bridge	Pre-Stress Girder	30	Very Good Condition

Table 4-2: Structure attributes of Segment I

Observations Summary:

The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

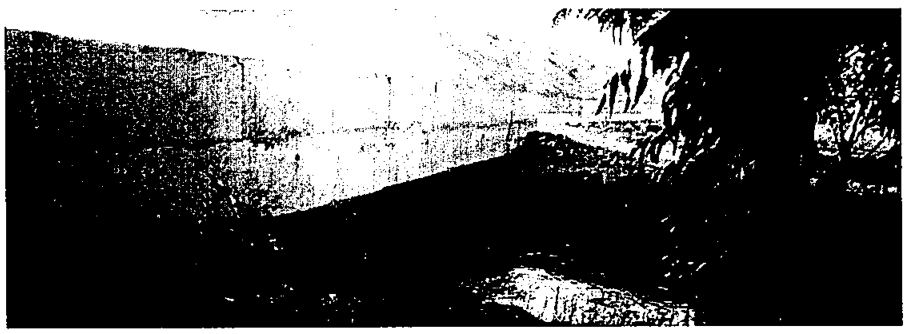


Figure 4-15: ID 3 Mardan Road

Figure 4-16: ID 2 Mardan By-pass

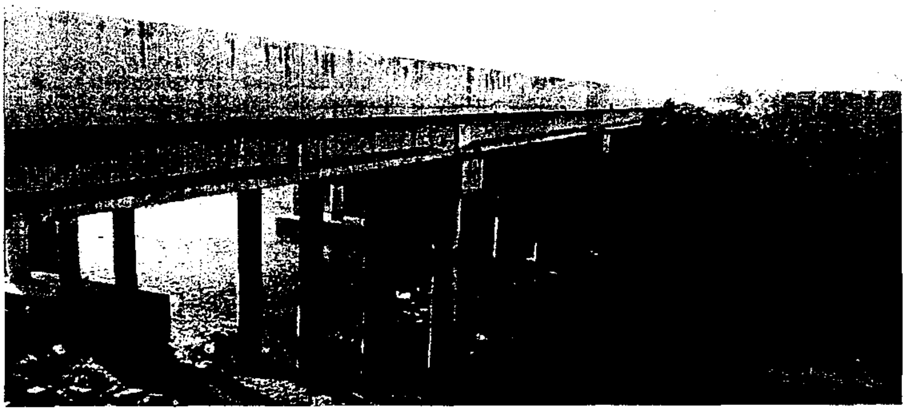


Figure 4-17: ID 5 Mardan Road

4.2.3.3.2 Segment II:

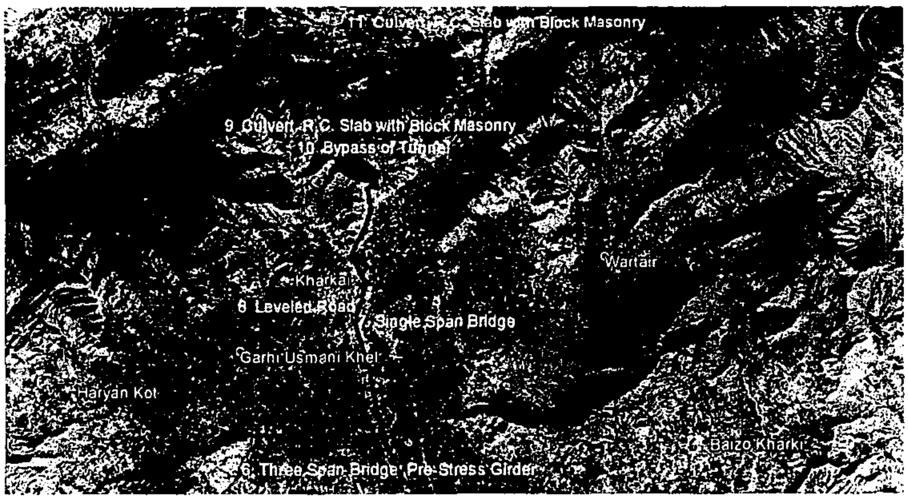


Figure 4-18: Part of section 3 transportation route

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
6	2+2 Two Lanes (one way)	Three Spans Bridge	Pre-Stress Girder	25	Very Good Condition
7	2+2 Two Lanes (one way)	Single Span Bridge	–	–	*Traffic Congestion
8	2+2 Two Lanes (one way)	Leveled Road	–	–	Check Post Dargai
9	2+2 Two Lanes (one way)	Culvert	R.C. Slab supported on Stones Masonry Wall	3	Very Good Condition
10	20'(Approx.)	Bypass of Tunnel	–	–	Very Good Condition

ID No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
11	Two Lanes (25 Feet)	Single Span Culvert	R.C. Slab supported on Stones Masonry Wall	2.5	Very Good Condition

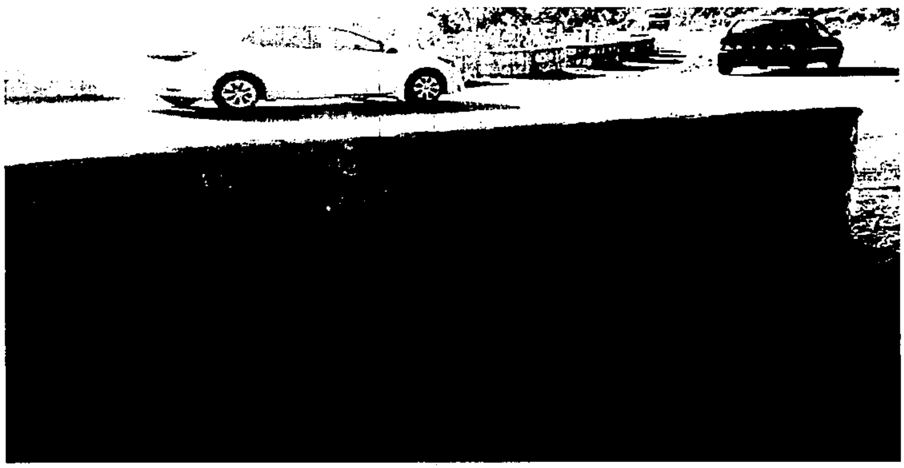


Figure 4-19: ID 9 ButtKhela Road

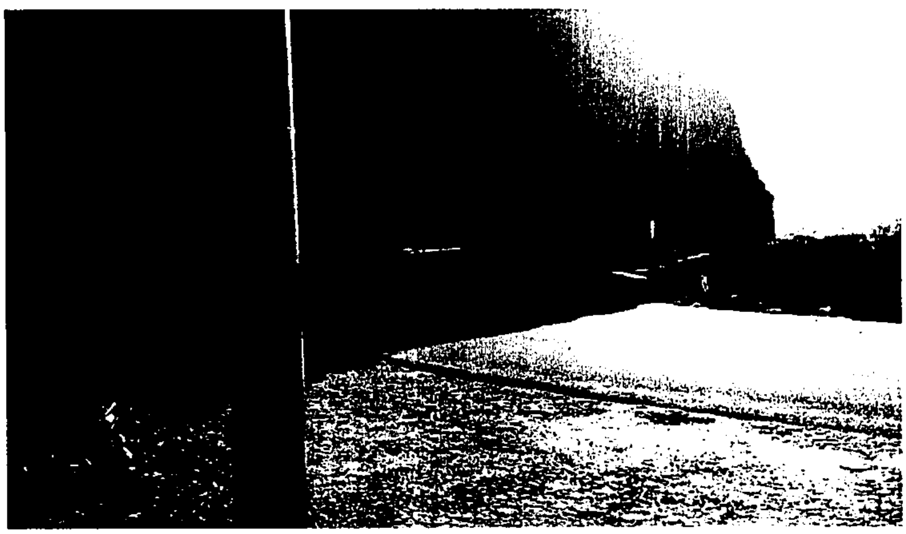


Figure 4-20: ID 10 Tunnel & its bypass at Buttkhela road

Observations Summary:

The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

4.2.3.3.3 Segment III:

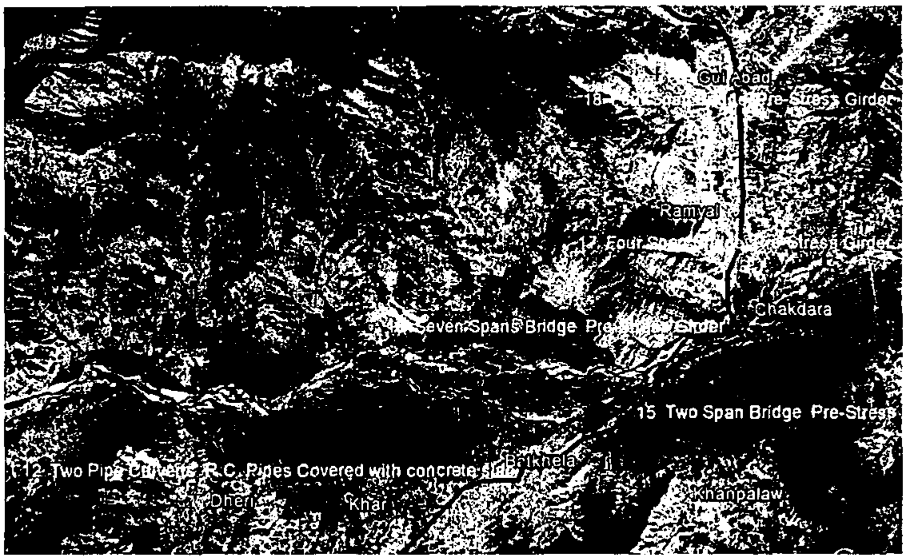


Figure 4-21: Part of section 3 transportation route

Table 4-3: Structure attributes of Segment III

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
12	Two Lanes (25 Feet)	Two Pipe Culverts	R.C. Pipes Covered with concrete slab	–	Very Good Condition
15	Two Lanes (25 Feet)	Two Spans Bridge	Pre-Stress Girder	28	Very Good Condition
16	2x25'(Two way)	Seven Spans Bridge	Pre-Stress Girder	28	Very Good Condition
17	Two Lanes (25 Feet)	Four Spans Bridge	Pre-Stress Girder	30	Very Good Condition
18	Two Lanes (25 Feet)	Four Spans Bridge	Pre-Stress Girder	30	Very Good Condition

Figure 4-22: ID 12 ButtKhela Road

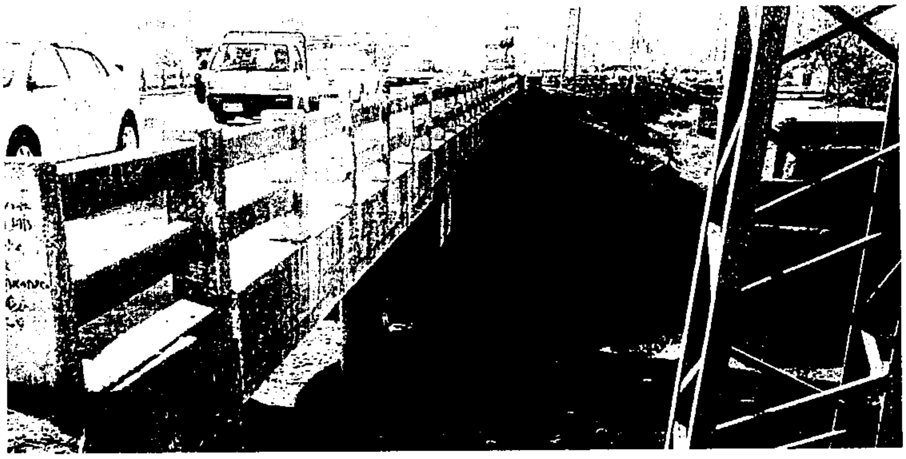


Figure 4-23: ID 15 ButtKhela Road

Observations Summary:
The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

4.2.3.3.4 Segment IV:

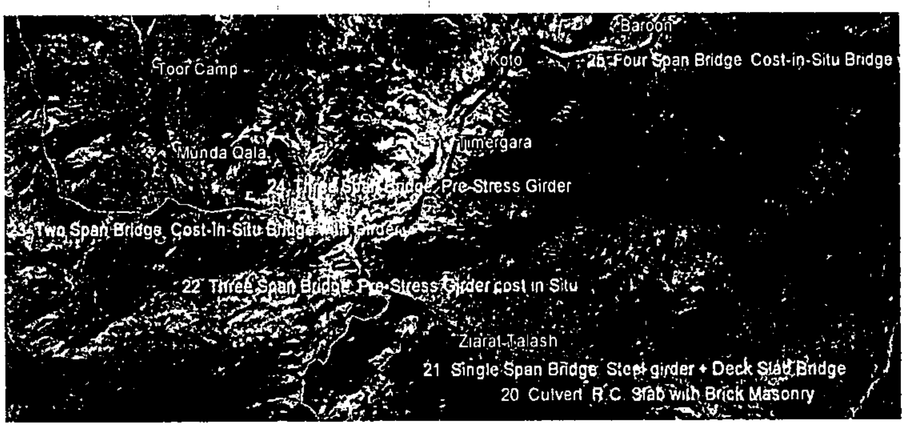


Figure 4-24: Part of section 3 transportation route

Table 4-4: Structure attributes of Segment IV

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
20	Two Lanes (25 Feet)	Culvert	R.C. Slab supported on Stones Masonry Wall	2.5	Very Good Condition
21	Two Lanes (25 Feet)	Single Span Bridge	Steel girder + Deck Slab Bridge	–	Very Good Condition
22	Two Lanes (25 Feet)	Three Spans Bridge	Pre-Stress Girder cast-in-situ	–	Very Good Condition

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
23	Two Lanes (25 Feet)	Two Spans Bridge	Cast-in-Situ RC Deck Slab with Girder	15	Very Good Condition
24	Two Lanes (25 Feet)	Three Spans Bridge	Pre-Stress Girder	30	Very Good Condition
25	Two Lanes (25 Feet)	Four Spans Bridge	Cast-in-Situ RC Deck Slab with Girder	15	Good Condition

Observations Summary:

The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

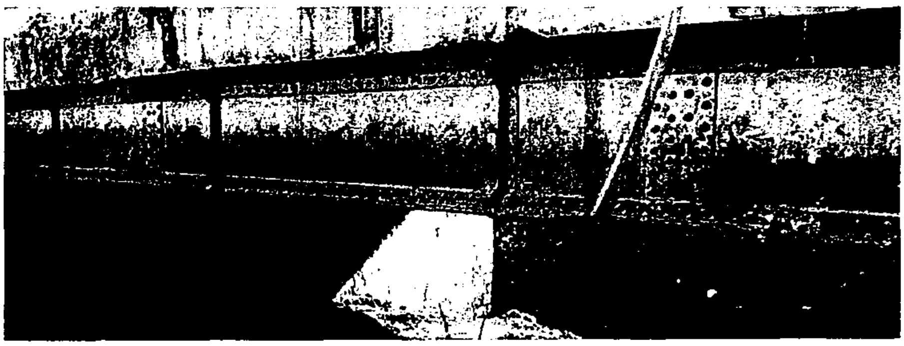


Figure 4-25: ID 21 Dir Chitral Road

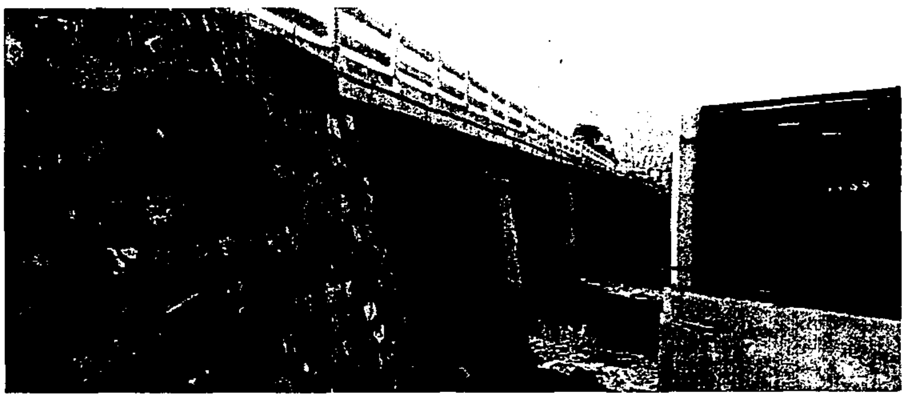


Figure 4-26: ID 25 Dir Chitral Road

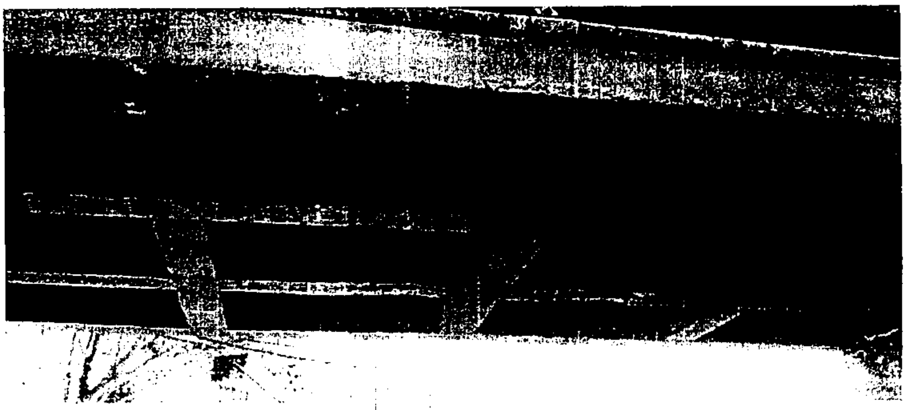


Figure 4-27: ID 25 Dir Chitral Road

4.2.3.3.5 Segment V:

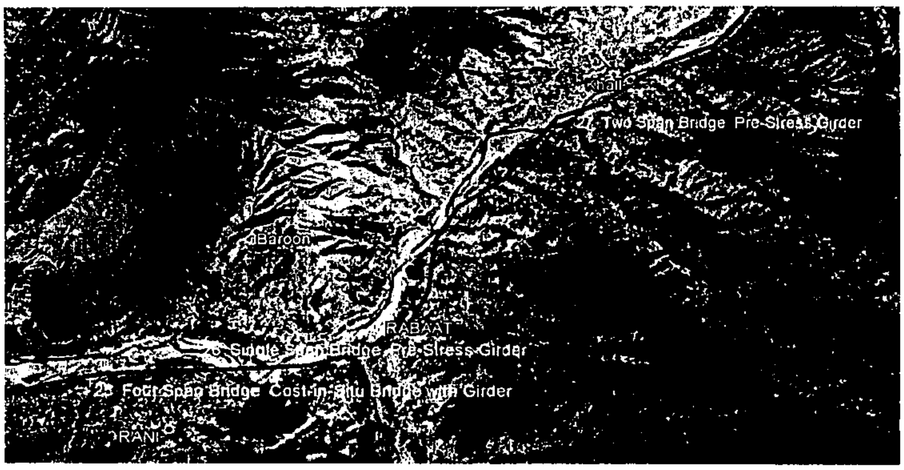


Figure 4-28: Part of section 3 transportation route

Table 4-5: Structure attributes of Segment V

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
26	Two Lanes (25 Feet)	Single Span Bridge	Pre-Stress Girder	50	Very Good Condition
27	Two Lanes (25 Feet)	Two Spans Bridge	Pre-Stress Girder	15	Very Good Condition

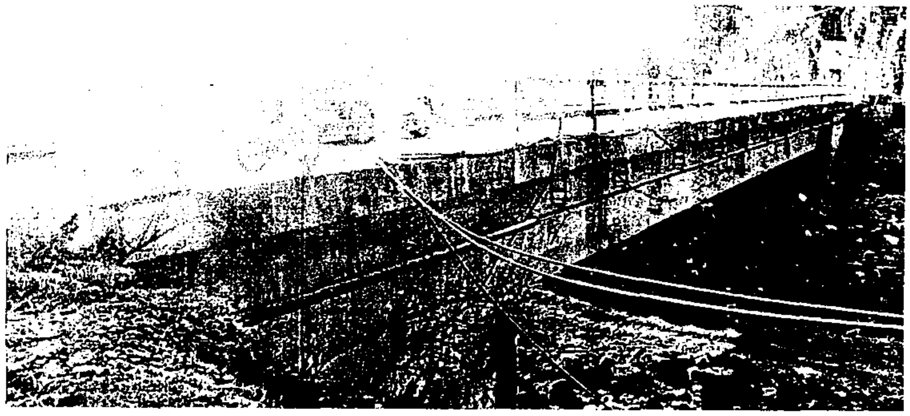


Figure 4-29: ID 26 Dir Chitral Road

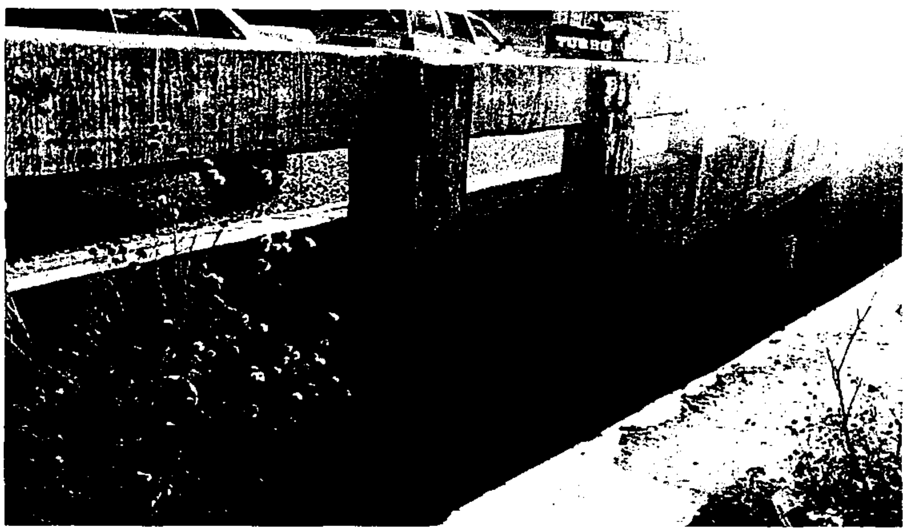


Figure 4-30: ID 27 Dir Chitral Road

Observations Summary:
The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

4.2.3.3.6 Segment VI:

Figure 4-31: Part of section-3 transportation route

Table 4-6: Structure attributes of Segment VI

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
28	Two Lanes (25 Feet)	Two Spans Bridge	Pre-Stress Girder	20	Very Good Condition
29	Two Lanes (25 Feet)	Two Spans Culvert	R.C. Slab supported on Stones Masonry Wall	3	Good Condition
30	Two Lanes (25 Feet)	Single Span Bridge	Pre-Stress Girder	30	Good Condition

Observations Summary:

The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

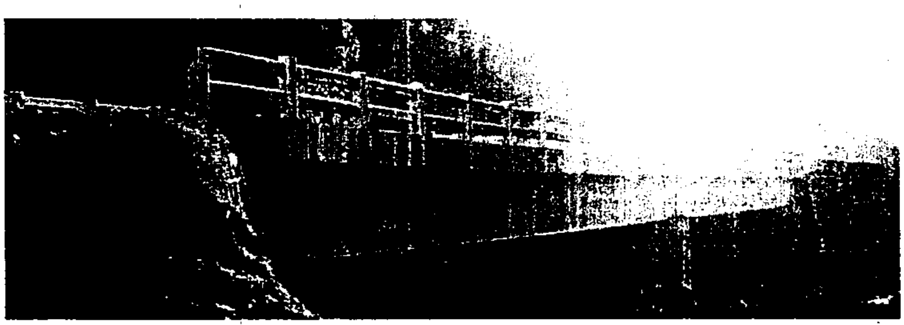


Figure 4-32: ID 28 Dir Chitral Road

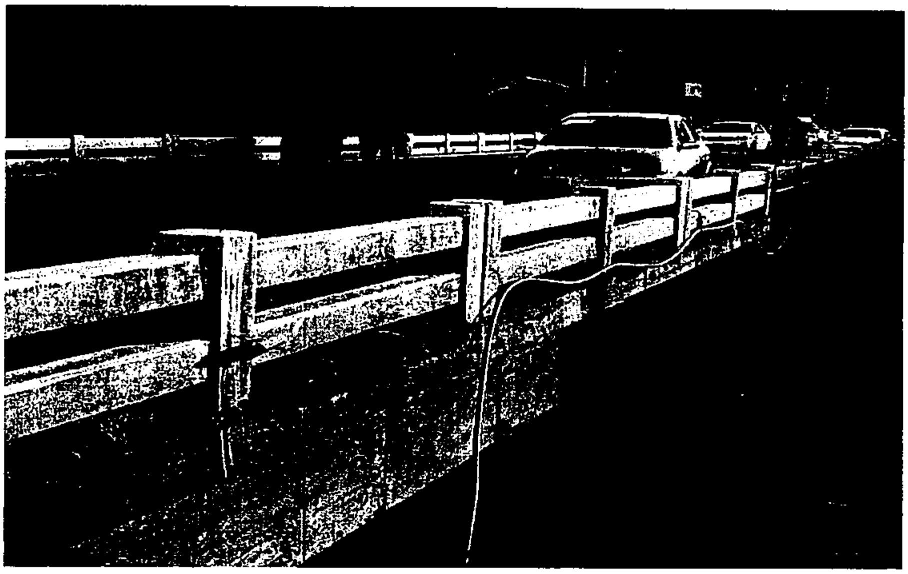


Figure 4-33: ID 30 Dir Chitral Road

4.2.3.3.7 Segment VII:

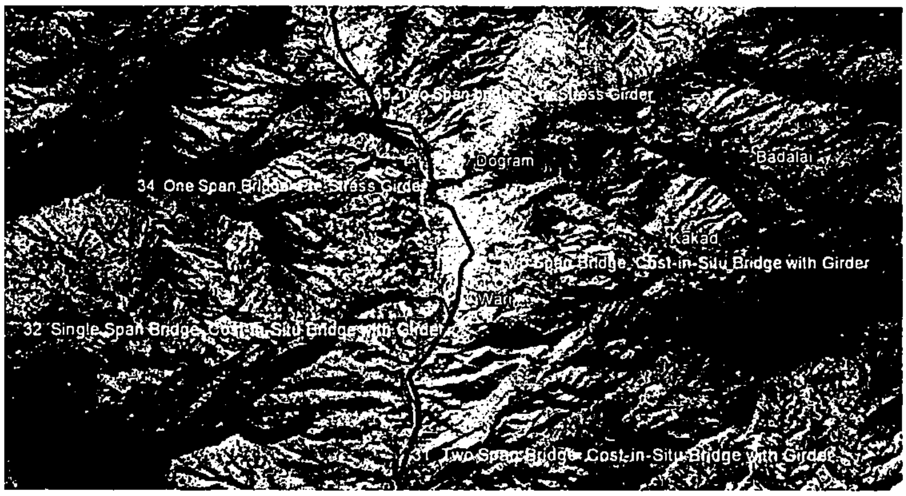


Figure 4-34: Part of section-3 transportation route

Table 4-7: Structure attributes of Segment VII

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
31	Two Lanes (25 Feet)	Two Spans Bridge	Cast-in-Situ RC Deck Slab with Girder	13	Good Condition
32	Two Lanes (25 Feet)	Single Span Bridge	Cast-in-Situ RC Deck Slab with Girder	15	Good Condition
33	Two Lanes (25 Feet)	Two Spans Bridge	Cast-in-Situ RC Deck Slab with Girder	15	Very Good Condition
34	Two Lanes (25 Feet)	One Span Bridge	Pre-Stress Girder	15	Very Good Condition

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
35	Two Lanes (25 Feet)	Two Spans Bridge	Pre-Stress Girder	15	Very Good Condition

Observations Summary:

The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

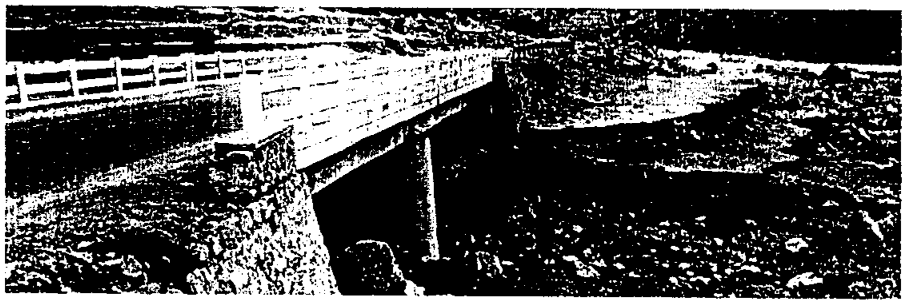


Figure 4-35: ID 31 Dir Chitral Road

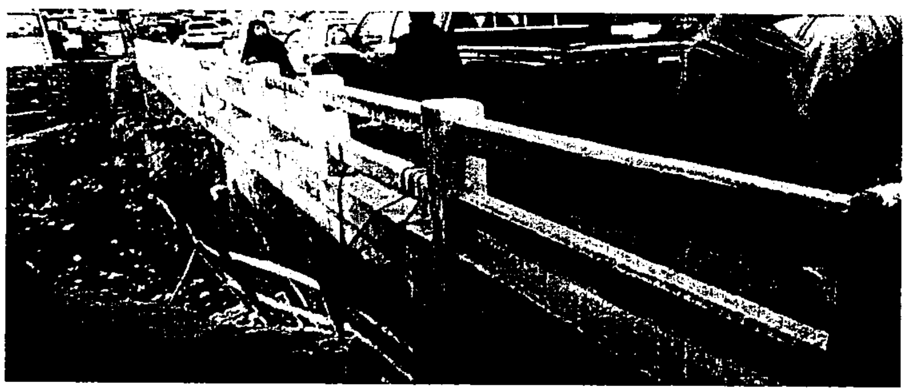


Figure 4-36: ID 33 Dir Chitral Road

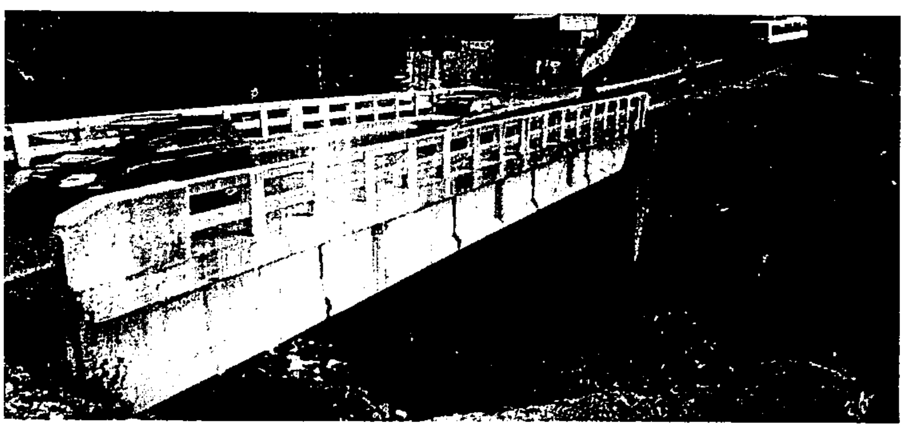


Figure 4-37: ID 34 Dir Chitral Road

4.2.3.3.8 Segment VIII:

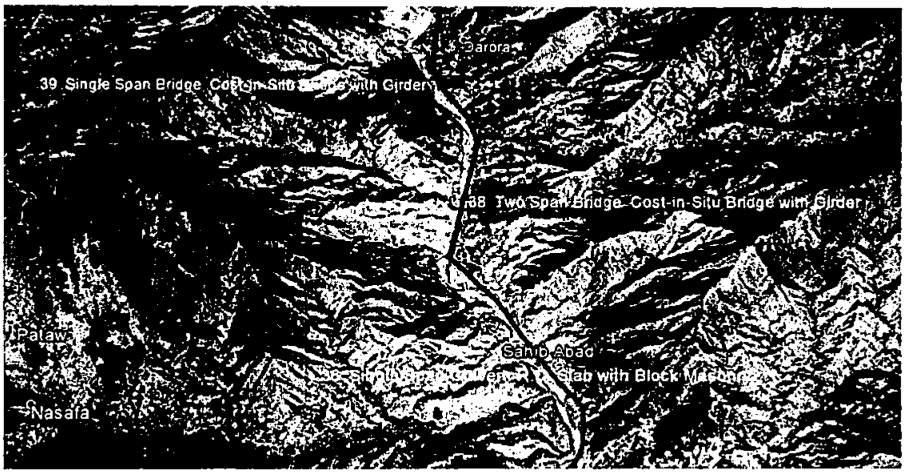


Figure 4-38: Part of section-3 transportation route

Table 4-8: Structure attributes of Segment VIII

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
36	Two Lanes (25 Feet)	Single Span Culvert	R.C. Slab supported on Stones Masonry Wall	6	Very Good Condition
38	Two Lanes (25 Feet)	Two Spans Bridge	Cast-in-Situ RC Deck Slab with Girder	13	Very Good Condition
39	Two Lanes (25 Feet)	Single Span Bridge	Cast-in-Situ RC Deck Slab with Girder	13	Very Good Condition

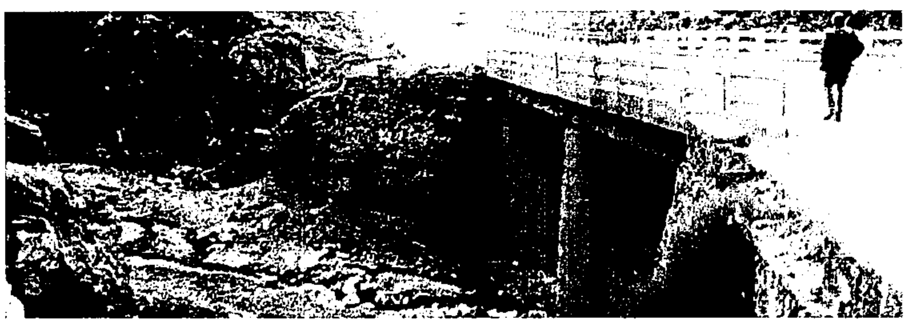


Figure 4-39: ID 38 Dir Chitral Road

Observations Summary:

The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

4.2.3.3.9 Segment IX:

Figure 4-40: Part of section-3 transportation route

Table 4-9: Structure attributes of Segment IX

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
63	Two Lanes (25 Feet)	Power House Site	–	–	Power House Location
64	Two Lanes (25 Feet)	Single Span Bridge	Pre-Stress Girder	30	Very Good Condition
65	Two Lanes (25 Feet)	Single Span Bridge	Cast-in-Situ RC Deck Slab with Girder	10	Very Good Condition
66	Two Lanes (25 Feet)	Single Span Bridge	Cast-in-Situ RC Deck Slab with Girder	10	Very Good Condition

Figure 4-41: ID 63 Powerhouse Location

Observations Summary:

The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

4.2.3.3.10 Segment X:

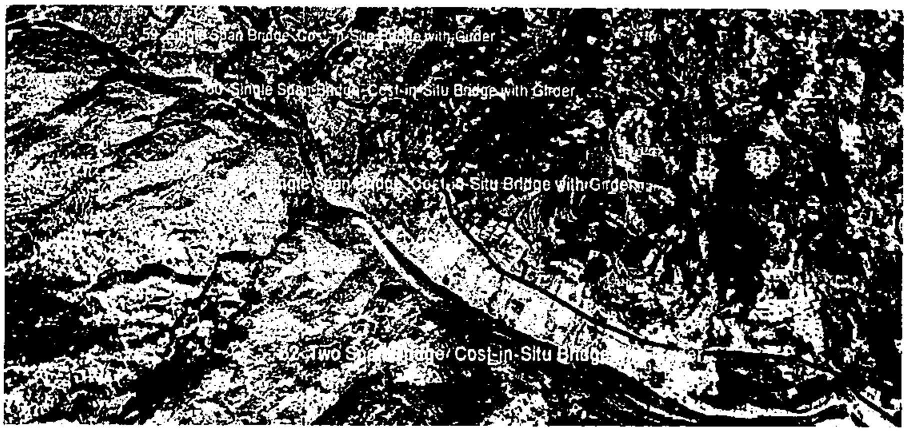


Figure 4-42: Part of section-3 transportation route

Table 4-10: Structure attributes of Segment X

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
59	Two Lanes (25 Feet)	Single Span Bridge	Cast-in-Situ RC Deck Slab with Girder	10	Very Good Condition
60	Two Lanes (25 Feet)	Single Span Bridge	Cast-in-Situ RC Deck Slab with Girder	13	Very Good Condition
61	Two Lanes (25 Feet)	Single Span Bridge	Cast-in-Situ RC Deck Slab with Girder	13	Very Good Condition
62	Two Lanes (25 Feet)	Two Spans Bridge	Cast-in-Situ RC Deck Slab with Girder	15	Very Good Condition

Observations Summary:

The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

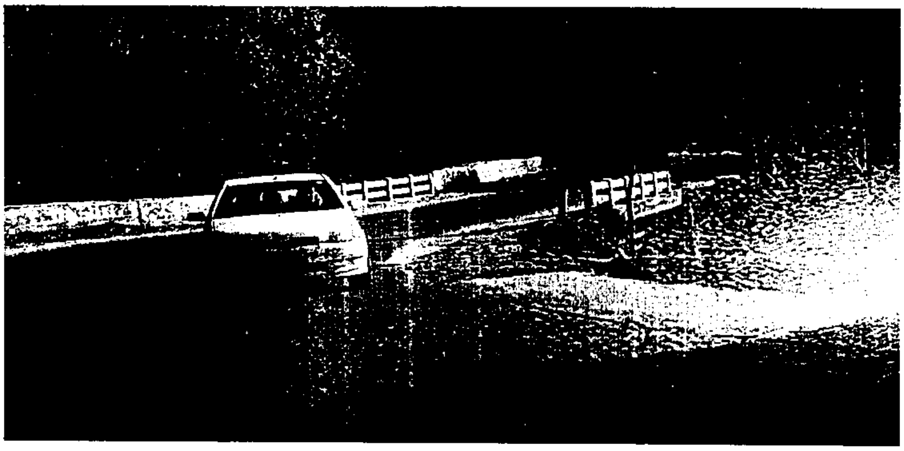


Figure 4-43: ID 59 Dir Chitral Road

Figure 4-44: ID 60 Dir Chitral Road

Figure 4-45: ID 62 Dir Chitral Road

4.2.3.3.11 Segment XI:

Figure 4-46: Part of section-3 transportation route

Table 4-11: Structure attributes of Segment XI

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
57	Two Lanes (25 Feet)	Single Span Culvert	R.C. Slab supported on Stones Masonry Wall	2	Very Good Condition
58	Two Lanes (25 Feet)	Single Span Bridge	Cast-in-Situ RC Deck Slab with Girder	13	Very Good Condition
37	Two Lanes (25 Feet)	Single Span Bridge	Cast-in-Situ with Girder	13	Very Good Condition

Figure 4-47 ID 57 Dir Chitral Road

Observations Summary:

The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

4.2.3.3.12 Segment XII:

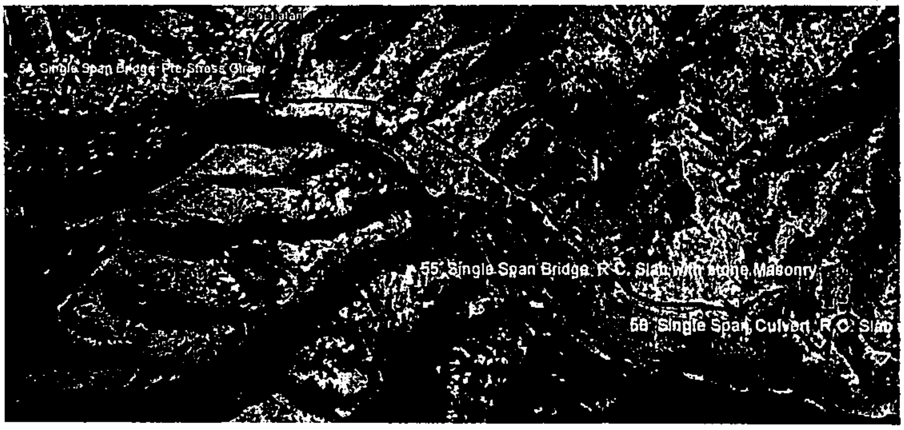


Figure 4-48: Part of section-3 transportation route

Table 4-12: Structure attributes of Segment XII

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
54	Two Lanes (25 Feet)	Single Span Bridge	Pre-Stress Girder	35	Very Good Condition
55	Two Lanes (25 Feet)	Single Span Bridge	R.C. Slab supported on Stones Masonry Wall	2	Very Good Condition
56	Two Lanes (25 Feet)	Single Span Culvert	R.C. Slab supported on Stones Masonry Wall	2	Very Good Condition

Observations Summary:

The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

4.2.3.3.13 Segment XIII:

Figure 4-49: Part of section-3 transportation route

Table 4-13: Structure attributes of Segment XIII

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
40	Two Lanes (25 Feet)	Single Span Bridge	Pre-Stress Girder	30	Very Good Condition
41	Two Lanes (25 Feet)	Single Span Bridge	R.C. Slab supported on Stones Masonry Wall	3	Very Good Condition
42	Two Lanes (25 Feet)	Single Span Bridge	R.C. Slab supported on Stones Masonry Wall	2	Very Good Condition
43	Two Lanes (25 Feet)	Three Spans Bridge	Pre-Stress Girder	30	Very Good Condition
44	Two Lanes (25 Feet)	Single Span Culvert	Cast-in-Situ RC Deck Slab with Girder	20	Very Good Condition

Observations Summary:
The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

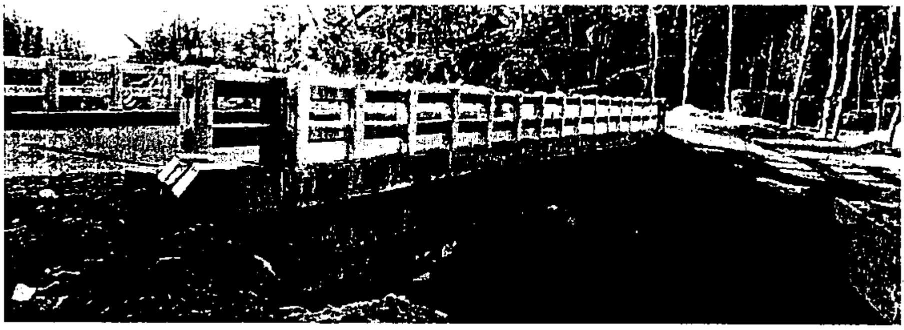


Figure 4-50: ID 40 Dir to Sharmai Road

Figure 4-51: ID 43 Dir to Sharmai Road

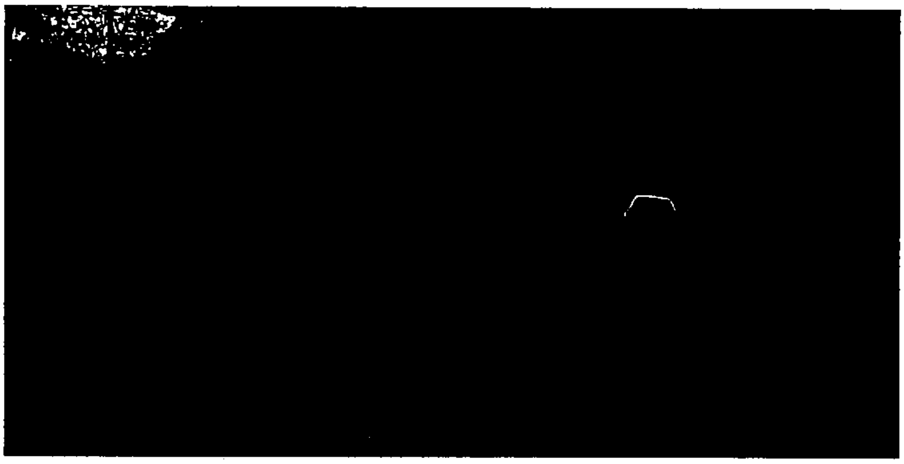


Figure 4-52: ID 44 Dir to Sharmai Road

4.2.3.3.14 Segment XIV:

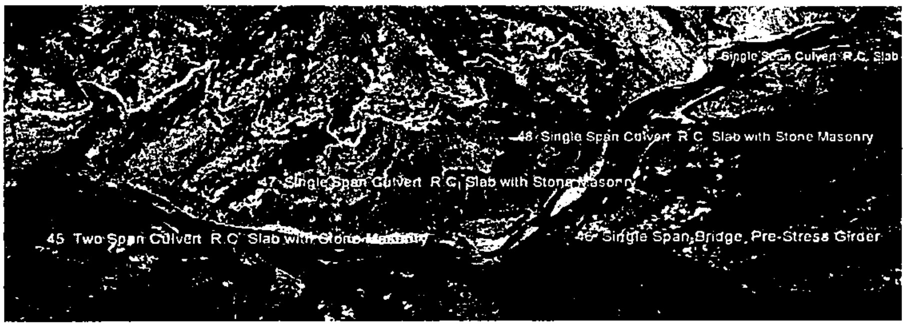


Figure 4-53: Part of section-3 transportation route

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
45	Two Lanes (25 Feet)	Two Spans Culvert	R.C. Slab supported on Stones Masonry Wall	3	Very Good Condition
46	Two Lanes (25 Feet)	Single Span Bridge	Pre-Stress Girder	20	Very Good Condition
47	Two Lanes (25 Feet)	Single Span Culvert	R.C. Slab supported on Stones Masonry Wall	2	Very Good Condition
48	Two Lanes (25 Feet)	Single Span Culvert	R.C. Slab supported on Stones Masonry Wall	2	Very Good Condition
49	Two Lanes (25 Feet)	Single Span Culvert	R.C. Slab supported on Stones Masonry Wall	3	Very Good Condition

Table 4-14: Structure attributes of Segment XIV

Observations Summary:
The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

Figure 4-54: ID 48 Dir To Sharmai Road

4.2.3.3.15 Segment XV:

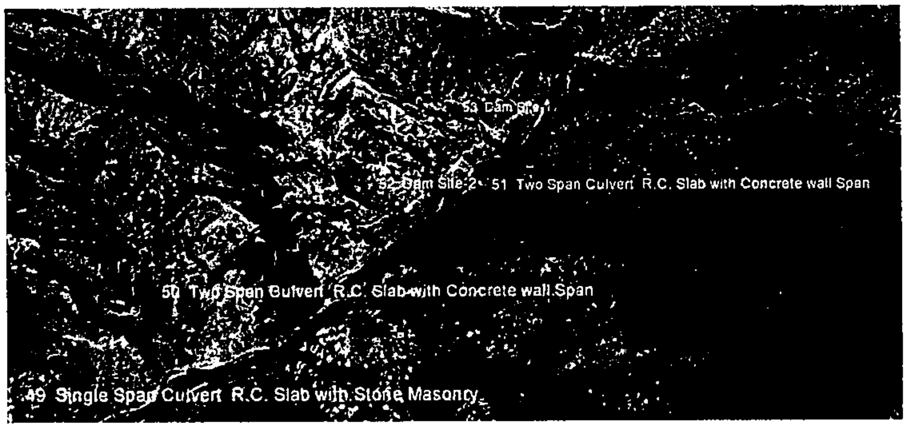


Figure 4-55: Part of section-3 transportation route

Table 4-15: Structure attributes of Segment XV

ID. No.	Road Width	Structure	Type of Structure	Span (meters)	Remarks
50	Two Lanes (25 Feet)	Two Spans Culvert	R.C. Slab supported on Concrete Wall	3	Very Good Condition
51	Two Lanes (25 Feet)	Two Spans Culvert	R.C. Slab supported on Concrete Wall	3	Very Good Condition
52	Two Lanes (25 Feet)	Dam Site-2	–	–	Asphalted Road.
53	Two Lanes (25 Feet)	Dam Site-1	–	–	Road yet to be asphalted

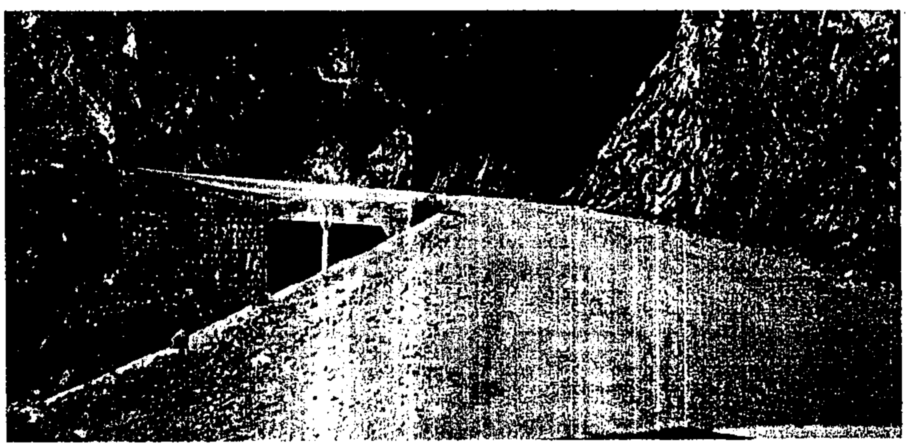


Figure 4-56: ID 50 Dir to Sharmai Road

Figure 4-57: ID 51 Dir To Sharmai Road

Observations Summary:
The road is carpeted and its width is uniform. Most of the bridges are newly constructed and are in very good condition. There are no bottle necks to cause hindrance in smooth traffic of trailers to be used for transportation of plant equipment for Sharmai Hydro Power Project.

4.2.4 Physical limitations and boundaries

4.2.4.1 Obstructions and clearances

The route is a dual carriage way having three (03) lanes in each direction. Each lane is 3.75 meters wide. Along this section Pakistan Steel Mill's conveyor is crossing the existing PQA road. The height of the conveyor is about 7.0 meters from the top of the road.

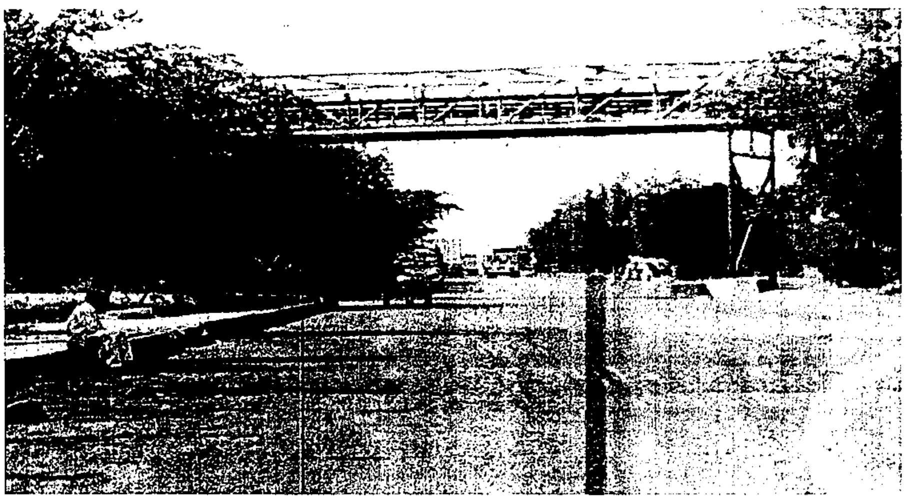


Figure 4-58: Pakistan steel mill conveyor belt crosses existing PQA road

There are three (03) culverts located along the proposed route. The structural capacity of the culvert is 12 tons per axle 2.5 meters apart. The distribution of axle load will be required to cross the existing culverts.

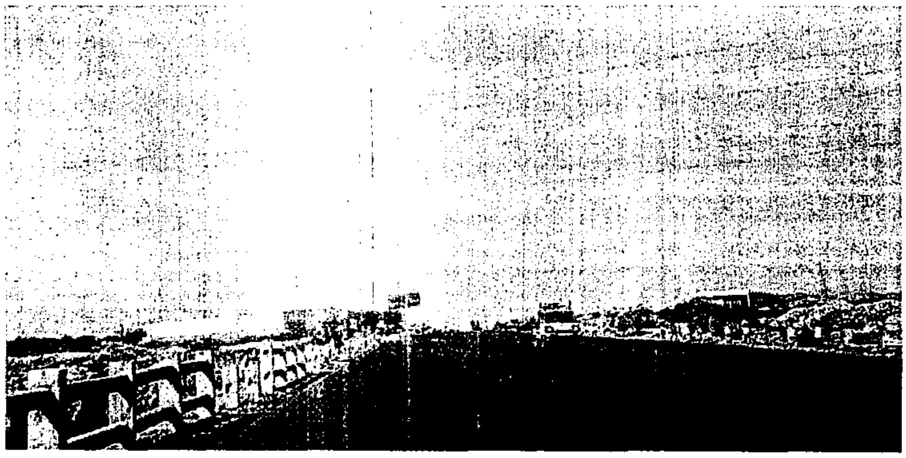


Figure 4-59: Culvert and H.T Lines crossing

Traffic Congestion at Tamergarah and Wari Main Bazar is obvious during day time rush hours that may be avoided by proper scheduling for smooth movement of trailers loaded with heavy equipment.

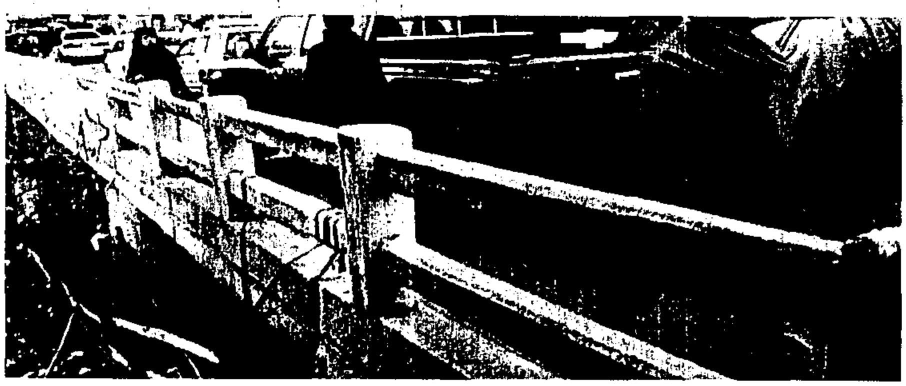


Figure 4-60: Traffic Congestion at Wari Main Bazar

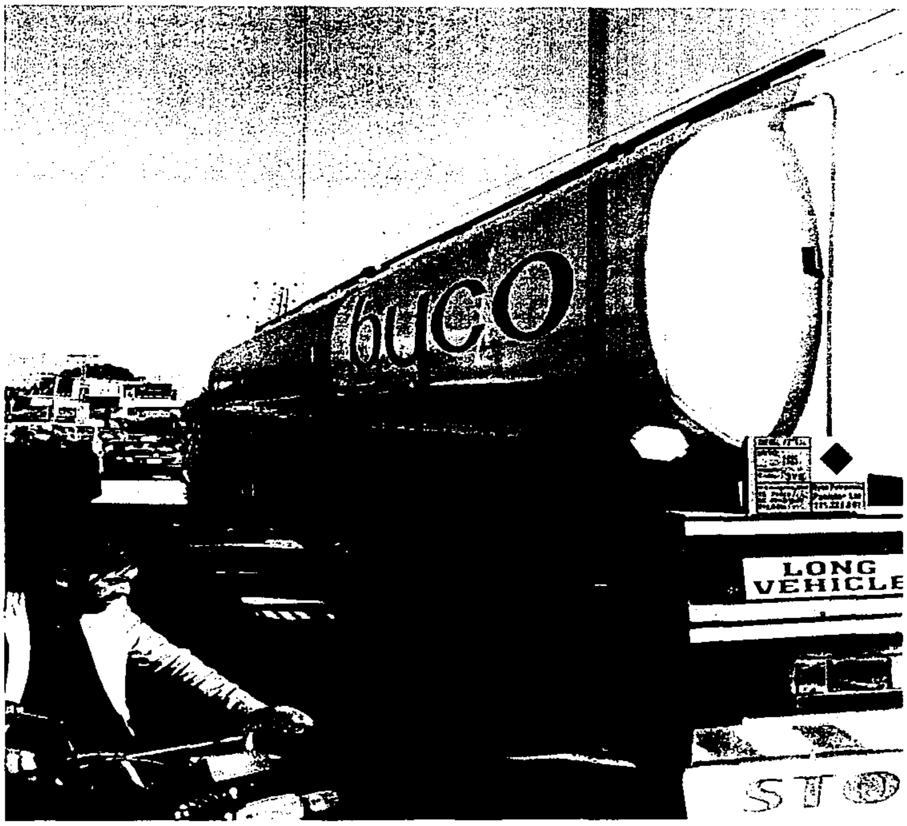


Figure 4-61: Traffic Congestion at Chakdara

4.2.4.2 Loading capability of existing culverts

The culverts and bridges are designed for class "A", loading as per West Pakistan code of practice for highway bridges 1967. The distribution of axle load will be required to cross the existing culverts.

4.2.4.3 Loading capability of existing road network

Road structure generally consists of 300 mm sub-base course, 200 mm base course and 100 mm bituminous concrete. Loading Capability of existing PQA main access road is adequate for one time movement of equipment load as provided by M/s Fichtner.

4.2.5 Site infrastructure

4.2.5.1 Telecommunication options

The Project site is remote and therefore telecommunication signals are weak especially at DAM, reservoir and surge tank sites. Since the road is under construction at these sites it is anticipated that once the road is fully operational the different Mobile communication service providers might consider the network augmentation along the major road which could result in more reliable voice communication. The telecommunication signals at Powerhouse sites are slightly better.

The nearest telephone exchange from the project site at Dir City which is approx. 8 km from project site. The voice and data line from exchange to project site would require some cost to either deploy microwave link or optical fibre to enhance the telecommunication services at project site. For better telecommunication a booster tower may be arranged by coordinating with some Telecom. Enterprise.

4.2.5.2 Water resources

There are water sources with favorable quality in all construction sites which can be directly utilized after appropriate treatment.

4.2.5.3 Railway transportation option

For power plant equipment transportation, railway option is neither efficient nor commonly used in Pakistan as it involves multiple equipment handling efforts viz. loading on trailer at the port, off-loading from trailer and loading on railway wagon at railway station nearest to the port, then off-loading from railway wagon at railway station nearest to the Project Site and loading on trailer for highway transportation and finally off-loading at the Project Site. Whereas power plant equipment transportation by highways / road option involves once loading on trailer at the port and off-loading at the Project Site which is more convenient, requires less time & efforts and commonly used practice in Pakistan.

Conclusively, option for power plant equipment transportation by railway is not recommended.

4.2.5.4 Site visit photographs

Figure 4-62: ID 53 Demarcation of Dam Site 1

Figure 4-63: ID 52 Demarcation of Dam Site 2

4.2.6 Conclusion

Having 'Heavy Equipment Transportation Route' physically surveyed, it is concluded that by avoiding timing of traffic congestion at Chakdara, Tamergarah and Wari Main Bazar; there will be no difficulty for loaded trailers for transportation of heavy equipment from Bin Qasim Port to Sharmai Hydropower Project Site at Darora.

5. Geological Interpretive Report

5.1 Introduction

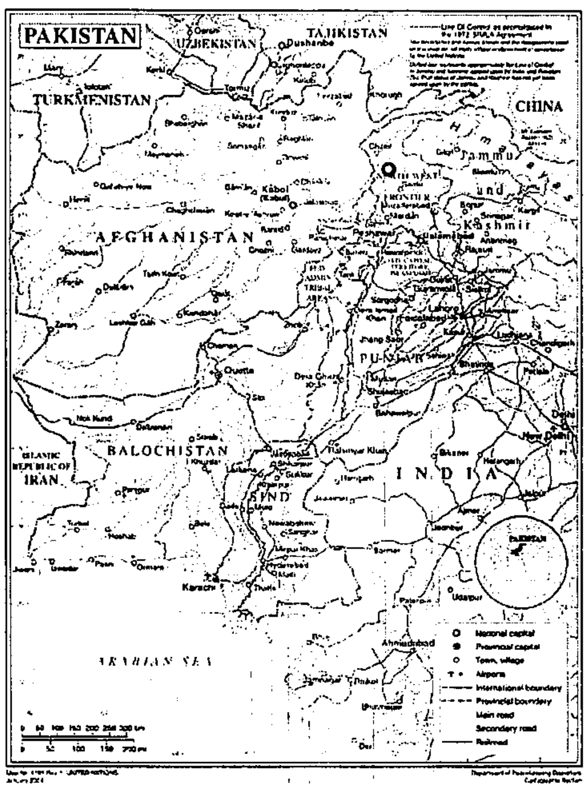
5.1.1 General

SAPPHIRE Electric Company Ltd. plans to construct SHARMAI Hydro Power Plant in northern Pakistan, Khyber Pakhtunkhwa Province about 248 km from the city of Peshawar. The Project site is located upstream in the Panjkora River, one of the tributaries of the Indus River in Khyber Pakhtunkhwa. Moreover, the Panjkora River is one of the largest rivers in the province and drains an area of about 1,900km2, which is slightly more than 40% of the whole Dir district.

The geographical coordinates of the intake and powerhouse are as follows in Table 5-1:

Table 5-1: Coordinates of the site

	Longitude	Latitude
Intake (Dam site)	71°56'33.37"	35°10'26.28"
Powerhouse	71°59'13.21".	35°6'23.10"



Project location

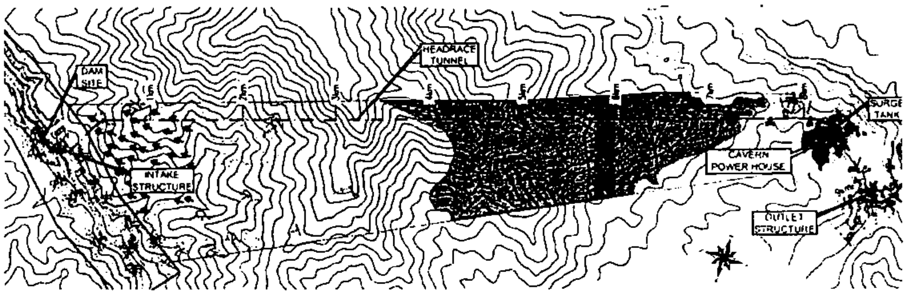


Figure 5-1: Location of SHPP

The general location of the SHPP site is shown in the figure above.

Within the scope of the feasibility study, geological, geotechnical and geophysical investigations were carried out, which are reflected in the present "Geotechnical Interpretative Report".

This report presents the interpretation of the findings of the Geological, Geotechnical and Geophysical investigations that were conducted between 12.2017 and 06.2018.

Following Annexes forms the integral part of this report:

Annex 01	Geological Surface mapping
Annex 02	Factual report
Annex 03	Seismic refraction report
Annex 04	Lab report on quarry sites
Annex 05	Seismic Hazard analysis

5.1.2 Salient features of the Project

The total capacity of the proposed Sharmai Hydro Power Plant (SHPP) is 150 MW. The SHPP comprises of an Intake (Dam site), which is located about 2 km upstream from Chutiyatan along Dir- Sheringal Road on Panjkora river. The power house is located on the main GT road N-45 near Darora village on the confluence of Usherai Khwar and Panjkora River.

The followings are the salient features of the project

- Sharmai reservoir with a capacity of about 32.2 million m³, formed by a 45m high retention concrete dam
- a diversion tunnel at the right abutment
- a gated spillway at the right abutment
- stilling basin structure at the Panjkora riverbed
- 7.8 km - Pressure headrace tunnel with a surge shaft, pressure shaft and an underground powerhouse cavern
- underground powerhouse, housing 3 vertical Francis turbines
- underground Tailrace tunnel
- outdoor switchyard

- 220 kV double circuit transmission line, connecting the power plant to the Chakdara Substation;
- access roads to the plant and the site infrastructure.

5.1.3 Reference system and map projection

The following reference system was specified to create the topographic planning data and implement the project:

- UNIVERSAL TRANSVERSE MERCARTOR (UTM)
- ZONE 42 NORTH
- GCS WGS 1984/ DATUM WGS 1984
- SPHEROID WGS 1984
- FALSE EASTING 500000
- FALSE NORTHING 0
- CENTRAL MERIDIAN -69/SCALE 0,9996
- Authority: 32642 (EPSG)
- The reference system for elevation: EGM 96.

Vertical:

- Orthometric height (EGM 2008/ WGS84).

5.1.4 Objective

The objective of the Geotechnical Interpretative Report is to provide basic information about

- Regional/ Local Geological Situation
- Regional/ Local tectonics and Seismicity
- Geological Mapping
- Geotechnical Characteristics of the Sites
- Construction Materials and Quarries.

Based on the information on the studies carried out prior to this, several potential sites for dam and power stations were investigated and the present location of the SHPP was considered to be the most favorable site in terms of topographical and geological conditions.

Nevertheless, some uncertainties/constraints exist, and general potential risk issues shall be taken into account during detailed design, scheduling and construction as summarized below.

- presence of unknown associated faults or shear zones that will affect excavations or have influence on groundwater flow and support of the tunnels
- zones with extraordinary deep weathering being weak zones in the foundation and tunnels and requiring special measures

- presence of large boulders as overburden that affect the excavation of superficial materials like e.g. alluvium, colluvium etc.
- due to presence of faults, sheared zones or crushed zones, the quality of the rock will be worse than actually assumed
- crushed or sheared zones are expected to be associated to high rock mass permeability
- irregular surface of the riverbed with boulders and other coarse material can create seepage under the cofferdams.

The said risks related to the geological conditions at site are considered manageable, with the possible cost implications deemed accounted for in the contingency amount.

5.2 Site Investigations

5.2.1 Geological data

The following information were obtained from the Geological department of Pakistan and was used for the study purpose
[1] Geological map of the NW Himalaya, Scale 1:1000000
[2] Geological map of Pakistan, Scale 1:10000000
[3] Geological map of DIR Quadrangle, Scale 1:50,000 Ministry of Petroleum and Natural resources

5.2.2 Field investigations

In the framework of the Feasibility Study, several geological, geotechnical and geophysical investigation works and studies have been carried out. These are, amongst other:

- Core-drillings (18 straight rotary drillings in overburden and NQ size thin walled double tube core barrel in rock varying between 22 m and 250 m)
- Test pits (14 pits - 0.8 m to 3 m deep)
- Permeability tests (58 Lugeon tests at different boreholes/ depths)
- Seismic refraction Survey (10 lines of 2,465 meters in total)
- Standard penetration test (SPTs) in boreholes (2 in total)
- Plate load test (1 in total)
- Installation of piezometers (6 in total)
- Geological mapping:
- Incorporation of Campaign results in an updated geological map
- Seismic hazard assessment study.

In Figure 5-2 the location of the drill holes and test pits has been shown:

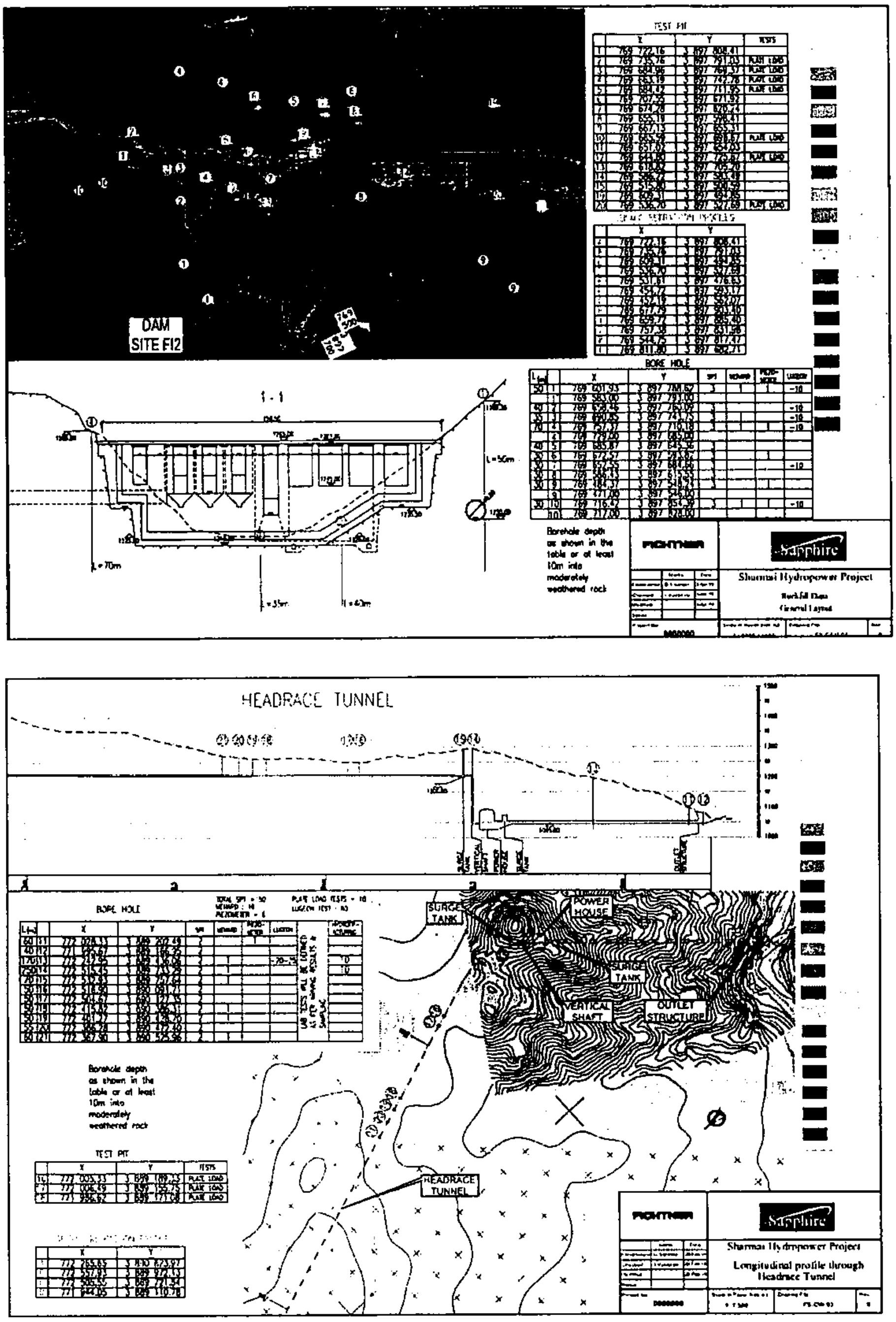


Figure 5-2: Location of drill locations and test pits

5.2.3 Laboratory investigations

The samples obtained from geological surface mapping task, geological drilling works, test pit excavations and from quarry sites were sent to the geotechnical laboratory for further investigation. Different mechanical, physical and chemical tests were undertaken to determine the properties of encountered rock and soil at site.

The investigation works were carried out as per existing international norms. The details to the investigation and the results has been include in the factual report.

The physical and mechanical investigation include sieve analysis, moisture content, density, point load test, unconfined compression tests, hook triaxial tests, slake durability tests, point load tests and Los Angeles abrasion value test. Likewise, the chemical tests included qualitative petrographic tests and quantitative potential alkali reactivity tests on the samples from quarry site.

5.2.4 Geological mapping

The scope of the geological surface mapping comprises

- geological surface mapping
- description of the regional geology / hydrogeology setting
- description of the local tectonic setting
- description of the different geological units by determining the type, nature, extent and general characteristics of rock and soil material which are present

with

Detailed geological maps of the site at scale 1:2,000 and 1:2500 • Geological cross-sections at the dam axis and perpendicular to it, along the diversion/headrace tunnel and the surge shaft alignment, at the powerhouse and the tailrace,

- Stereonet diagrams of joint measurements
- Mapping records
- Photo documents.

The geological surface mapping fieldwork for the investigation was carried out over the period December 2017 to April 2018.

5.2.5 Geotechnical field investigations

The geotechnical investigation carried out in early 2018 has been performed to assess the composition, properties and condition of the rock masses and terrain for the feasibility level design of the project structures.

The scope of the geotechnical investigation to fulfill the tasks comprises

- geomorphologic features of the area
- geological composition of the area
- tectonic-structural properties of the area
- hydrogeological properties of the area
- engineering-geological properties of the area
- geotechnical properties of soils and rocks
- presentation of the executed investigation works and previous document
- presentation of geological composition of the wider project area

- presentation of engineering-geological and hydrogeological conditions at
- the location of the project
- cross sections and longitudinal prognostic profiles which include all
- indications of investigation works of engineering geological, hydrogeological and other works, including measurement of physical - mechanical parameters
- results of laboratory tests

The fieldwork for the geotechnical investigation was carried out over the period February to July 2018 and comprised the following:

- Core-drillings
- Test Pits
- Permeability tests
- Standard penetration tests
- Installation of Piezometers
- Laboratory tests on rock/soil samples
- Plate load tests
- Geodetic Survey of all test pit and borehole positions.

the borehole details and the test pits details with general lithology description are shown in the following
Table 5-2 and Table 5-3:

Table 5-2: Borehole details

Borehole	Coordinates	Depth (m)	General Lithology
BH1	N 769601 E 3897788	55	Slightly/Moderately/Highly weathered Quartz Mica Gneiss
BH2	N 769658 E 3897760	27	Slightly/Moderately/Highly weathered Quartz Mica Gneiss
BH3	N 769690 E3897743	30	Slightly/Moderately/Highly weathered Quartz Mica Gneiss
BH4	N 769757 E3897710	48	Slightly/Moderately/Highly weathered Quartz Mica Gneiss
BH5	N 769685 E3897645	40	Slightly/Moderately/Highly weathered Quartz Mica Gneiss
BH6	N 769672 E 3897593	35	Slightly/Moderately/Highly weathered Quartz Mica Gneiss
BH7	N 769652 E 3897684	30	Slightly/Moderately/Highly weathered Quartz Mica Gneiss
BH8	N 769593 E 3897615	22	Slightly/Moderately/Highly weathered Quartz Mica Gneiss
BH9	N 769484 E 3897548	55	Slightly/Moderately/Highly weathered Quartz Mica Gneiss
BH10	N 769716 E 3897854	30	Slightly/Moderately/Highly weathered Quartz Mica Gneiss
BH11	E 772028 N 3889202	60	Slightly/Moderately/Highly weathered Tonalite
BH12	N 771995 E 3889166	40	Slightly/Moderately/Highly weathered Tonalite
BH13	N 772242 E 3889436	170	Slightly/Moderately/Highly weathered Tonalite
BH14	N 772515 E 3889733	250	Slightly/Moderately/Highly weathered Tonalite

Borehole	Coordinates	Depth (m)	General Lithology
BH15	N 772532 E 3889757	70	Slightly/Moderately/Highly weathered Tonalite
BH16	N 772516 E 3890091	54	Slightly/Moderately/Highly weathered Tonalite
BH19	N 772401 E 3890428	55	Slightly/Moderately/Highly weathered Tonalite

A total length of 1300 m core drilling was carried out. All the boreholes were drilled using the conventional straight rotary drillings in overburden and NQ size thin walled double tube core barrel in rock. The drill machines formed holes of 76 mm diameter to recover core of 47.6 mm diameter. For drilling with casing the casing used had an outer diameter of 96 mm and inner diameter of 82 mm.

Table 5-3: Test pit details

Test Pit No.	Coordinates	Depth	General Lithology, Description
TP1			
TP2			
TP3			
TP4			
TP5	E 769684 N 3897711		Silty, sandy gravely boulders
TP6			
TP7	E 769674 N 3897620	1.5	Silty, sandy, clayey and boulders
TP9	E 769667 N 3897655	3	Silty, sandy and gravely boulders
TP10	E 769665 N 3897698	0.8	Silty, sandy clayey with few gravel and boulders
TP11	E 769651 N 3897654	3	Silty, sandy, clayey with few gravel and boulders

The results of the investigation are attached in Appendix 03 and are used as information source for the geological and geotechnical description in following chapters. For more detailed information about the geological and geotechnical situation, it is recommended to consider the results of the investigation campaign attached in the Appendix 1 together with the summary and description as represented in the following sections for tis report.

5.2.6 Results and recommendations

The results of the surface mapping and geotechnical/ geological investigation confirmed that the SHPP site located in a hilly rugged terrain has a geology comprising the geological rock formations of both metamorphic and igneous origin.

It is important to note that the information given in this report relates specifically to the positions at which outcrops were inspected and that the geology between these positions has been interpolated. It is possible that variations in geology may be encountered elsewhere on site on further inspection and/or during construction. These variations must be taken into consideration during detail design and site supervision during construction.

For details see also Annex 01 - Geological Surface Mapping Report.

5.3 Geological Situation

5.3.1 Regional geology

The geological framework of the area surrounding Pakistan has two broad geological partitions namely the Gondwana and Tethyan domain. The southeastern portion of Pakistan fits in to Gondwana domain and is uphold by the Indo-Pakistan crustal plate. The northern most and western parts of Pakistan belong to Tethyan domain and present a composite geology and crustal formations.

The Gondwanian domain is characterized by a continental crust and crystalline basement consolidated in the Precambrian and a platform type development in the Paleozoic. In Asia it is represented largely by the Arabian and Indian shields (kazmi and Jain 1982). The latter forms the Indo-Pakistan subcontinent. Traditionally the Indo. Pakistan subcontinent has been divided into three principal physiographic and geologic divisions a) Peninsular Region b) Himalayan Foredeep and c) the Himalayas (Wadia 1957) as shown in Figure 5-3.

The Tethyan domain covers India and the northern range of Arabia and Africa. This domain stretches from the pacific to the Mediterranean. This evolved through the unique process of rifting, successive fragmentation and calving of several continental blocks of Gondwanaland and their northerly migration across the vast oceanic spaces to unite with Laurasia (Kazmi and Jain 1982). The Tethyan Domain is comprised of an enormous orogenic collage consisting of several continental blocks, stitched together as it were by a amifying network of sutures.

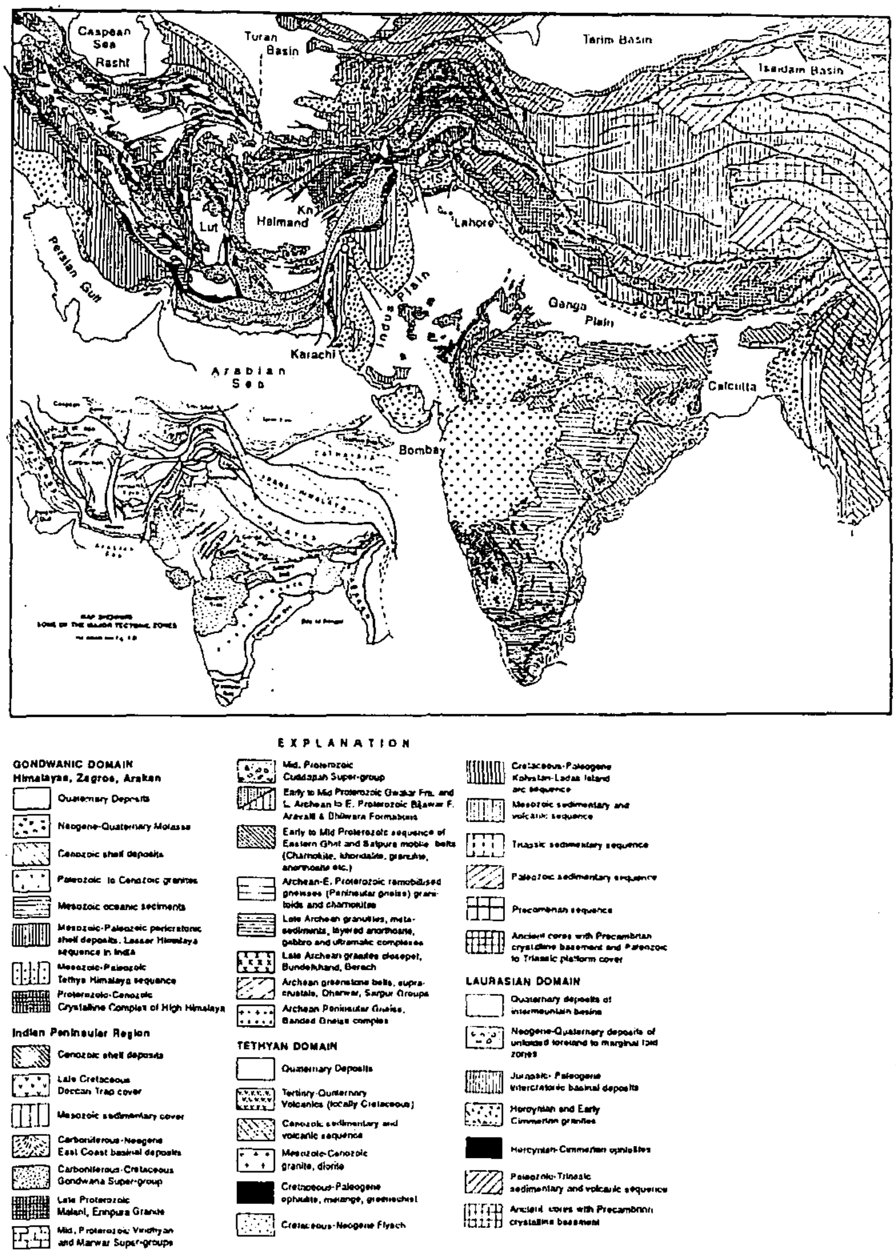
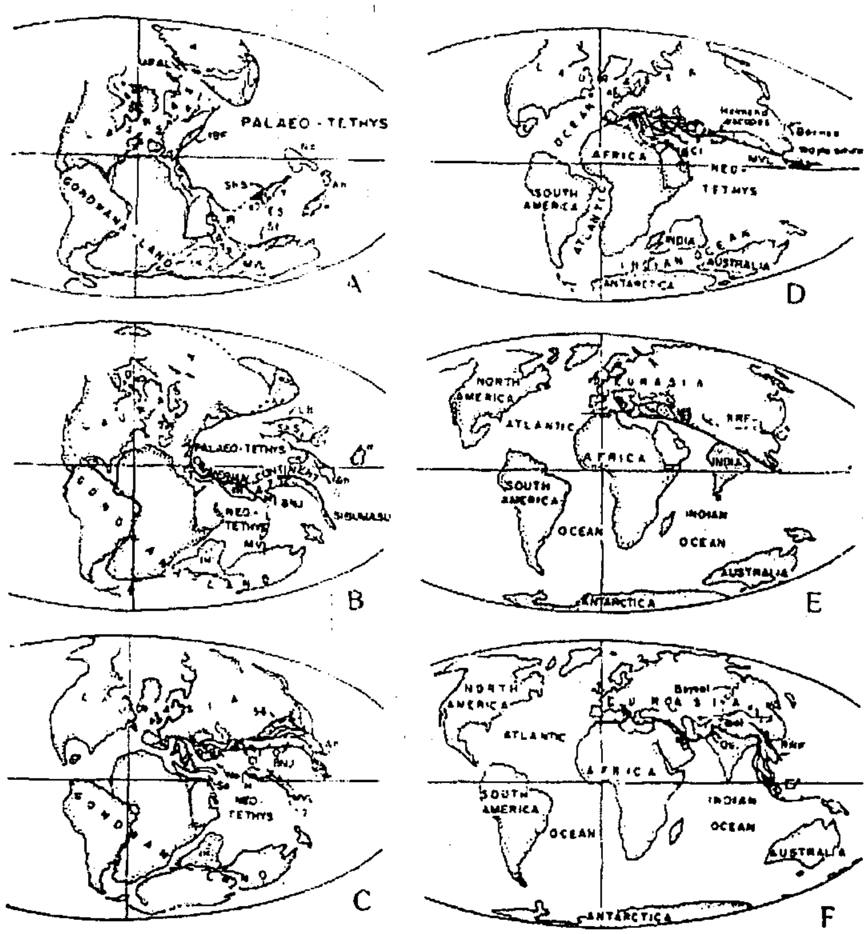


Figure 5-3: Geological sketch map of Pakistan and surrounding regions (compiled from Kazmi and Rana 1982, Stöcklin 1977, Gansser 1980, CGMW 1990, GSI 1977, Drury et al. 1984, Radhakrishna et. al. 1986, Sugden et al. 1990, Ramakrishnan 1986, Kravchenko 1979)

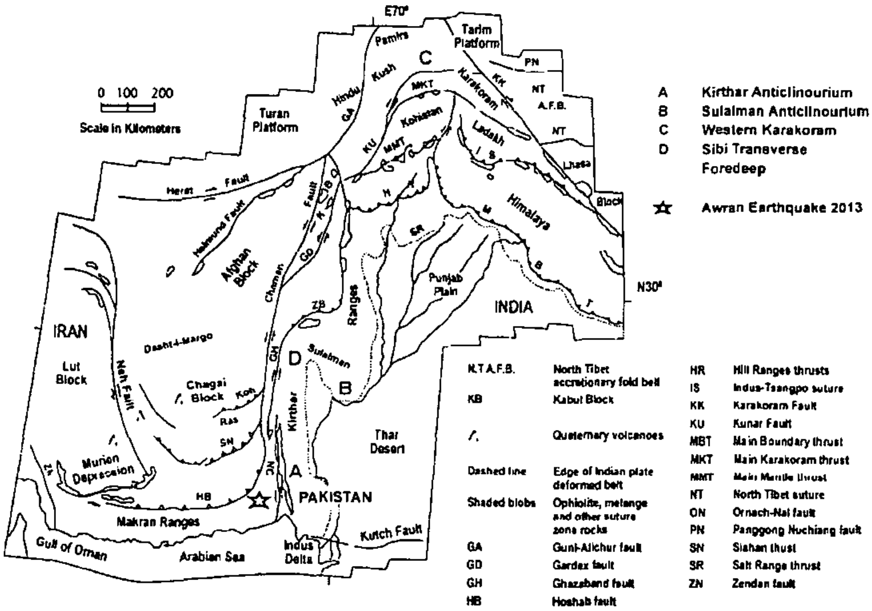
Pakistan may be divided in to large tectonic zones on the basis of plate tectonic settings (geological structures, organic history i.e. age and nature of the deformation, magmatism and metamorphism and lithology) of the area. These major litho-tectonic units from north to south are as shown in Figure 5-4.



Generalised reconstruction of the continents showing evolution of the Tethyan Domain (modified from Sengör et al. 1988) A, Late Permian; B, Early Triassic; C, Late Jurassio; D, Late Cretaceous; E, Middle Eocene; F, Late Miocene.

A–Afghan block, *An*–Annamia, *B*–Bitlis-Pötürge fragment, *BNJ*–Waser/Rushan-Pshart/Banggong Co-Nu Jiang/Mandalay Ocean, *CI*–Central Iranian microcontinent, *CS*–Chola Shan, *ES*–Emei Shan, *F*–Farah block, *H*–Helmand block (sensu Sengör 1984), *Hu*–Huanan block *s.l.*, *IBF*–Istanbul-Balkan fragment, *IR*–Iranian block (undisrupted: i.e., Sanandaj-Sirjan zone + north-west Iran and central Iranian microcontinent), *K*–Kirschir block, *L*–Lhasa block, *LB*–Luochou Arc, *MVL*–Mount Victoria Land, *NC*–North China, *No*–northern branch of Neo-Tethys, *p*–Pachelma aulacogen, *Q*–Quangtang. *Qu*–Quetta-Sibi graben, *RRF*–Red River Fault, *s's'*–Tarim, *S'*–Pamir-west Quangtang block, *S"*–

Figure 5-4: Generalized reconstruction of the continents showing evolution of Tethyan Domain (modified from Sengor et. al. 1988)



Tectonic structures and faults of Pakistan (After Kazmi, 1979).

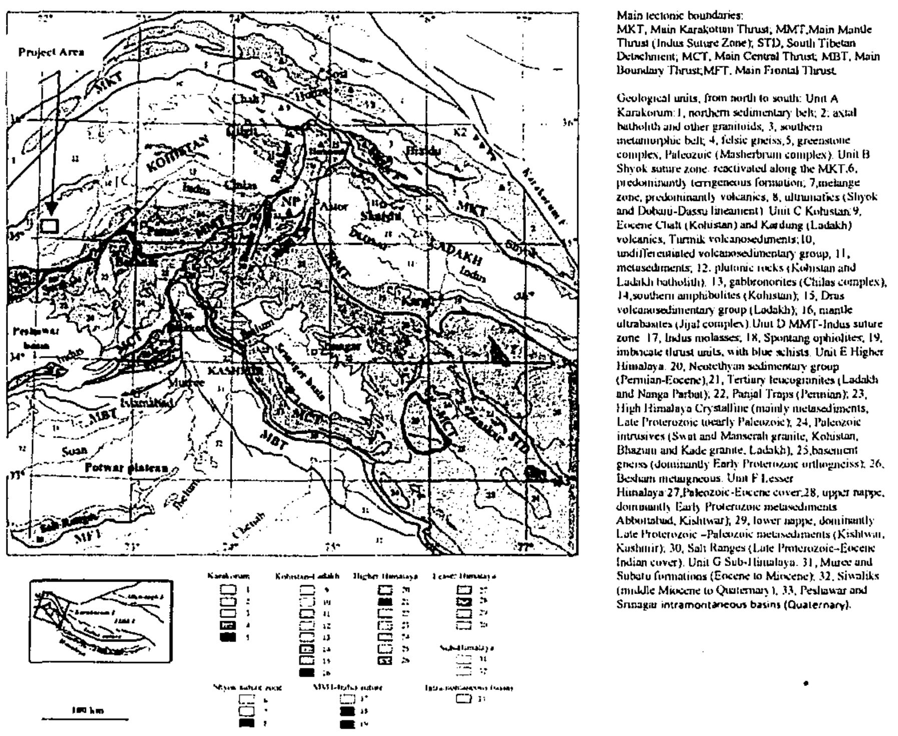


Figure 5-5: Geological map of Pakistan -the NW Himalaya (modified after Bossart and Ottiger [1989], Burbank et al. [1986], Greco et al. [1989], Burg et al. [2005a], DiPietro et al. [2000], DiPietro and Pogue [2004], Edwards et al. [2000], Fontan et al. [2000], Gaetani [1997], Greco [1991], Kasmi and Jan [1997], Le Fort and Pe^cher [2002], Lombardo and Rolfo [2000], Reuber [1989] Rolland et al. [2000, 2002], Schneider et al. [2001], Steck [2003], Tahirkheli [1996], Valdiya [1998], Wadia [1975], and Zanchi and Gaetani [1994]).

The tectonic setting of Pakistan is shown in figure 4 along with the geological formations of NW-Pakistan. Pakistan is located along the western plate boundary of the Asian plate with Afghanistan belonging to the west, Tadjik Basin, Tarim Basin and Qiantang to the north and the Indian plate belonging to the east. The western boundary with Afghanistan is fault bounded by the Charman fault.

At the northern boundary of Pakistan the Kohistan Magmatic Arc was formed between the Asian plate and the Indian plate in late cretaceous as shown in Figure 5-5. The magmatic arc is bounded by Main Karakoram Thrust (MKT or Shyok suture zone) in the north which is subducted below Asian plate and Main Mantle Thrust (MMT) to the south where the Indian plate is subducted. These faults in the region are still active and presents northern Pakistan as one of the seismically active zone in the world.

5.3.2 Geology of Pakistan

Pakistan can be divided into nine tectonic zones (Kazmi Jain 1997) as shown in Figure 5-6:

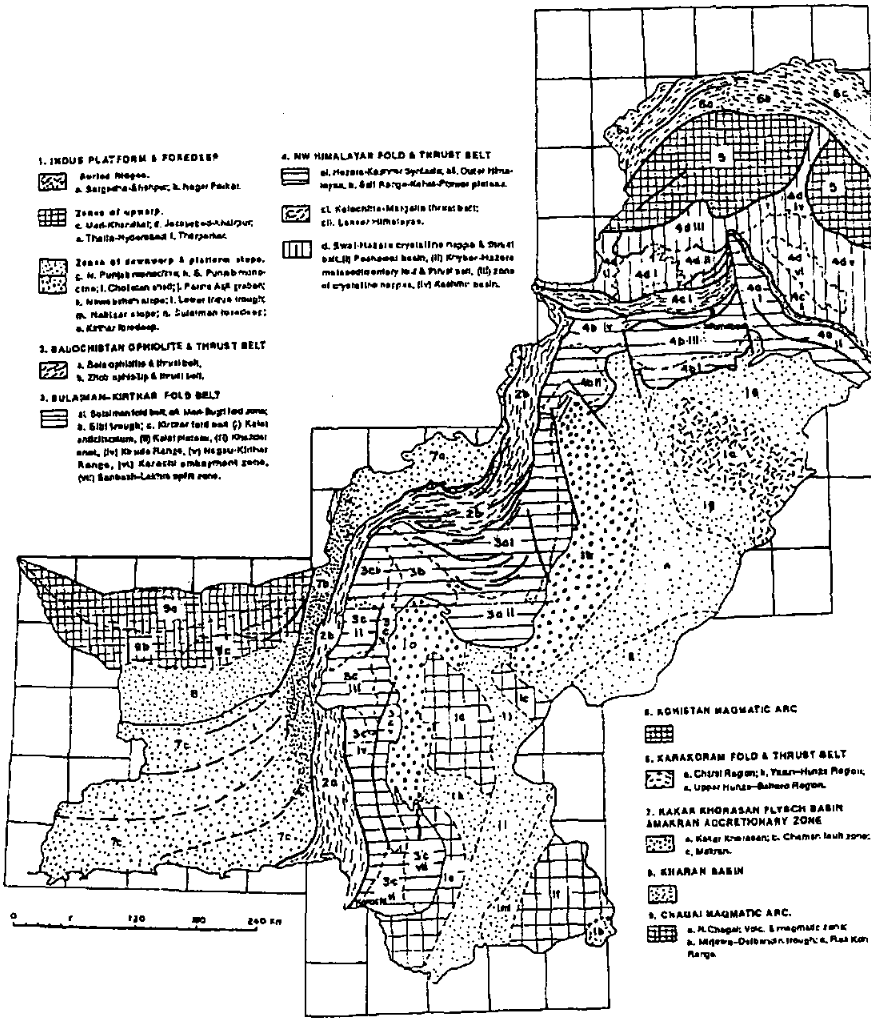


Figure 5-6: Map showing tectonic zones of Pakistan (Kazmi Jain 1997)

These 9 zones are as follows:
1) Indus platform and foredeep, 2) Balochistan Ophiolite and fold and thrust belt, 3) Sulaiman-Kirthar fold belt, 4) Northewest Himalayan fold and thrust belt, 5) Kohistan magmatic arc, 6) Karokaram fold and thrust belt,
7) Karakoram fold and thrust belt, 8)Kakar Khorasan flysh basin and
9) Makran accretionary zone, Kharan basin and Chagai magmati arc.

The details to the general geology is covered in depth in the geological surface mapping included as Annex to this report.

5.3.3 Geology of Northern Pakistan

The proposed SHPP site is located in the western part of the Kohistan magmatic arc, which is an intraoceanic arc bounded by MMT and MKT (see Figure 5-7).

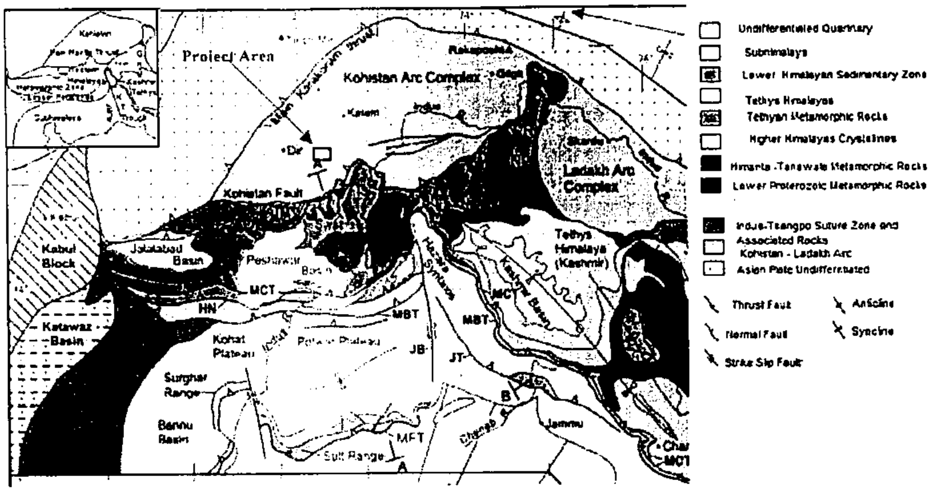


Figure 5-7: Tectonic Map of the NW Himalayas (DiPietro and Mouase 2004) The Kohistan Arc Complex is developed on the Hazara Massif (Arsentyev et al., 1978). HKT Trough – Hazara-Kashmir Transverse Trough

The dominant rocks of the region are plutonic (intrusive) igneous rocks of granitoid as the Kohistan Batholith and Chilas complex, Kamila Amphibolites, and minor volcanic rocks and metamorphosed sedimentary rocks. The principal rock types of Kohistan Arc from top to base are shown in Figure 5-8:

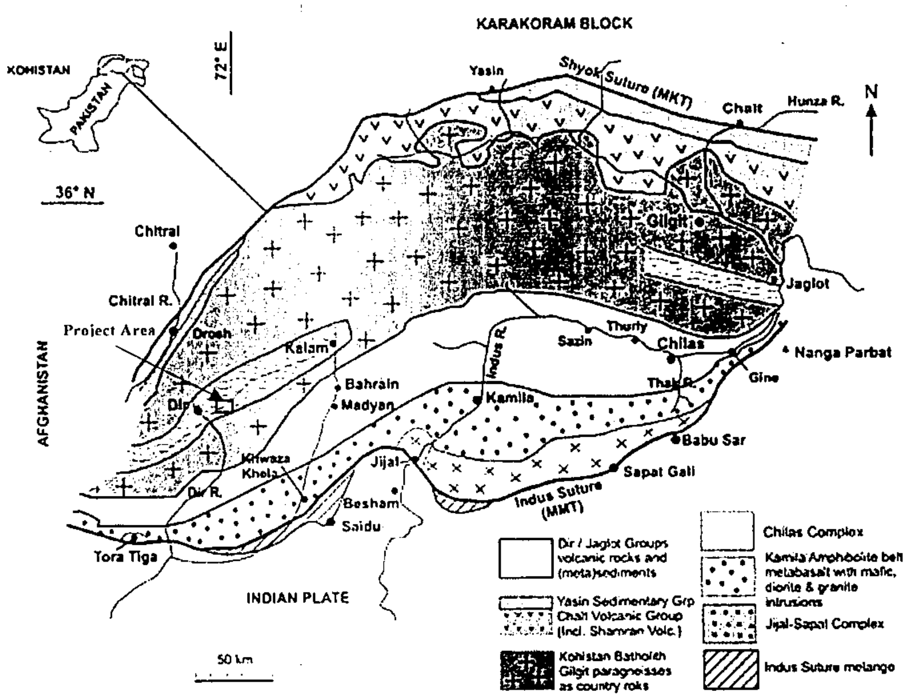
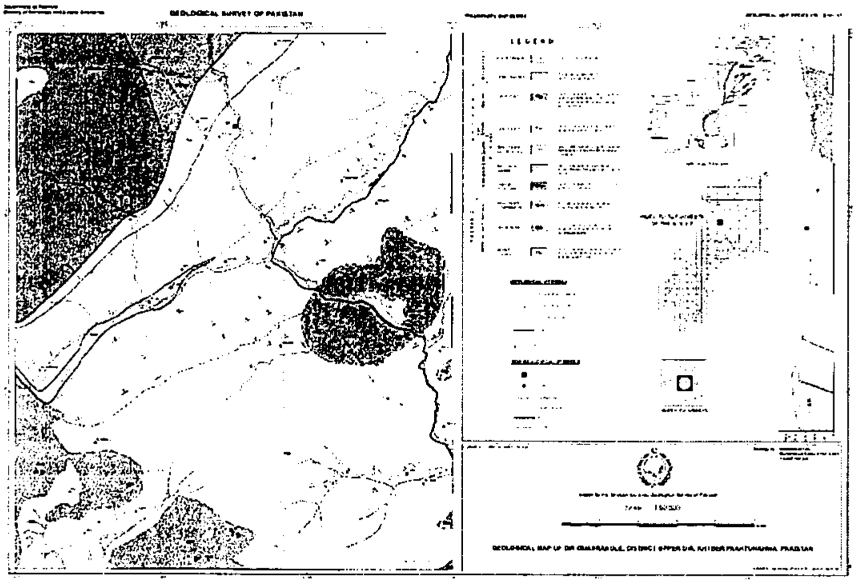


Figure 5-8: Principal rock types of Kohistan Arc

5.3.4 Project Site Geology

Tertiary and Cretaceous formations dominate the geology around the project site. The outline of geological units based on the geological map of scale 1:50,000 obtained from Ministry of Petroleum and natural resources, Pakistan 2018 are shown in Figure 5-9.



Source: Ministry of Petroleum and natural resources, Pakistan 2018

Figure 5-9: Geological map of project area

As per the geological map of project area:

1. The tertiary formations are
   - Ultrar (Tuv)- Dir Group: Light gray, maroom red color tuff and agglomerate (andacite and dacite) of Late Eocene
   - Baraul Banda SlatesPhyllites (TbP)- Dir Group: Gray, green maroon color thinly bedded fine texture with occasional thin beds of limestone from Paleocene to Early Eocene
   - Baraul Banda Quartzite (Tbq)- Dir Group: Light to dark gray and brownish gray, thin to thick bedded fine grained and cherty at places
   - Shandur Granodiorite (Ksg): Light gray to brownish in color medium grained hard and massive from Early Cenozoic.

2. The Cretaceous formations are
   - Baraul Banda Meta volcanics (Kbm): Brownisch and dark brown in color metamorphosed basaltic quartz.From middle to late Cretaceous
   - Deshai Diorite (Kdd): Gray greenish gray medium grained composed of hornblended biotite with subordinate quartz
   - Banded Amphibolite (Mzb): Light to dark green fine to medium grained, foliated and composed of plagioclase hornblende and quartz.

5.3.5 Litho-Stratigraphy and Minerology

Rock types of the project area are broadly classified into different units. They are quartz mica gneiss, impure marble, sub Phyllite/Slate, meta Sandstone, Tonalite, Epidosite, Parama Amphibolite and Granite. Lithological descriptions of rock units exposed on the area at respective locations are given in details in the geological surface mapping which forms the Annex of this report.

During the course of geological surface mapping survey, sampling were done from most of the rock units encountered at the project site and were sent to lab for the petrographic analysis. Petrographic analysis is carried out to identify the mineral and its quantification comprising the rock. In Table 5-4, the result of the petrographic analysis has been summarized:

Table 5-4: Summary of petrographic analysis on rock samples from project site

Area	Intake (Reservoir and Dam site) and Tunnel					Tunnel, Surgetank, Powerhouse and Tailrace					
Rock Type	Sub Phyllite / Slate	Low Grade Impure Marble	Calcareous Meta Sandstone	Quartz mica Gneiss	*Granite*	*Tonalite*	*Para Amphibolite*	*Quartz Mica epidote Gneiss*	*Epidosite*	*Granite*	Granite
Texture and Structure	fine to very fine grained	calcite	anhedral to subidioblastic. Idioblastic crystals	fine to medium grained	medium Grained hypidiomorphic and subporphyritic	medium Grained hypidiomorphic and subporphyritic	fine to medium grained	fine to medium grained	fine to medium grained	medium Grained hypidiomorphic and subporphyritic	fine to medium grained
Minerology											
Albite/Oligoclase	0.5 - 1.5			0.5 - 1.5			23.5	2	1.5		29.5
Amphibole				2 - 22.5	1.5	14 - 14	35.5	9.5		3	4.5
Biotite	2 - 14.5	4.5	13.5	11 - 36.5	5.5	2.5 - 2.5	3	11.5	0.5	9.5	10.5
Calcite	6.5 - 20.5	3.5		2 - 17			3	1			
Carbonate		51			2.5	1.5 - 1.5	0.2			2	2.5
Chert	2 - 3										
Chlorite	2 - 9.5	2	18	2.5 - 8.5	1	1.5 - 1.5	4.5	2	4.5	3.5	2.5
Epidote			6.5		3	2 - 2			48.5	4.5	1.5
Haematite/Limonite	1 - 1.5	1	1	0.2 - 1.5		0.3 - 0.3		0.3	0.3	0.5	0.5
K-Feldspar				0.2 - 1	15.5	3.5 - 3.5	4	1	1	17	16
Magnetite	1.5 - 2		2.5	1.5 - 3	1.5	1 - 1	3.5	1	1	2	2
Microcrystalling Quartz		20	32								
Muscovite/Sericite	4 - 5.5			2.5 - 20.5	6.5	1.5 - 1.5	2	12.5	1	2.5	2.5
Plagioclase				0 - 0	22.5	20 - 20				25.5	
Quartz	12 - 22.5	15.5	20.5	26.5 - 52.5	39.5	25 - 25	20.5	30.5	45.5	27.5	28.5
Sericite/ILLite	27.5 - 57	2.5	6								
Slate/Argillite	1.5 - 1.5										
Sphene					0.2	0.1 - 0.1				0.3	0.3
Sphene	traces		traces	traces			traces	traces	traces		
Tourmaline					0.3	0.3 - 0.3				2	
Tourmaline	traces		traces	traces				traces	traces		
Volcanic Rock Fragments	1 - 2										
Zircon	traces		traces	0.1 - 0.3	0.2	0.2 - 0.2	0.3	0.2	0.3	0.2	0.2

5.3.6 Geological structures

The project area lies between two tectonic features described as Main Karakorum fault and main Karakorum thrust and is presented in Figure 5-10:

Source Google earth

Figure 5-10: Geological structures (Project area Sharmai HPP)

5.3.6.1 Main Karakoram Thrust (MKT)

The Main Karakoram Thrust follows roughly the course of the Shyok suture zone and defines the southern margin of the Karakoram plate. It is a north of north-northeast dipping thrust fault that offsets regional metamorphic isograds of the M2 stage on the hanging-wall, and also isograds in the Ladakh Terrain in the foot-wall, The Main Karakoram Thrust places Karakoram metamorphic rocks over low-grade rocks of the Shyok suture zone and the northern margin of the Kohistan-Ladakh Terrain. The latest phase of metamorphism in the Karakoram (M4) in the Thalle and Braldu Valleys, is a retrogressive event and is inferred to be related to late-stage motion on the Main Karakoram Thrust. Although no neo-tectonic motion has been reported for the MKT there appears to be a considerable difference in topography either side of the fault and it is inferred that recent rapid uplift and exhumation of the entire southern part of the Karakoram Terrain has occurred by thrust culmination on the hanging wall of the Main Karakoram Thrust.

5.3.6.2 Main Mantle Thrust (MMT)

The Main Mantle Thrust refers to the part of the Indus Suture Zone exposed in Pakistan, marking the suture zone between the Kohistan island arc terrane and the Indian Plate. The MMT, for its most parts in Northern Pakistan is oriented more or less in an east-west trend, but in the Nanga Parbat region, it forms a north trending loop surrounding the Nanga Parbat Syntaxis. Also, whereas elsewhere it dips to the north, in the Nanga Parbat region it is virtually vertical, or at places, even overturned.

The MMT in the Nanga Parbat Syntaxis marks the contact between the Cretaceous Kohistan rocks and the metasediments of the Nanga Parbat Group. In much of the region the contact is sharp between the two groups of lithologies, although there is evidence that locally the MMT comprised more than one fault, which involved tectonic slivers from both the Kohistan and Indian Plate rocks. This is particularly true for part of the MMT exposed at the eastern margin of the Syntaxis at Stak.

5.3.6.3 Bedding and foliation

Bedding and foliation is well observed in all litho-stratigraphic units. Since the area mainly comprises metamorphic rocks and igneous, foliation is common compared to bedding. The details on the foliation and the rock properties has been discussed in the following chapters 3.7.

5.3.6.4 Joints

Joints are well observed on sub-phyllite, mica gneiss, tonalite and granite. Details of joint measurement and its statistical analysis are presented in surface geological study of the project area and are summarized in following chapter 3.7.

5.3.6.5 Shear zone

Shear zones if present are usually characterized by shear/crushed rock with bands different rock soil units with certain trend of foliation plane. These shear zones are usually the result of the tectonic activity.

Although, the project region lies in the vicinity between the two-major tectonic thrust zones namely MKT and MMT, no such marked shear zones were identified during the course of geological and geotechnical investigation of SHPP.

5.3.7 Engineering Geology of Project Area

This chapter includes the findings of the **geological surface mapping** and **geotechnical investigation** and **geophysical seismic refraction surv**ey carried out at the site. The geological surface mapping is included as Annex 01 and geotechnical investigation as Factual Report, Annex 02 and geophysical seismic refraction survey as Annex 03.

This chapter includes engineering geological mapping of major engineering structures of the project, rock mass classification of the area with stability analysis, detail geotechnical, geological and geophysical investigation, rock quality designation and preliminary support design. Statistical joint analysis of the area has been done based on detail measurement of all discontinuities. Rock mass rating (RMR) and rock tunneling quality index (Q) are used for rock mass classification which helps to study characteristics and quality of rock mass of the proposed underground structures i.e. headrace tunnel, surge tank power cavern and tailrace tunnel.

5.3.7.1 Engineering geological conditions at the reservoir

The surface map based on the geological mapping at the intake and reservoir is shown in Figure 5-11. The extent of reservoir at SHPP comprises of rock of metamorphic and igneous origin. The proposed dam site is mostly dominated by quartz mica gneiss.

At the upstream part and the downstream site, sub phyllite is abundant. Apart from these rocks, the metasandstones, impure marble have also been traced at the upstream side of the proposed dam site. At the downstream side quartz mica gneiss along with sub phyllite and granite outcrop are visible. In addition to these rock units, the area is overlain by overburden soil in the form of coarse- and fine-grained soil in different composition and/or in the form of boulders.

The rock/soil types, their essential characteristic features and the measured structural units are shown in Table 5-5 and some of the site outcrops pictures are shown in Figure 5-12:

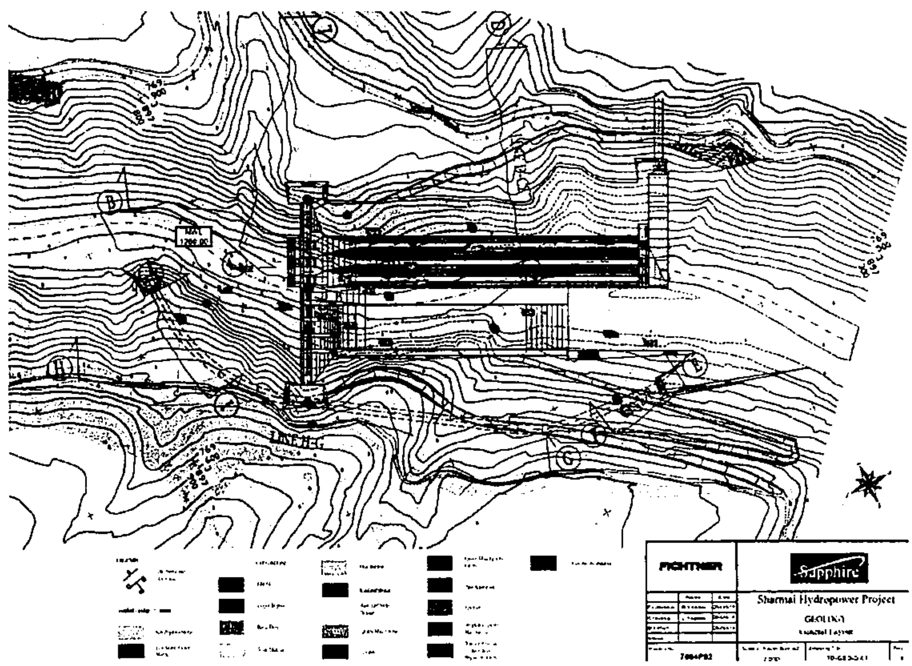


Figure 5-11: Surface geology at the intake and reservoir

Table 5-5: Rock/soil units measured at reservoir and intake

Rock type/ Soil units	Characteristic	Foliation	Joint Set
Sub-Phyllites Slates	greenish brown to greenish grey, moderately weathered, joints are moderate to sparsely jointed, filled with clay, surface staining is also seen at some places, with smooth planer surfaces and persistence is medium to high	Strike: N40E to N80E Dip. 40SE to N80SE and 40NW to 75NW	J1: N70E/55NW J2: N20W/60SW
Low Grade Impure Marble	fine grained, moderately strong to strong, majorly sparsely jointed, generally filled with clayey matrix, partially filled with quartz, the joint surfaces are smooth to plane smooth and undulating. Joints are of medium to high in persistence	Strike: N75E to N85E Dip. 50NW to 55NW	J1 N25E/70SE J2 N70W/45SW
Calcareous Meta Sandstone	grayish black, fine grained, closely foliated, tight to moderately open joint sets, the surfaces of joints are undulating planer and having no joint seepages, also having the quartz in the infilling materials.	Strike: N45W and N40E to N50E Dip. 20NE and 40NW to N70W	J1 N70W to N80W/ 75NE & 40SW J2 N20E to N30W to N70W / 40SE, 50SW & 55NE
Quartz Mica Gneiss	dark grey to greenish grey on fresh surface, weathers to rusty brown to yellowish Brown in color, fine to medium grained, strong to very strong, slightly to moderately weathered, Medium to widely jointed and also closely to medium jointed at places	Strike. N50E to N70E Dip. 30NW to 80NW	J1 N45W to N80W & N40E to N70E / 25NE to 50NE, 30SW to 45SW J2 N50W to N75W & N35E to N60E / 45SW to 50SW, 40SE to 55SE J3 N05W to N30W / 70NE to 80NE
Granite	smoky light in color, medium to coarse grained, strong to very strong, slightly to moderately weathered. Medium to widely jointed and also closely to medium jointed at places, making a sharp contact with Quartz mica Gneiss		J1 N35E to N50E Strike with 40NW to 80NW J2 N70W to N80W strike with 40SW to 60SW J3 N30E to N30E strike with 30SE to 70SE
Angular Boulders, Gravel with Fines (ABGM)	rounded to sub rounded and with varies sizes and shapes River deposits are Fine to medium to course grained with boulders are various sizes and shapes rounded to sub rounded		

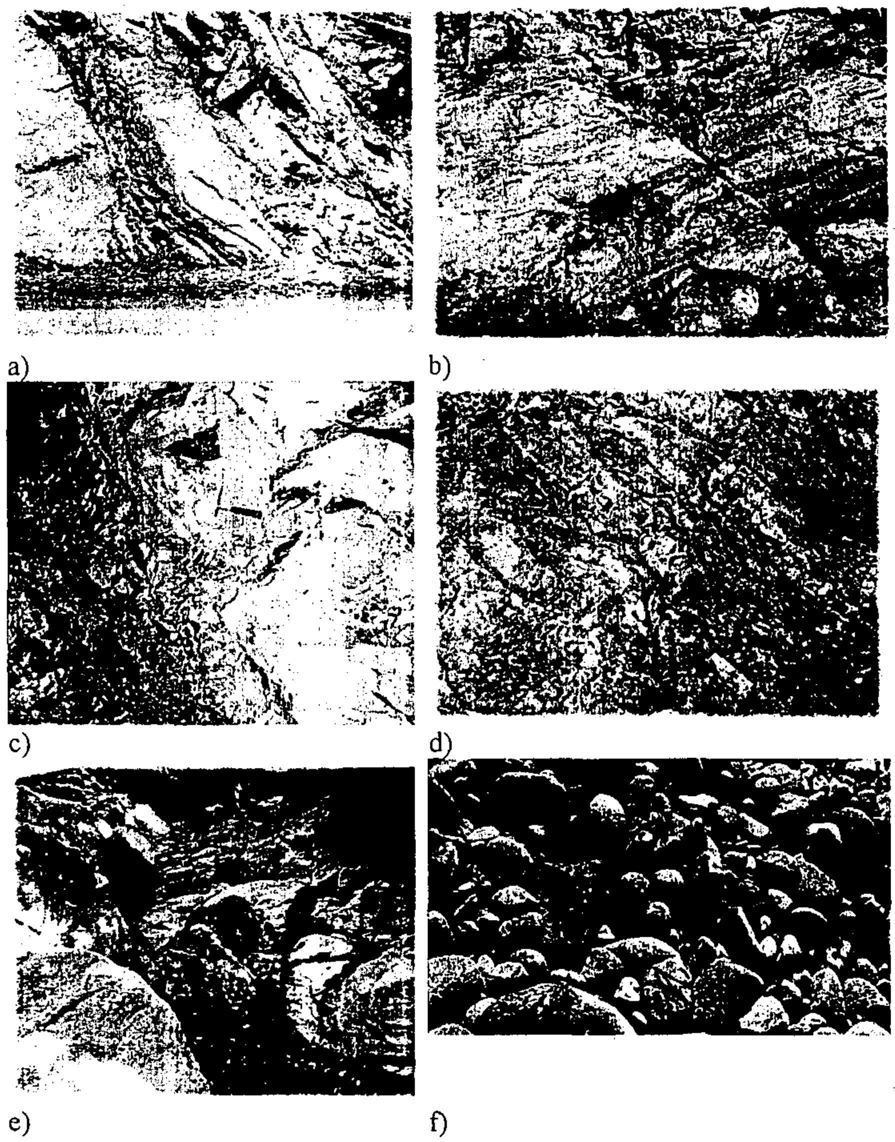
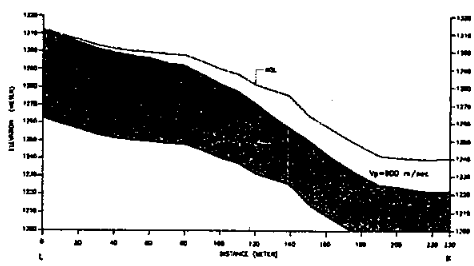
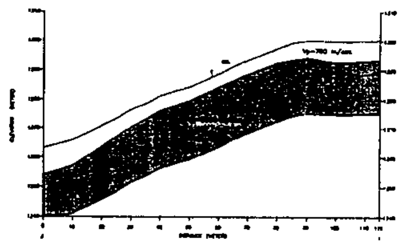


Figure 5-12: An outcrop of a) Quartz mica gneiss b) Sub-Phyllite/Slates c) low grade impure marble and d) calcareous meta sand stone e) Granite and f)Angular Boulders, Gravel with fines (ABGM)

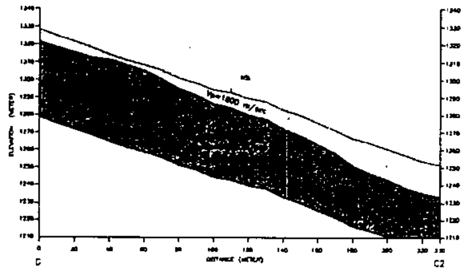
Figure 5-13 shows the geophysical seismic refraction profiles on the upstream and downstream from center to left and right abutments. The thickness of the overburden on the left abutment decreases towards hillside from around 20m at the center to around 6m. On the right abutment, the thickness of the overburden material remains almost constant by 15-20 m from river towards the hill side.



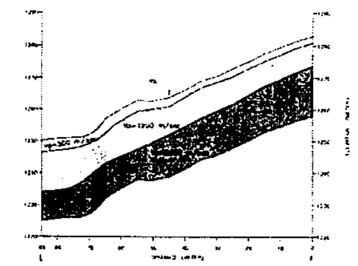
a) left abutment towards river center (u/s)



b) river towards right abutment (u/s)



c) left abutment towards river center (d/s)



d) river towards right abutment (d/s)

Figure 5-13: Geophysical seismic refraction result, a) and b) section upstream and c) and d) section downstream of reservoir

5.3.7.2 Engineering geological conditions at Dam site

The proposed concrete dam is on a wide flat alluvium overburden terrace. The terrace consists of a thick accumulation of alluvial/colluvium deposit comprising of light- to dark grey, rounded to sub-rounded, fine to coarse-grained boulders/ rock pieces of metamorphic origin.

The depth of this overburden as per borelogs varies from 6 m along the left bank up to 20 m in the middle of the river and 15 m-20 m on the right bank (Figure 5-14). The overburden is underlain by light gray, slightly to highly weathered quartz mica gneiss.

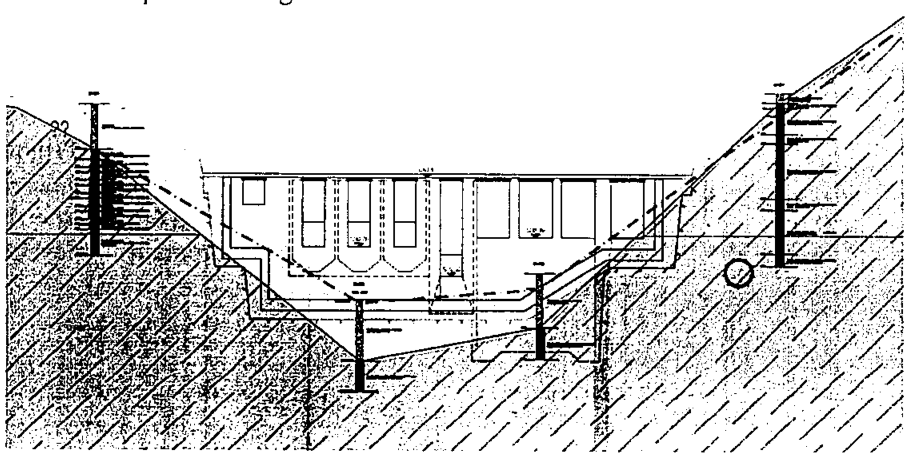


Figure 5-14: Dam longitudinal section with borehole locations

Along the right and left abutment, hill slope ranges between 30° and 70°. The visible surface rock-outcrop consists of quartz mica gneiss. These are dark grey to greenish grey on fresh surface and weathers to rusty brown to yellowish. Moreover, these are fine to medium grained, strong to very strong, slightly to moderately weathered, medium to widely jointed and closely to medium jointed at places

During the geotechnical investigation work, along the dam axis 4 boreholes (BH1 to BH4) with depth varying between 27 m and 55 m were drilled.

The Field permeability tests in the form of Lugeon tests were carried out in these bore holes at the depth of rock with varying weathering categories ash shown in Table 5-6.

Table 5-6: Lugeon tests in boreholes at dam site BH1, BH2, BH3 and BH4

Bore Hole	Tiefe (m)	Lugeon	hydraulic conductivitiy (Fell et. al., 2005) (m/s)	Interpretation (T. Houlsby 1976)	Classification	Rock discontinuity condition
BH1						
	5 - 10	0.02	< 1E-07	Turbulent Flow	very low	very tight
	10 - 15	0.01	< 1E-07	Turbulent Flow	very low	very tight
	15 - 20	0.00	< 1E-07	Turbulent Flow	very low	very tight
	20 - 25	0.00	< 1E-07	Void Filling	very low	very tight
	25 - 30	0.00	< 1E-07	Turbulent Flow	very low	very tight
	30 - 35	0.00	< 1E-07	Turbulent Flow	very low	very tight
	35 - 40	0.00	< 1E-07	Turbulent Flow	very low	very tight
	40 - 45	0.00	< 1E-07	Turbulent Flow	very low	very tight
	45 - 50	0.00	< 1E-07	Void Filling	very low	very tight
	50 55	0.00	< 1E-07	Void Filling	very low	very tight
BH2						
	17 - 22	0.00	< 1E-07	Dilation	very low	very tight
	22 - 27	0.00	< 1E-07	Turbulent Flow	very low	very tight
BH3						
	20 - 25	0.63	< 1E-07	Turbulent Flow	very low	very tight
	25 - 30	3.13	< 6E-07	Dilation	low	tight
BH4						
	23 - 28	0.01	< 1E-07	Void Filling	very low	very tight
	28 - 33	0.00	< 1E-07	Turbulent Flow	very low	very tight
	40 - 45	0.00	< 1E-07	Void Filling	very low	very tight

The Lugeon value for all the BHs at different depths except for 1 test at BH 3, 25-30 m showed values less than 3. As per the results the rock can be classified as having very low permeability and the corresponding rock discontinuity to be very tight. Apart from this, the pressure flow (p-q) diagram indicated most of the tests to lie in the range of turbulent flow region. As per Houlsby's (1976) interpretation, this indicates that the hydraulic conductivity of the rock mass decreases as the water pressure increases, which characterize the behavior of rock masses exhibiting partly open to moderately wide cracks.

The Borelogs at these locations indicate the general trend of rock joints showing partly open to very tight aperture (between 0.5 mm-<0.1mm) with mostly quartz or clayey matrix infilling and having roughness varying between undulating rough to planar smooth.

The Laboratory investigation results on the samples obtained from BH1, BH2, BH3 and BH4 are presented in Table 5-7Table 5-7: Laboratory results BH1, BH2, BH3 and BH4

Borehole Number	Samples Depth (m)	UCS				Shear Strength		Brasilian Test (Tensile Strength)	Pointload Index		Density	Water content	Porosity	Water absorption	Slake durability index
		σ_c (MPa)	E (GPa)	ν (-)	Deformation (GPa)	φ (°)	c (MPa)	σ_t (MPa)	$I_{s,50}$ (Mpa)	Remarks	ρ (g/cm³)	w_n (%)	n (%)	w_a (%)	s (%)
BH-1	10.76 - 11.00										2.70	0.15	1.38	0.51	99.5
	11.00 - 11.36								9.67	Diam.					
	13.80 - 14.10								9.22	Diam.					
	14.16 - 14.50	150.4	49.2		24.6										
	16.67 - 16.87								2.40	Diam.	2.79	0.21	1.15	0.41	98.2
	18.96 - 19.22								3.95	Diam.					
	19.50 - 19.75										2.73	0.10	1.64	0.60	98.7
	23.76 - 24.13								1.98	Diam.					
	24.24 - 24.38								1.54	Diam.					
	25.96 - 26.17								11.19	Diam.					
	26.17 - 26.50								11.19	Diam.	2.69	0.08	1.12	0.42	99.6
	27.60 - 27.92	47.3	62.5		31.2			8.93							
	45.13 - 45.42								6.58	Diam.					
	47.18 - 47.44	150.4	62.1		31.1										
	48.00 - 48.50										2.69	0.08	1.57	0.58	99.6
	49.61 - 49.91	126.2	92.7		46.3										
	51.12 - 51.31								4.17	Diam.					
	52.60 - 52.84	92.8	48.8		24.4										
	52.60 - 52.84										2.78	0.08	1.41	0.51	99.3
	54.64 - 54.85														
BH-2	17.90 - 18.00								5.71	Diam.					
	18.68 - 18.93	209.8	87.2		43.6										98.9
	20.35 - 20.50								10.09	Diam.					
	20.50 - 20.80	196.5	91.7		45.8						2.73	0.11	0.92	0.34	99.6
	21.17 - 21.37					44.0	2.2								
	21.40 - 21.57								8.34	Diam.					
	22.69 - 22.87	198.8	10.3		51.6						2.76	0.09	0.78	0.28	99.5
	23.68 - 23.82										28.80	0.09	0.76	0.26	99.6
	25.06 - 25.24	157.4	78.1		39.1										
	25.50 - 25.64								5.49	Diam	2.77	0.15	1.23	0.4[illegible]	99.5
BH3	20.57 - 20.69								8.12	Diam.					
	21.53 - 21.78										2.80	0.10	0.84	0.30	99.5
	22.19 - 22.35								4.83	Diam.					
	22.82 - 22.96										2.76	0.20	0.84	0.31	99.5
	23.55 - 23.71								8.34	Diam.					
	24.10 - 24.25								9.66	Diam.					
	25.00 - 25.28										2.70	0.05	0.27	0.10	99.5
	25.78 - 26.00					41.2	4.2								
	26.20 - 26.43							22.46			2.94	0.14	0.40	0.14	99.5
	26.85 - 27.00								5.05	Diam.					
	27.13 - 27.27										2.73	0.08	0.64	0.23	99.4
	27.64 - 27.80								10.75	Diam.					
	29.70 - 29.91	122.8	51.5		25.7										
BH4	29.59 - 29.73	34.9	31.8		15.9						2.95	0.54	4.76	1.62	
	30.00 - 30.71								8.33	Diam.	2.73	0.15	1.20	0.44	
	38.00 - 38.27														
	38.35 - 38.60										2.76	0.06	0.63	0.23	
	38.79 - 39.00	58.5	30.1		15.0				4.01	Diam.					
	39.74 - 40.00								7.90	Diam.					
	40.00 - 40.30										2.75	0.34	1.40	0.51	
	40.35 - 40.60							17.13							
	42.27 - 42.47								6.78	Diam.					
	42.69 - 43.00	52.2	51.6		25.8										
	44.51 - 44.71								9.38	Diam.					
	44.75 - 45.00	69.7	28.2		14.1										
	45.57 - 45.80														
	47.59 - 47.89	92.8	54.9		27.4			8.67	10.75	Diam.					

In general the Laboratory test results shows the range of rock parameters with unit weight: 27-29 kN/m³, internal angle of friction: 41-44°, cohesion: 2.2-4.2 MPa, unconfined compressive strength:38-209 MPa, tensile strength: 4 - 22 MPa, point load strength: 2-11 MPa, young's modulus: 40 GPa to 80 GPa, slake durability > 99% (very high durability, Gamble chart) and water absorption: 0.5- 1.6%.

Table 5-7: Laboratory results BH1, BH2, BH3 and BH4

Borehole Number	Samples Depth (m)	UCS				Shear Strength		Brasilian Test (Tensile Strength)	Pointload Index		Density	Water content	Porosity	Water absporption	Slake durability index
		σ_c (MPa)	E (GPa)	v (-)	Deformation (GPa)	φ (°)	c (MPa)	σ_t (MPa)	$I_{s,50}$ (Mpa)	Remarks	ρ (g/cm³)	w_n (%)	n (%)	w_a (%)	s (%)
BH-1	10.76 - 11.00										2.70	0.15	1.38	0.51	99.5
	11.00 - 11.36								9.67	Diam.					
	13.80 - 14.10								9.22	Diam.					
	14.16 - 14.50	150.4	49.2		24.6										
	16.67 - 16.87								2.40	Diam.	2.79	0.21	1.15	0.41	98.2
	18.96 - 19.22								3.95	Diam.					
	19.50 - 19.75										2.73	0.10	1.64	0.60	98.7
	23.76 - 24.13								1.98	Diam.					
	24.24 - 24.38								1.54	Diam.					
	25.96 - 26.17								11.19	Diam.					
	26.17 - 26.50								11.19	Diam.	2.69	0.08	1.12	0.42	99.6
	27.60 - 27.92	47.3	62.5		31.2			8.93							
	45.13 - 45.42								6.58	Diam.					
	47.18 - 47.44	150.4	62.1		31.1										
	48.00 - 48.50										2.69	0.08	1.57	0.58	99.6
	49.61 - 49.91	126.2	92.7		46.3										
	51.12 - 51.31								4.17	Diam.					
	52.60 - 52.84	92.8	48.8		24.4										
	52.60 - 52.84										2.78	0.08	1.41	0.51	99.3
	54.64 - 54.85														
BH-2	17.90 - 18.00			-					5.71	Diam.					
	18.68 - 18.93	209.8	87.2	-	43.6										98.9
	20.35 - 20.50								10.09	Diam.					
	20.50 - 20.80	196.5	91.7		45.8						2.73	0.11	0.92	0.34	99.6
	21.17 - 21.37					44.0	2.2								
	21.40 - 21.57								8.34	Diam.					
	22.69 - 22.87	198.8	10.3		51.6						2.76	0.09	0.78	0.28	99.5
	23.68 - 23.82										28.80	0.09	0.76	0.26	99.6
	25.06 - 25.24	157.4	78.1		39.1										
	25.50 - 25.64								5.49	Diam.	2.77	0.15	1.23	0.45	99.5
BH3	20.57 - 20.69								8.12	Diam.					
	21.53 - 21.78										2.80	0.10	0.84	0.30	99.5
	22.19 - 22.35								4.83	Diam.					
	22.82 - 22.96										2.76	0.20	0.84	0.31	99.5
	23.55 - 23.71								8.34	Diam.					
	24.10 - 24.25								9.66	Diam.					
	25.00 - 25.28										2.70	0.05	0.27	0.10	99.5
	25.78 - 26.00					41.2	4.2								
	26.20 - 26.43							22.46			2.94	0.14	0.40	0.14	99.5
	26.85 - 27.00								5.05	Diam.					
	27.13 - 27.27										2.73	0.08	0.64	0.23	99.4
	27.64 - 27.80								10.75	Diam.					
	29.70 - 29.91	122.8	51.5		25.7										
BH4	29.59 - 29.73	34.9	31.8		15.9						2.95	0.54	4.76	1.62	
	30.00 - 30.71								8.33	Diam.	2.73	0.15	1.20	0.44	
	38.00 - 38.27														
	38.35 - 38.60										2.76	0.06	0.63	0.23	
	38.79 - 39.00	58.5	30.1		15.0				4.01	Diam.					
	39.74 - 40.00								7.90	Diam.					
	40.00 - 40.30										2.75	0.34	1.40	0.51	
	40.35 - 40.60							17.13							
	42.27 - 4[illegible].47								6.78	Diam.					
	42.69 - 43.00	52.2	51.6		25.8										
	44.51 - 44.71								9.38	Diam.					
	44.75 - 45.00	69.7	28.2		14.1										
	45.57 - 45.80														
	47.59 - 47.89	92.8	54.9		27.4			8.67	10.75	Diam.					

Owing to the heterogenous composition of the overburden, it is presumed that the permeability of the overburden is relatively high. To capsule these permeable openings, grout curtain is recommended as shown in Figure 5-15. The minimum depth of the grout curtain should not be less than 2/3 of the height of the dam (2/3 x45 =30 m) or the thickness of the overburden. This however needs to be verified during the time of construction based on the test grouting and permeability tests at these sections. In case of high permeability of the rock, second row of grout curtain should be applied on the same plane as the first one.

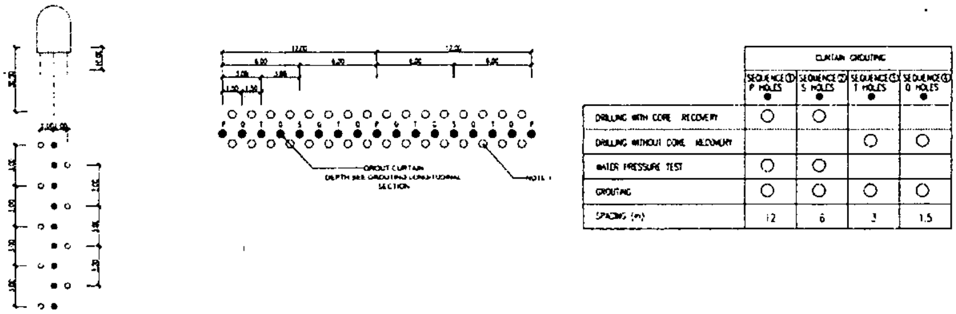


Figure 5-15: Recommended grout curtain pattern (3 rows)

5.3.7.3 Engineering conditions of Diversion Tunnel

Proposed portal lies on rocky slope facing toward North-east. Hill slope ranges 30°–70°. The portal and diversion tunnel consist of mostly the rock type quartz mica gneiss. The rock is light gray to dark gray, medium-banded to massive, strong and moderately weathered. The quartz gneiss is slightly weathered and massive with random joints and fractures (Figure 5-16). The quartz mica gneiss measured shows a foliation and in general three joint sets. The general trend of these sets is given in the following Table 5-8.

Figure 5-16: A view joint pattern and foliation trend of Quartz Mica Gneiss

Table 5-8: General trend of attitude measured at different sections along the diversion tunnel are tabulated as follows

	Strike	Dip
Foliation	N50E to N70E	30NW to 80 NW
Joint J1	N45W to N80W N40E to N70E	25NE to 50NE 30SW to 45SW
Joint J2	N50W to N75W N35E to N60E	45SW to 50SW 40SE to 55SE
Joint J3	N50W to N30W	70NE to 80NE

Stereographic projection of major joint sets measured in the inlet portal/ diversion tunnel is shown in Figure 5-17. Orientation of major joint i.e. foliation is nearly parallel to the tunnel axis and dips more than 45° which is an unfavorable condition on excavation. Assuming friction angle of 40° for rock, the stereographic projection shows 6 day-light wedges w1 to w6 with only one wedge w4 between foliation F and Joint J3 falling within the failure envelop. This wedge may trigger failure during excavation.

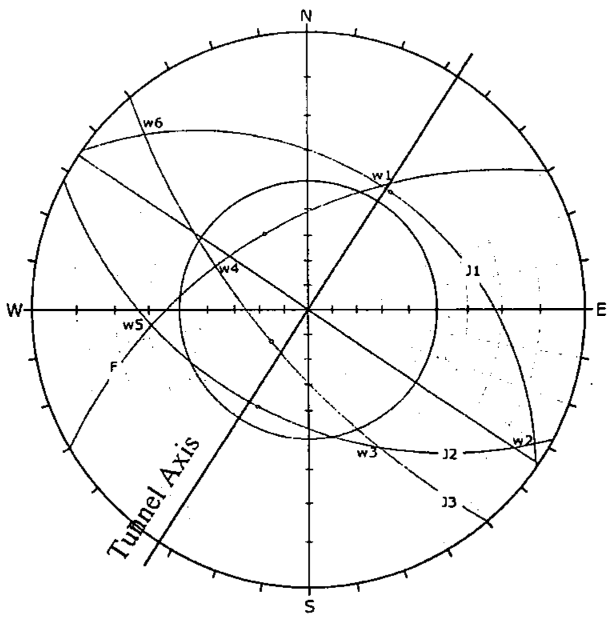


Figure 5-17: Stereographic projection of discontinuities measured at the headrace tunnel inlet portal

During geotechnical investigation along the diversion tunnel 3 boreholes (BH1, BH9 and BH10) with varying between 55 m and 27 m were drilled.

In total 15 Lugeon tests were carried out in BH1 and BH11 at the different depths. The result of the tests is summarized in the following Table 5-9. The computed Lugeon values were all less than 1 and the flow regime as turbulent. this as per Houbsley indicate the hydraulic conductivity of the rock mass decreases as the water pressure increases. This behavior is characteristic of rock masses exhibiting partly open to moderately wide cracks.

The discontinuity information recorded at site and in the boreholes 1 and 10 showed in general joints having partly open to very tight aperture (between 0.5 mm-<0.1mm) with quartz or clayey matrix and roughness varying between undulating rough to planar smooth.

The Laboratory investigation results on the samples obtained from BH 1 and BH10 are presented in

Table 5-9

Borehole Number	Samples Depth (m)	UCS σ_c (MPa)	UCS E (GPa)	UCS ν (-)	UCS Deformation (GPa)	Shear Strength φ (°)	Shear Strength c (MPa)	Brasilian Test (Tensile Strength) σ_t (MPa)	Pointload Index $I_{s,50}$ (Mpa)	Pointload Index Remarks	Density ρ (g/cm³)	Water content w_n (%)	Porosity n (%)	Water absporption w_a (%)	Slake durability index s (%)
BH-1	10.76 - 11.00										2.70	0.15	1.38	0.51	99.5
	11.00 - 11.36								9.67	Diam.					
	13.80 - 14.10								9.22	Diam.					
	14.16 - 14.50	150.4	49.2		24.6										
	16.67 - 16.87								2.40	Diam.	2.79	0.21	1.15	0.41	98.2
	18.96 - 19.22								3.95	Diam.					
	19.50 - 19.75										2.73	0.10	1.64	0.60	98.7
	23.76 - 24.13								1.98	Diam.					
	24.24 - 24.38								1.54	Diam.					
	25.96 - 26.17								11.19	Diam.					
	26.17 - 26.50								11.19	Diam.	2.69	0.08	1.12	0.42	99.6
	27.60 - 27.92	47.3	62.5		31.2			8.93							
	45.13 - 45.42								6.58	Diam.					
	47.18 - 47.44	150.4	62.1		31.1										
	48.00 - 48.50										2.69	0.08	1.57	0.58	99.6
	49.61 - 49.91	126.2	92.7		46.3										
	51.12 - 51.31								4.17	Diam.					
	52.60 - 52.84	92.8	48.8		24.4										
	52.60 - 52.84										2.78	0.08	1.41	0.51	99.3
	54.64 - 54.85														
BH9	10.00 - 10.25								10.97	Diam.					
	20.41 - 20.62										2.76	0.37	2.58	0.94	99.5
	24.27 - 24.50	21.6	63.1		31.6										
	25.70 - 25.91										2.73	0.13	0.67	0.26	99.6
	31.62 - 31.93														
	32.06 - 32.30								12.51	Diam.					
	33.13 - 33.48					50.1	2.4	17.80							
	36.39 - 36.59								11.85	Diam.					
	39.00 - 39.19										2.72	0.12	1.70	0.63	99.4
	41.32 - 41.49								8.12	Diam.					
	42.69 - 43.00					49.5	0.1	20.44							
	44.00 - 44.20										2.72	0.19	2.02	0.74	99.3
	47.14 - 47.32								8.12	Diam.					
	48.69 - 48.90								2.46	Diam.					
	50.00 - 50.27	73.0	51.0		25.5										
	51.81 52.00										2.69	0.16	2.62	0.98	99.4
	52.85 53.00								5.93	Diam.					
	53.68 54.00	119.0	46.3		23.2										
	54.80 55.00										2.80	0.19	3.12	1.12	99.4
BH10	4.45 - 4.59								11.41	Diam.					
	6.11 - 6.25								1.76	Diam.					
	6.31 - 6.50										2.77	0.47	1.65	0.60	98.6
	6.88 - 7.00										2.97	0.49	0.96	0.33	98.2
	10.40 - 10.60	34.0	16.1		80.6										
	11.78 - 11.95	30.0	26.8		13.4										
	13.50 - 13.62								1.76	Diam.					
	20.30 - 20.44								5.27	Diam.	2.75	0.44	3.13	1.15	98.5
	20.44 - 20.53								9.66	Diam.					
	21.72 - 22.00	28.2	43.5		21.8										
	22.22 - 22.44								7.24	Diam.	2.82	0.36	1.75	0.63	99.4
	23.00 - 23.22	18.5	99.7		49.8				68.00	Diam.					
	24.30 - 24.62					32.2	4.4	11.23							
	29.65 - 29.70										2.79	0.76	0.85	0.31	99.0
	29.70 - 30.00					43.1	1.4	13.53							

Table 5-10.

Table 5-9: Lugeon test results BH1 and BH10

Bore Hole	Tiefe (m)	Lugeon	hydraulic conductivitiy (Fell et. al., 2005) (m/s)	Interpretation (T. Houlsby 1976)	Classification	Rock discontinuity condition
BH1						
	5 - 10	0.02	< 1E-07	Turbulent Flow	very low	very tight
	10 - 15	0.01	< 1E-07	Turbulent Flow	very low	very tight
	15 - 20	0.00	< 1E-07	Turbulent Flow	very low	very tight
	20 - 25	0.00	< 1E-07	Void Filling	very low	very tight
	25 - 30	0.00	< 1E-07	Turbulent Flow	very low	very tight
	30 - 35	0.00	< 1E-07	Turbulent Flow	very low	very tight
	35 - 40	0.00	< 1E-07	Turbulent Flow	very low	very tight
	40 - 45	0.00	< 1E-07	Turbulent Flow	very low	very tight
	45 - 50	0.00	< 1E-07	Void Filling	very low	very tight
	50 55	0.00	< 1E-07	Void Filling	very low	very tight
BH10						
	5 - 10	0.03	< 1E-07	Turbulent Flow	very low	very tight
	10 - 15	0.01	< 1E-07	Turbulent Flow	very low	very tight
	15 - 20	0.00	< 1E-07	Turbulent Flow	very low	very tight
	20 - 25	0.00	< 1E-07	Turbulent Flow	very low	very tight
	25 - 30	0.01	< 1E-07	Turbulent Flow	very low	very tight

In general, the laboratory test of the rock showed the physical and mechanical parameters with unit weight: 27-29 kN/m³, internal angle of friction: 32-50°, cohesion: 0.1-4.4 MPa, unconfined compressive strength:12-73 MPa, tensile strength: 11 - 20 Mpa, point load strength: 1.7-12.5 MPa, young's modulus: 30 GPa to 60 GPa, slake durability > 99%. (very high durability, Gamble chart) and water absorption: 0.5- 1.6%.

Borehole Number	Samples Depth (m)	UCS				Shear Strength		Brasilian Test (Tensile Strength)	Pointload Index		Density	Water content	Porosity	Water absporption	Slake durability index
		σ_c (MPa)	E (GPa)	v (-)	Deformation (GPa)	φ (°)	c (MPa)	σ_t (MPa)	$I_{s,50}$ (Mpa)	Remarks	ρ (g/cm³)	w_n (%)	n (%)	w_a (%)	s (%)
BH-1	10.76 - 11.00										2.70	0.15	1.38	0.51	99.5
	11.00 - 11.36								9.67	Diam.					
	13.80 - 14.10								9.22	Diam.					
	14.16 - 14.50	150.4	49.2		24.6										
	16.67 - 16.87								2.40	Diam.	2.79	0.21	1.15	0.41	98.2
	18.96 - 19.22								3.95	Diam.					
	19.50 - 19.75										2.73	0.10	1.64	0.60	98.7
	23.76 - 24.13								1.98	Diam.					
	24.24 - 24.38								1.54	Diam.					
	25.96 - 26.17								11.19	Diam.					
	26.17 - 26.50								11.19	Diam.	2.69	0.08	1.12	0.42	99.6
	27.60 - 27.92	47.3	62.5		31.2			8.93							
	45.13 - 45.42								6.58	Diam.					
	47.18 - 47.44	150.4	62.1		31.1										
	48.00 - 48.50										2.69	0.08	1.57	0.58	99.6
	49.61 - 49.91	126.2	92.7		46.3										
	51.12 - 51.31								4.17	Diam.					
	52.60 - 52.84	92.8	48.8		24.4										
	52.60 - 52.84										2.78	0.08	1.41	0.51	99.3
	54.64 - 54.85														
BH9	10.00 - 10.25								10.97	Diam.					
	20.41 - 20.62										2.76	0.37	2.58	0.94	99.5
	24.27 - 24.50	21.6	63.1		31.6										
	25.70 - 25.91										2.73	0.13	0.67	0.26	99.6
	31.62 - 31.93														
	32.06 - 32.30								12.51	Diam.					
	33.13 - 33.48					50.1	2.4	17.80							
	36.39 - 36.59								11.85	Diam.					
	39.00 - 39.19										2.72	0.12	1.70	0.63	99.4
	41.32 - 41.49								8.12	Diam.					
	42.69 - 43.00					49.5	0.1	20.44							
	44.00 - 44.20										2.72	0.19	2.02	0.74	99.3
	47.14 - 47.32								8.12	Diam.					
	48.69 - 48.90								2.46	Diam.					
	50.00 - 50.27	73.0	51.0		25.5										
	51.81 52.00										2.69	0.16	2.62	0.98	99.4
	52.85 53.00								5.93	Diam.					
	53.68 54.00	119.0	46.3		23.2										
	54.80 55.00										2.80	0.19	3.12	1.12	99.4
BH10	4.45 - 4.59								11.41	Diam.					
	6.11 - 6.25								1.76	Diam.					
	6.31 - 6.50										2.77	0.47	1.65	0.60	98.6
	6.88 - 7.00										2.97	0.49	0.96	0.33	98.2
	10.40 - 10.60	34.0	16.1		80.6										
	11.78 - 11.95	30.0	26.8		13.4										
	13.50 - 13.62								1.76	Diam.					
	20.30 - 20.44								5.27	Diam.	2.75	0.44	3.13	1.15	98.5
	20.44 - 20.53								9.66	Diam.					
	21.72 - 22.00	28.2	43.5		21.8										
	22.22 - 22.44								7.24	Diam.	2.82	0.36	1.75	0.63	99.4
	23.00 - 23.22	18.5	99.7		49.8				68.00	Diam.					
	24.30 - 24.62					32.2	4.4	11.23							
	29.65 - 29.70										2.79	0.76	0.85	0.31	99.0
	29.70 - 30.00					43.1	1.4	13.53							

Table 5-10: Laboratory results on samples of BH 1 and BH10

calculated RQD, RMR, Q and GSI values of the rocks encountered at BH 1, BH9 and BH10 are presented in
Table 5-11.

Table 5-11: RQD and Q-value computation BH1, BH9 and BH10

BH1 Depth (m)	RQD	Jn	Jr	Ja	Jw	SRF	RQD ÷ Jn	Jr ÷ Ja	Jw ÷ SRF	Q*	Q	RMR	GSI =RMR-5	Rock Quality based on Q-values
3 - 5.5	7.86	6	1	4	1	2	1.31	0.25	0.5	0.33	0.16	28	23	very Poor
5.5 - 15	45.54	6	1	4	0.6	2	7.59	0.25	0.3	1.90	0.57	39	34	very Poor
15 - 18.5	15.42	6	1.5	1.5	0.33	1	2.57	1	0.33	2.57	0.85	43	38	very Poor
18.5 - 30.5	40.47	4	1.5	1	0.33	1	10.12	1.5	0.33	15.18	5.01	58	53	Good
30.5 - 37	14.83	4	3	1	0.33	1	3.71	3	0.33	11.12	3.67	56	51	Good
37 - 45	21.75	4	3	0.75	0.33	1	5.44	4	0.33	21.75	7.18	62	57	Good
45 - 55	37.6	6	3	0.75	0.33	1	6.27	4	0.33	25.07	8.27	63	58	Good

BH9 Tiefe (m)	RQD	Jn	Jr	Ja	Jw	SRF	RQD ÷ Jn	Jr ÷ Ja	Jw ÷ SRF	Q*	Q	RMR	GSI =RMR-5	Rock Quality based on Q-values
10 - 20	8	6	1	2	0.66	1.5	1.33	0.5	0.44	0.67	0.29	33	28	very Poor
20 - 25	14.6	6	1	1	0.33	1.5	2.43	1	0.22	2.43	0.54	38	33	very Poor
25 - 30	8	6	1	1	0.33	1.5	1.33	1	0.22	1.33	0.29	33	28	very Poor
30 - 35	37.6	6	1	1	0.33	1.5	6.27	1	0.22	6.27	1.38	47	42	Poor
35 - 40	35.5	6	2	1	0.33	1.5	5.92	2	0.22	11.83	2.60	53	48	Poor
40 - 45	47	6	2	1	0.33	1.5	7.83	2	0.22	15.67	3.45	55	50	Poor
45 - 50	19	6	2	1	0.33	1.5	3.17	2	0.22	6.33	1.39	47	42	Poor
50 - 55	20	4	2	1	0.33	1	5.00	2	0.33	10.00	3.30	55	50	Good

BH10 Depth (m)	RQD	Jn	Jr	Ja	Jw	SRF	RQD ÷ Jn	Jr ÷ Ja	Jw ÷ SRF	Q*	Q	RMR	GSI =RMR-5	Rock Quality based on Q-values
4 - 10	20	6	1	1	0.66	1.5	3.33	1	0.44	3.33	1.47	47	42	Poor
10 - 15	27	6	1	1	0.66	1.5	4.50	1	0.44	4.50	1.98	50	.45	Poor
15 - 20	10	6	1	1	0.33	1.5	1.67	1	0.22	1.67	0.37	35	30	very Poor
20 - 25	53	4	1	1	0.33	1	13.25	1	0.33	13.25	4.37	57	52	Poor
25 - 30	10	6	1	1	0.33	1	1.67	1	0.33	1.67	0.55	39	34	very Poor

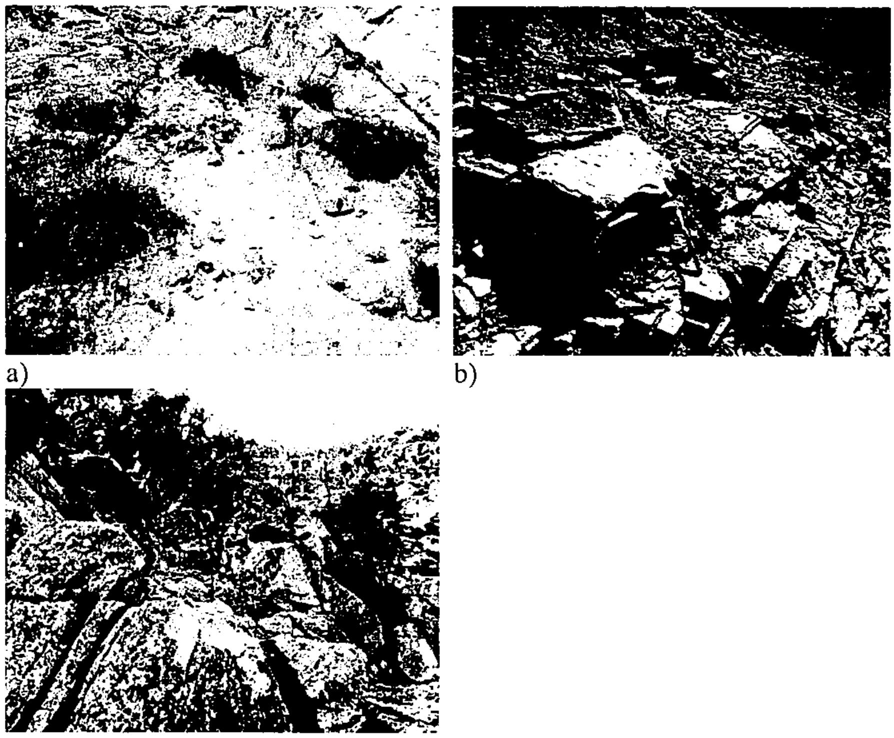
RQD value calculated based on boreholes core recovery at site at the depth of planned tunnel vary between 10% and 37%.

5.3.7.4 Engineering conditions of Headrace Tunnel

The proposed intake tunnel alignment passes through the rocky hills as shown in the Figure 5-19. The total length of the intake tunnel is about 8 km and passes through several geological formations. Based on the field investigation and literature review, the tunnel section comprises of the different lithological units namely Tonalite, granite and quartz mica gneiss. The brief summary of the characteristic of these rock formations measured and recorded are shown in Table 5-12 and some of the outcrops photos are shown in Figure 5-18.

Table 5-12: Characteristic features of rock at tunnel alignment

Rock type/ Soil units	Characteristic	Foliation	Joint Set
Tunnel Alignment			
Granite	half white in color, medium to coarse grained, strong to very strong, slightly to moderately weathered, Medium to widely jointed and also closely to medium jointed at places, making a sharp contact with Quartz mica Gneiss. Granite is exposed in the form of patches mostly from Sino hydro dam towards tunnel alignment.		J1 N40E to N70E / 40 to 75NW N30W / 50NE J2 N60W to N70W/ 60 to 85SW J3 N10E to N30E/ 60 to 80SE
Quartz Mica Gneiss	light grey to dark grey on fresh surface, weathers to rusty brown to yellowish Brown in color, fine to medium grained, strong to very strong, slightly to moderately weathered, Medium to widely jointed and also closely to medium jointed at places.	N40E to N60E/ 25 to 60 NW	J1 N20E to N80E/ 30 to 75 SE dip, N50W/ 25 to 40SW N50E to N60E/ 25 to 55NE J2 N30W to N80W / 60SW, N 0E, to N60E. 20 SE to 60 SE and N20W to N80W / 30 to 60 NE J3 N05W / 80NE dip and N30W to 80SW
Tonalite	grayish brown to light grey, slightly to moderately weathered, joints aperture is moderate wide to open filled with clayey matrix, weak to medium weak surface staining is also seen at some places, with smooth planer surfaces and persistence is low to medium.		J1 N40E to N60E/ 40SE to 75SE J2 N30W to N50W/ 30SW to 60SW J3 N20W to N70W/ 30NEto 70NE



a) b) c)

Figure 5-18: Rock outcrops along the Tunnel a) Granite b) Quartz mica gneiss and c) Tonalite

Furthermore, about 3 km of the tunnel passes through the rock portion which comprises of metamorphic origin namely quartz mica gneiss. The rest of the alignment up to the tailrace passes through igneous formati on Tonalite (see Figure 5-19).

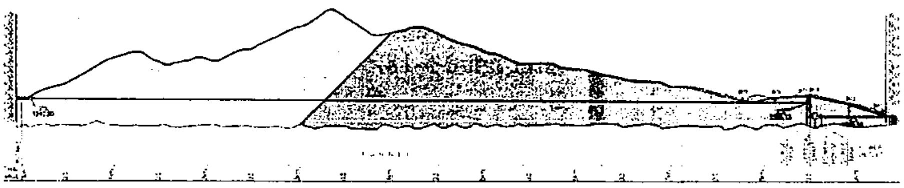


Figure 5-19: Section through the intake tunnel, surge tank, PH and tailrace

The Hill slope range of the tunnel area lie between 15°–37°. Owing to the security reason and difficult terrain with thick forests, the portion of the tunnel part from end of intake diversion tunnel up to about 7.8 km, no any geological or geotechnical/geophysical ground investigation could be carried out in this part. The details presented herein are solely based on the extrapolation of the available data from geological/geotechnical and geophysical investigation at intake, surge tank and tailrace including the available geological literature and geological measurements on the outcrops of the foothills. Therefore, for better and precise understanding of the geology at this part, detail investigation during the construction or detail design phase needs to be foreseen.

It is presumed that the overburden thickness for this part of the tunnel is about 8 m-10m and is underlain by quartz mica Gneiss. The total depth of the tunnel in this part measured from the surface of the hill vary between 35 m and 975 m.

The quartz mica Gneiss along the tunnel alignment is light grey to dark grey on fresh surface, weathers to rusty brown to yellowish brown in color, fine to medium grained, strong to very strong, slightly to moderately weathered, medium to widely jointed and closely to medium jointed at places.

The general attitude information taken from available data and measured at different accessible locations in the vicinity of the tunnel alignment are tabulated as follows:

Table 5-13: General attitude data of quartz mica Gneiss at tunnel section

	Strike	Dip
Foliation	N40E to N60E	25NW to 60 NW
Joint J1	N20E to N80E N50E to N60E	30SE to 75 SE 25SE to 55SE
Joint J2	N30W to N80W N30E to N60E	20SW to 60SW 20NE to 80NE
Joint J3	N5W to N30W	30NE to 80SW

Stereographic projection of major joint sets measured along the tunnel alignment is shown in Figure 5-20.

The Orientation of major joint i.e. foliation is nearly parallel to the tunnel axis and dips more than 45° which is a very unfavorable condition on excavation. Assuming friction angle of 40° for the quartz mica Gneiss, the stereographic projection shows 4 day-light wedges w1 to w4, with 2 wedges (w1 and w2) lying within the failure envelop. These wedges may trigger failure during excavation.

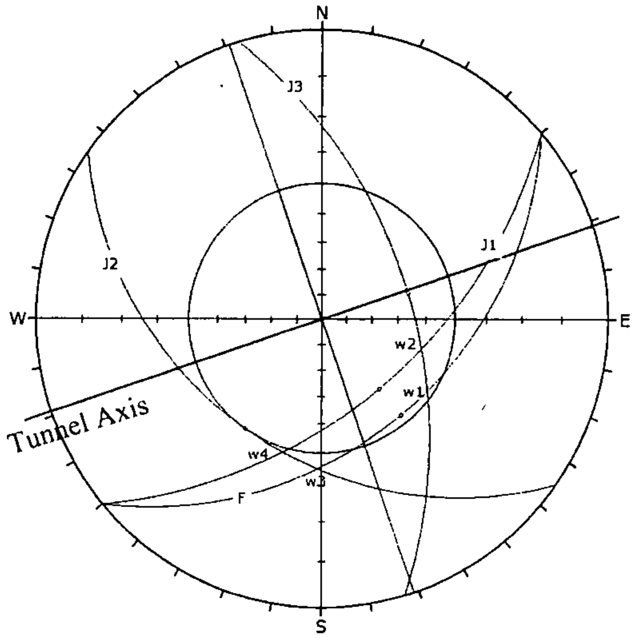


Figure 5-20: Stereographic projection of Tonalite discontinuities measured at the headrace tunnel inlet portal.

From mid of tunnel alignment area towards the surge tank area Tonalite is exposed. Tonalite is grayish brown to light grey, slightly to moderately weathered, joints aperture is moderate wide to open filled with clayey matrix, weak to medium weak surface staining is also seen at some places, with smooth planer surfaces and persistence is low to medium. The general trend of joints measured on exposed outcrop are presented in the following Table 5-14:

Table 5-14: General attitude data of Tonalite at tunnel section

	Strike	Dip
Joint J1	N40E to N60E	40SE to 75SE
Joint J2	N30W to N50W	30SW to 60SW
Joint J3	N20W to N70W	30NE to 70NE

Stereographic projection of major joint sets measured along the tunnel alignment is shown in Figure 5-21.

Assuming friction angle of 40° for the Tonalite rock, the stereographic projection shows 3 day-light wedges w1, w2 to w3. All these wedges fall outside the failure envelop although wedges w1 and w2 lie close to the failure envelop.

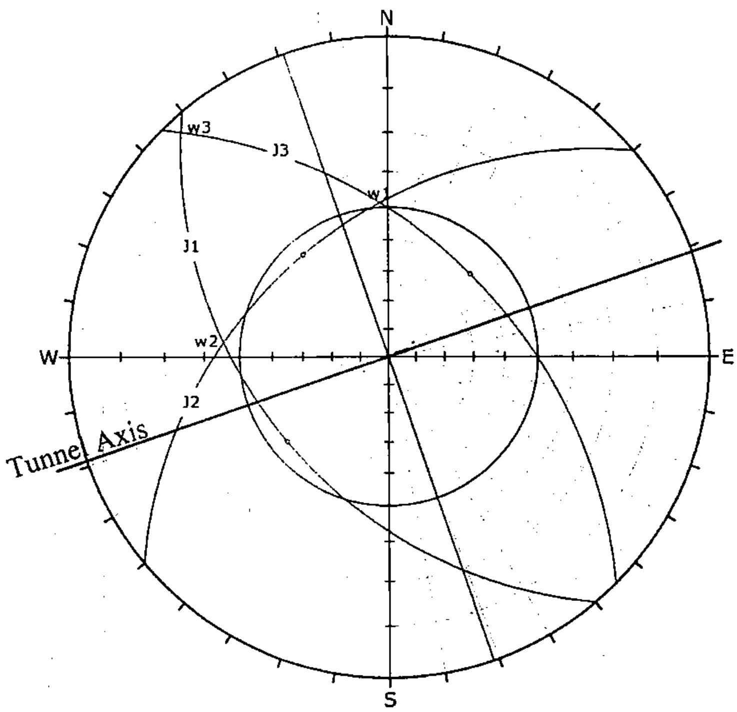


Figure 5-21: Stereographic projection of discontinuities measured at the headrace tunnel inlet portal

Along the intake tunnel close to surge tank area, during the course of geotechnical/geological investigation, 2 boreholes (BH16 and BH19) with depth up to 55 m and 27 m were drilled. In these boreholes, altogether 7 Lugeon tests were carried out in BH16 and BH19 at the different depths as shown in Table 5-15.

Table 5-15: Summary of Lugeon test at BH16 and BH19

Bore Hole	Tiefe (m)	Lugeon	hydraulic conductivitiy (Fell et. al., 2005) (m/s)	Interpretation (T. Houlsby 1976)	Classification	Rock discontinuity condition
BH16						
	40 - 45	0.00	< 1E-07	Turbulent Flow	very low	very tight
	45 - 50	0.00	< 1E-07	Turbulent Flow	very low	very tight
	50 - 55	0.00	< 1E-07	Turbulent Flow	very low	very tight
BH19						
	32 - 35	0.01	< 1E-07	Turbulent Flow	very low	very tight
	40 - 45	0.00	< 1E-07	Turbulent Flow	very low	very tight
	45 - 50	0.00	< 1E-07	Turbulent Flow	very low	very tight
	50 - 55	0.00	< 1E-07	Turbulent Flow	very low	very tight

The computed Lugeon were all less than 1, which indicates very low permeability of the rock and the corresponding rock discontinuity to be very tight. Apart from this, the pressure flow (p-q) diagram indicated most of the tests to lie in the range of turbulent flow region. As per Houlsby's (1976) interpretation, this indicates that the hydraulic conductivity of the rock mass decreases as the water pressure increases, which characterize the behavior of rock masses exhibiting partly open to moderately wide cracks.

The discontinuity information recorded at site showed in general joints having partly open to very tight aperture (between 0.5 mm-<0.1mm) with quartz or clayey matrix and roughness varying between undulating rough to planar smooth.

On the samples obtained from BH16 and BH19 several laboratory tests were carried out. The summary of the tests are shown in
Table 5-16.

Table 5-16: Summary of Lab tests at BH16 and BH19

Borehole Number	Samples Depth (m)	UCS				Shear Strength		Brasilian Test (Tensile Strength)	Pointload Index		Density	Water content	Porosity	Water absorption	Slake durability index
		σ_c (MPa)	E (GPa)	ν (-)	Deformation (GPa)	φ (°)	c (MPa)	σ_t (MPa)	$I_{s,50}$ (Mpa)	Remarks	ρ (g/cm³)	w_n (%)	n (%)	w_a (%)	s (%)
BH16	15.23 - 15.47										2.65	0.29	1.79	0.68	
	21.48 - 21.68										2.72	0.60	1.87	0.69	
	28.76 - 29.00										2.67	0.32	1.89	0.71	
	34.00 - 34.17										2.68	2.09	6.16	2.35	
	40.00 - 40.20										2.73	0.40	1.30	0.48	
	45.15 - 45.39										2.72	0.49	1.60	0.59	
	52.75 - 53.00										2.69	0.85	2.47	0.92	
	54.76 - 55.00										2.75	0.93	3.39	1.24	
	12.28 - 12.53								11.67	Diam.					
	18.69 - 18.92								9.03	Diam.					
	26.46 - 26.63								9.94	Diam.					
	31.49 - 31.69								4.40	Diam.					
	37.44 - 37.68								7.47	Diam.					
	46.34 - 46.55								10.75	Diam.					
	51.76 - 52.00								9.88	Diam.					
	54.10 - 54.34								10.31	Diam.					
	11.76 - 12.00	158.0	65.3		32.6										
	17.48 - 17.76	94.0	60.0		30.0										
	23.19 - 23.42	96.0	78.2		39.1										
	33.52 - 33.85	95.0	100.0		50.1										
	38.19 - 38.44	97.0	53.0		26.5										
	41.48 - 41.80	106.0	52.1		26.1										
	49.06 - 49.39							11.52							
	53.46 - 53.71					44.9	4.2	12.38							
BH19	34.14 - 34.31								7.97	Diam.					
	35.19 - 35.48	68.0	37.0		18.5										
	36.28 - 36.49	66.0	75.4		37.7						2.71	0.62	2.07	0.8	
	37.64 - 37.82								9.22	Diam.					
	41.87 - 42.00	106.0	88.3		44.2						2.84	0.92	3.25	1.15	
	42.00 - 42.40	49.0	50.1		25.1										
	44.00 - 44.25					44.8	3.3	8.06							
	45.00 - 45.19								9.66	Diam.					
	47.08 - 47.31	176.0	43.0		21.5						2.74	0.30	1.37	0.50	
	49.11 - 49.32								9.38	Diam.					
	50.04 - 50.19	108.0	51.5		25.7						2.68	0.65	2.16	0.81	
	51.70 - 52.00					47.4	3.5	17.56							
	53.46 - 53.63	126.0	56.4		28.2										
	53.63 - 53.90					45.8	0.7	14.68			2.74	0.45	2.02	0.74	
	54.07 - 54.29	155.0	63.1		31.6										

Laboratory test of the rock showed the physical and mechanical parameters with unit weight: 27-28 kN/m³, internal angle of friction: 44.8°-47.8°, cohesion: 0.7-4.2 MPa, unconfined compressive strength:49-158 MPa, tensile strength: 8 – 17.5 MPa, point load strength: 4.4-9.3 MPa, young's modulus: 10000 MPa to 80000 Mpa, slake durability > 99% (very high durability, Gamble chart) and water absorption: 0.5- 2.35%

Based on the drilling and borelogs, RQD, RMR, Q and GSI values were measured/ calculated on these boreholes, the summary of these computation is presented in the following Table 5-17:

Table 5-17: RQD, RMR, Q-value and GSI calculation, BH16 and BH19

BH16 Depth (m)	RQD	Jn	Jr	Ja	Jw	SRF	RQD + Jn	Jr + Ja	Jw + SRF	Q*	Q	RMR	GSI =RMR-5	Rock Quality based on Q-values
7.5 - 11	6.28	6	1.5	1	0.66	1.5	1.05	1.5	0.44	1.57	0.69	41	36	very Poor
11 - 20	58.33	6	1.5	1	0.66	1.5	9.72	1.5	0.44	14.58	6.42	61	56	Fair
20 - 30	39.4	4	1	1	0.33	1	9.85	1	0.33	9.85	3.25	55	50	Poor
30 - 40	47.3	4	1	1	0.33	1	11.83	1	0.33	11.83	3.90	56	51	Poor
40 - 50	13.6	4	1	0.75	0.33	1	3.40	1.3	0.33	4.53	1.50	48	43	Poor
50 - 55	61.2	4	1.5	0.75	0.33	1	15.30	2	0.33	30.60	10.10	65	60	Good

BH19 Depth (m)	RQD	Jn	Jr	Ja	Jw	SRF	RQD + Jn	Jr + Ja	Jw + SRF	Q*	Q	RMR	GSI =RMR-5	Rock Quality based on Q-values
32 - 35	37	6	1	2	0.33	1.5	6.17	0.5	0.22	3.08	0.68	41	36	very Poor
35 - 40	60	4	1	1	0.33	1	15.00	1	0.33	15.00	4.95	58	53	Poor
40 - 45	92	3	1.5	1	0.33	1	30.67	1.5	0.33	46.00	15.18	68	63	Good
45 - 50	78	3	1.5	1	0.33	1	26.00	1.5	0.33	39.00	12.87	67	62	Good
50 - 55	86	3	1.5	1	0.33	1	28.67	1.5	0.33	43.00	14.19	68	63	Good

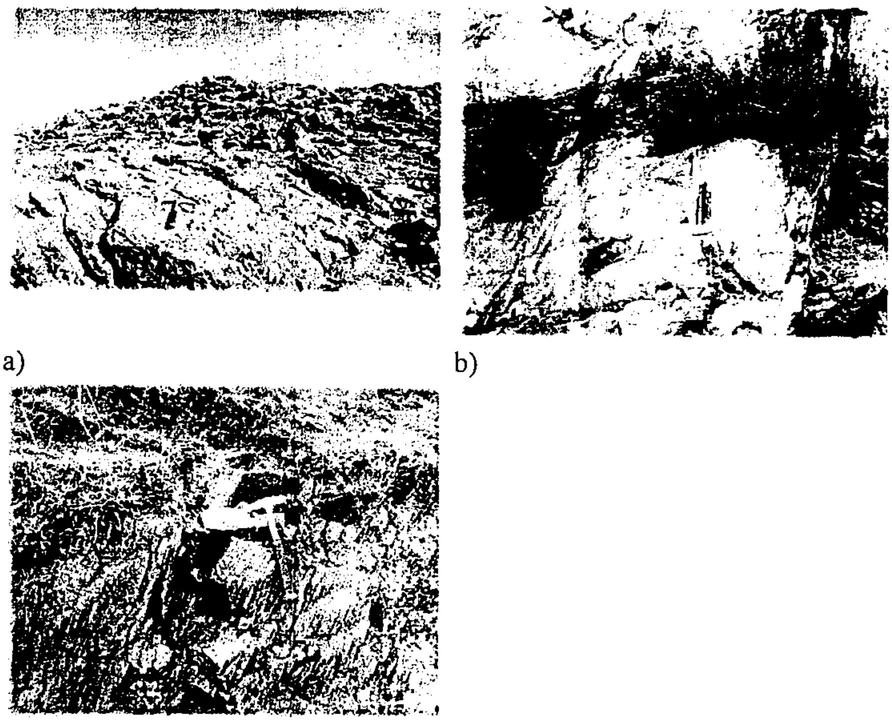
The measured RQD values lie between 82%-88% and Q-values between 10 and 14 respectively. Value of RQD sand Q values suggests good rock quality.

5.3.7.5 Engineering geological conditions of surge tank and vertical pressure shaft

The characteristics of the rock with measured structural attitude at the surge tank, powerhouse and outlet area are given in the following Table 5-18 and the photos in Figure 5-22 :

Table 5-18: Characteristics of rocks at surge tank, powerhouse and outlet area

Rock type/ Soil units	Characteristic	Foliation	Joint Set
Surge Tank			
Tonalite	grayish brown to light grey, moderately to highly weathered, joints aperture is moderate wide to open filled with clayey matrix, weak to medium weak surface staining is also seen at some places, with smooth planer surfaces and persistence is low to medium.		J1 N10E to N70E, N30W to N45W// 10NW to 35NW and N50E to N60E J2 N30E to N50E / 40SE to 60SE J3 N40W to N70W / 35SW to 55SW
Epidosite	greyish brown to brownish grey, moderately weathered, joints are moderately to widely open , filled with clay, surface staining is also seen at some places, with smooth planer surfaces and persistence is medium to high.	N50W/ 20SW	J1 N10E/ 70NW J2 N60E/ 70 NW J3 N40W/ 50NE
Quartz Mica Epidote Gneiss	is greenish grey to brownish grey, slightly to moderately weathered, joints are moderately open to, filled with clayey matrix, surface staining is also seen at some places, with smooth planer surfaces and persistence is medium to high.	N60E to N70E/ 10NW to 80NW	J1 N10W/ 80NE J2 N10W/50SW
Powerhouse and outlet area			
Para Amphibolite	greyish brown to brownish grey, moderately weathered, joints are moderately to widely open , filled with clay, surface staining is also seen at some places, with smooth planer surfaces and persistence is medium to high.	N70E/ 60SE to 70SE	J1 N30W to N80W/ 70SW to 80SW J2 N10E to N30E/ 70NW to 80NW
Tonalite	grayish brown to light grey, slightly to moderately weathered, joints aperture is moderate wide to open filled with clayey matrix, weak to medium weak surface staining is also seen at some places, with smooth planer surfaces and persistence is low to medium.		J1 N40E to N60E/ 40SE to 75SE J2 N30W to N50W/ 30SW to 60SW J3 N20W to N70W/ 30NE to 70NE
Granite	smoky light in color, medium to coarse grained, strong to very strong, slightly to moderately weathered, Medium to widely jointed and also closely to medium jointed at places, making a sharp contact with Quartz mica Gneiss. Granite making a transitional contact with Tonalite at frontal part of the outlet area.		J1 N10E/ 60SE J2 N70W/ 30NE J3 N80W/ 80SW



a) b)

c)

Figure 5-22: Rock outcrops at Surge tank Area a) Tonalite b) Epidosite and c) Quartz Mica Epidote Gneiss

The surge tank is a simple cylindrical type with 1.9 m diameter and the vertical shaft with 3 m diameter. Surface mapping and drilling revealed thick overburden 12 m to 13 m thick on hill slope in the area. The hill slope ranges 10°–15°. The total vertical length of the surge tank from the surface/ crown is 70 m. The total length of the vertical shaft is around 150 m with the total overburden varying between 90m and 250m.

The surface mapping showed presence of light gray Tonalite, small patches of greyish brown Epidosite, greenish grey Epidote and Quartz Mica Epidote Gneiss in the vicinity of the surge tank. These rock masses are slightly to moderately weathered having moderately open to tight joint with clayey matrix infilling, smooth planer surface and medium to high persistence. The details are given in the geological surface mapping. The major rock type encountered at the Surge tank area is Tonalite. The general trend of the attitude for Tonalite is given in the following Table 5-19 and the stereographic projection is shown in Figure 5-23.

Table 5-19: General attitude of Tonalite in Surge tank area

	Strike	Dip
Joint J1	N10E to N70E N30W to N45W	10NW to 35NW 50NE to 60NE
Joint J2	N30E to N50E	40SE to 60 SE
Joint J3	N40W to N70W	35SW to 55SW

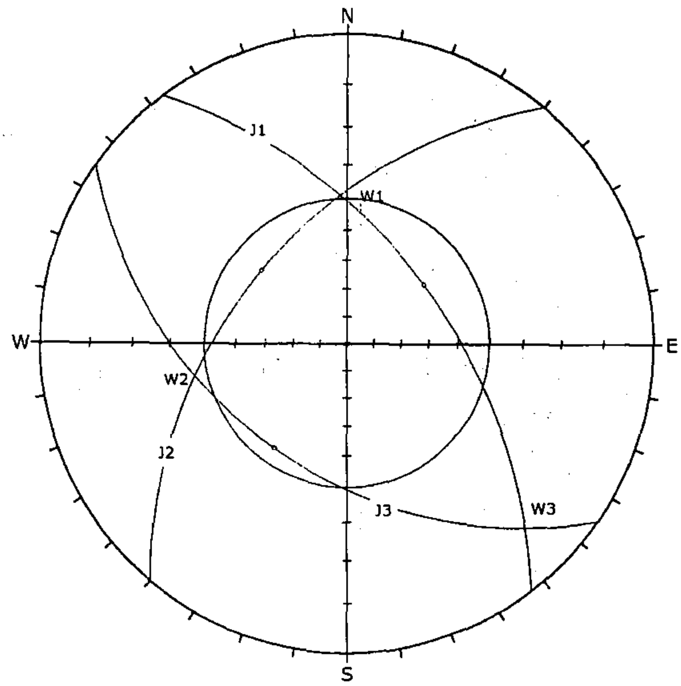


Figure 5-23: Stereographic projection and kinematic analysis of discontinuities measured in the surge tank

Stereographic projection of major joint sets obtained from statistical analysis of joints is shown in Figure 5-23. Stereographic diagram shows distinct three discontinuity wedges w1, w2 and w3. Assuming friction angle of 40° for rock, all the wedges fall outside the failure envelop.

During geotechnical investigation phase two borehole (BH14 and BH15) with depth from 55 m and 250 m was drilled at the location of surge tank, power house and vertical shaft. The major rock type encountered in these boreholes was Tonalite.

In total 12 Lugeon tests were carried out in these boreholes at different depths from 305m up to 245m. The summary of the test is presented in Figure 5-22. The computed Lugeon were all less than 1, which indicates very low permeability of the rock mass. The computed Lugeon values were all less than 1 and the flow regime as turbulent. this as per Houbsley indicate the hydraulic conductivity of the rock mass decreases as the water pressure increases. This behavior is characteristic of rock masses exhibiting partly open to moderately wide cracks.

Table 5-20: Summary of Lugeon's test at BH 14 and BH15

Bore Hole	Tiefe (m)	Lugeon	hydraulic conductivitiy (Fell et. al., 2005) (m/s)	Interpretation (T. Houlsby 1976)	Classification	Rock discontinuity condition
BH14						
	87 - 92	0.00	< 1E-07	Turbulent Flow	very low	very tight
	102 - 107	0.00	< 1E-07	Void Filling	very low	very tight
	120 - 125	0.00	< 1E-07	Turbulent Flow	very low	very tight
	220 - 225	0.00	< 1E-07	Turbulent Flow	very low	very tight
	230 - 235	0.00	< 1E-07	Void Filling	very low	very tight
	235 - 240	0.00	< 1E-07	Dilation	very low	very tight
	240 - 245	0.00	< 1E-07	Turbulent Flow	very low	very tight
BH15						
	30 - 35	0.00	< 1E-07	Turbulent Flow	very low	very tight
	35 - 40	0.00	< 1E-07	Turbulent Flow	very low	very tight
	40 - 45	0.00	< 1E-07	Turbulent Flow	very low	very tight
	45 - 50	0.00	< 1E-07	Turbulent Flow	very low	very tight
	60 - 65	0.00	< 1E-07	Turbulent Flow	very low	very tight

The discontinuity information recorded at site showed in general joints having partly open to very tight aperture (between 0.5 mm-<0.1mm) with quartz or clayey matrix and roughness varying between undulating rough to planar smooth.

In addition to the drilling works, several laboratory tests were carried out on the samples obtained from drilling works. The summary of the lab tests are presented in Table 5-21.

Table 5-21: Summary of Lab tests on BH 14 and BH15

Borehole Number	Samples Depth (m)	UCS				Shear Strength		Brasilian Test (Tensile Strength)	Pointload Index		Density	Water content	Porosity	Water absorption	Slake durability index
		σ_c (MPa)	E (GPa)	ν (-)	Deformation (GPa)	φ (°)	c (MPa)	σ_t (MPa)	I_{s50} (Mpa)	Remarks	ρ (g/cm³)	w_n (%)	n (%)	w_a (%)	s (%)
BH14	17.60 - 17.83								6.30	Diam.					
	18.00 - 18.25										2.86	0.60	2.26	0.79	99.4
	29.36 - 29.59										2.80	0.55	2.14	0.77	99.0
	31.60 - 31.90	92.0	45.3		22.7										
	36.13 - 37.00										2.69	0.18	1.27	0.47	99.4
	51.62 - 51.80								8.12	Diam.					
	56.71 - 57.00	108.0	53.6		26.8										
	57.29 - 57.56										2.75	0.28	0.97	0.35	99.4
	65.47 - 65.61								10.19	Diam.					
	79.13 - 79.37	66.0	43.0		21.5										
	93.02 - 93.28					48.8	0.2	10.94							
	94.79 - 94.96								8.78	Diam.					
	96.60 - 96.84														
	119.52 - 119.75														
	130.02 - 130.30	147.0	83.9		42.0										
	131.32 - 132.00														
	143.00 - 143.74														
	145.13 - 145.43	79.0	95.1		47.6										
	148.50 - 148.60								9.80	Diam.					
	149.17 - 149.36										2.66	0.40	2.44	0.92	99.3
	157.20 - 157.40								10.16	Diam.					
	158.03 - 158.27										2.66	0.09	2.33	0.88	99.2
	170.00 - 170.39	66.0	44.9		22.4										
	195.00 - 195.28	138.0	28.6		14.3										
	197.02 - 197.26								7.90	Diam.					
	198.02 - 198.25										2.58	0.54	2.10	0.82	99.5
	199.56 - 199.79					35.8	4.5								
	200.02 - 200.17								6.90	Diam.					
	204.82 - 205.00										2.65	0.51	2.01	0.76	99.4
	215.42 - 215.63								10.31	Diam.					
	220.06 - 220.28	123.0	48.9		74.5										
	221.72 - 222.00										2.69	0.31	1.03	0.39	99.4
	229.00 - 229.34	161.0	76.0		38.0										
	230.08 - 230.32								12.73	Diam.					
	234.73 - 235.00										2.69	0.17	0.91	0.34	99.5
	241.66 - 241.90	140.0	61.4	30.7											
	248.00 - 248.20										2.73	0.34	1.17	0.43	
	249.76 - 250.00					46.9	4.2	13.53							
BH15	15.74 - 15.93										2.73	0.55	2.26	0.83	99.4
	16.09 - 16.29								2.19	Diam.					
	17.23 - 17.45	39.8	88.4		44.2										
	21.42 - 21.61								2.19	Diam.					
	24.70 - 24.93								9.44	Diam.					
	25.25 - 25.41										2.82	0.38	1.41	0.50	99.1
	26.58 - 27.00	128.5	28.1		14.1										
	28.10 - 28.36								10.23	Diam.					
	30.84 - 31.00										2.73	1.13	3.73	1.38	99.1
	34.63 - 34.87					142.8	0.0								
	35.82 - 36.00										2.70	0.64	3.08	1.15	98.6
	37.68 - 37.90								5.5[illegible]	Diam.					
	39.28 - 39.50	31.1	58.6		79.3										
	41.70 - 41.94								9.74	Diam.					
	43.67 - 43.95										2.89	0.28	1.62	0.56	99.1
	44.19 - 44.45	35.2	15.9		79.3										
	47.25 - 47.49								9.00	Diam.					
	49.01 - 49.19										2.81	0.18	1.02	0.36	99.3
	51.44 - 51.62	67.4	42.0		21.0										
	53.10 - 53.30										27.60	0.25	0.69	0.25	99.3
	55.01 - 55.28								10.97	Diam.					
	57.33 - 57.57	69.7	46.7		23.4										
	59.31 - 59.50								8.78	Diam.					
	61.72 - 62.00										2.77	0.60	2.49	0.90	99.6
	62.00 - 62.19								7.90	Diam.					
	63.00 - 63.24	51.3	17.1		0.9										
	67.68 - 67.86										2.71	0.37	1.20	0.45	99.70
	68.01 - 68.21					38.9	3.7	17.56							
	69.02 - 69.27										2.79	0.31	1.73	0.62	99.4
	69.27 - 69.50								10.36	Diam.					

In summary, the laboratory test of the rock showed the physical and mechanical parameters with unit weight: 27-28 kN/m³, internal angle of friction: 38.9°-48.8°, cohesion: 0.01-4.5 MPa, unconfined compressive strength:40-161 MPa, tensile strength: 10.9 – 17.5 MPa, point load strength: 5.5 - 12 MPa, young's modulus: 15900 MPa to 95000 Mpa, slake durability > 99% (very high durability, Gamble chart) and water absorption: 0.3 1.38%

Groundwater measured in the area is between 18 and 23.5 m.

Based on the drilling and borelogs, RQD, RMR, Q and GSI values were measured/ calculated on these boreholes, the summary of these computation is presented in the following Table 5-22:

Table 5-22: RQD, RMR, Q-value and GSI calculation, BH14 and BH15

BH14 Depth (m)	RQD	Jn	Jr	Ja	Jw	SRF	RQD ÷ Jn	Jr ÷ Ja	Jw ÷ SRF	Q*	Q	RMR	GSI = RMR-5	Rock Quality based on Q-values
13 - 18	17	6	1	2	0.66	2	2.83	0.5	0.33	1.42	0.47	37	32	very Poor
18 - 25	72	6	1	2	0.33	1.5	12.00	0.5	0.22	6.00	1.32	46	41	Poor
25 - 30	47	6	1	1	0.33	1.5	7.83	1	0.22	7.83	1.72	49	44	Poor
30 - 35	73	6	2	1	0.33	1.5	12.17	2	0.22	24.33	5.35	59	54	Fair
35 - 45	61	6	1	1	0.33	1.5	10.17	1	0.22	10.17	2.24	51	46	Poor
45 - 50	85	4	1	2	0.33	1.5	21.25	0.5	0.22	10.63	2.34	52	47	Poor
50 - 65	72	4	1	2	0.33	1.5	18.00	0.5	0.22	9.00	1.98	50	45	Poor
65 - 75	62	6	2	2	0.33	1.5	10.33	1	0.22	10.33	2.27	51	46	Good
75 - 85	76	4	1	2	0.33	1.5	19.00	0.5	0.22	9.50	2.09	51	46	Fair
85 - 90	42	6	2	1	0.33	2	7.00	2	0.165	14.00	2.31	52	47	Good
90 - 95	67	6	2	2	0.33	1.5	11.17	1	0.22	11.17	2.46	52	47	Good
95 - 100	25	6	1	2	0.33	2	4.17	0.5	0.165	2.08	0.34	34	29	Poor
100 - 105	28	6	1	1	0.33	2	4.67	1	0.165	4.67	0.77	42	37	Fair
105 - 118	43	6	1	1	0.33	2	7.17	1	0.165	7.17	1.18	46	41	Fair
118 - 130	62	4	2	1	0.33	2	15.50	2	0.165	31.00	5.12	59	54	Fair
130 - 150	88	3	1	1	0.33	1.5	29.33	1	0.22	29.33	6.45	61	56	Fair
150 - 160	77	3	1	1	0.33	1.5	25.67	1	0.22	25.67	5.65	60	55	Fair
160 - 170	80	3	1	1	0.33	1	26.67	1	0.33	26.67	8.80	64	59	Fair
170 - 190	93	2	2	2	0.33	1	46.50	1	0.33	46.50	15.35	69	64	Good
190 - 210	94	2	2	2	0.33	1	47.00	1	0.33	47.00	15.51	69	64	Good
210 - 220	86	2	2	2	0.33	1	43.00	1	0.33	43.00	14.19	68	63	Good
220 - 240	90	2	2	2	0.33	1	45.00	1	0.33	45.00	14.85	68	63	Good
240 - 250	88	2	2	2	0.33	1	44.00	1	0.33	44.00	14.52	68	63	Good

BH15 Depth (m)	RQD	Jn	Jr	Ja	Jw	SRF	RQD ÷ Jn	Jr ÷ Ja	Jw ÷ SRF	Q*	Q	RMR	GSI =RMR-5	Rock Quality based on Q-values
12 - 20	49.37	6	1.5	1	0.66	1.5	8.23	1.5	0.44	12.34	5.43	59.23	54.23	Fair
20 - 30	54.5	6	1.5	1	0.66	1.5	9.08	1.5	0.44	13.63	6.00	60.12	55.12	Fair
30 - 40	61.1	4	1	1	0.33	1	15.28	1	0.33	15.28	5.04	58.56	53.56	Fair
40 - 50	57.8	4	1	1	0.33	1	14.45	1	0.33	14.45	4.77	58.06	53.06	Fair
50 - 60	70.1	4	1.5	0.75	0.33	1	17.53	2	0.33	35.05	11.57	66.03	61.03	Good
60 - 70	45.4	4	1.5	0.75	0.33	1	11.35	2	0.33	22.70	7.49	62.12	57.12	Fair

RQD and Q value are between 25%-90% and 0.34-15.5 at vertical shaft/ power house area and 45% to 70% and 4.7 to 11.6 at surge tank area respectively. Value of RQD and Q suggests very Poor to good quality rock.

5.3.7.6 Engineering geological conditions of powerhouse and tail race

The underground power cavern has an approximate dimension of 50m x 60 m. Furthermore, the depth from the crown of the hill up to powerhouse varies between 193 m and 260 m. Likewise, the tail race tunnel has a length of about 670 m and has the overburden height from hill surface between 37 m and 240 m. The characteristic of the rock types are shown in Table

5-18. The geology of the area comprises mainly tonalite with an exposure of para amphibolite and granite at the outlet area as shown below and further in Figure 5-25.

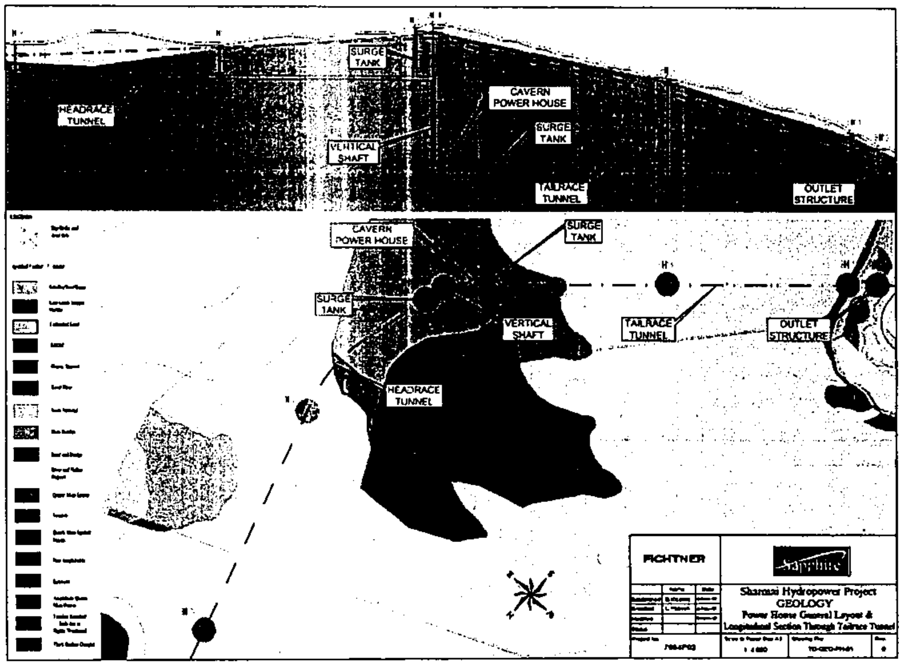


Figure 5-24: Geological conditions at Power house and outlet Area

These rock masses are slightly to moderately weathered having moderately open to tight joint (<0.5 mm), with clayey matrix infilling smooth planer surface and medium to high persistence. The details are covered in the geological mapping report.

a) b)

c)

Figure 5-25: Rock outcrops at Power house and outlet Area a) Para Amphibolite b) Tonalite and c) Granite

The major rock type encountered in these boreholes was Tonalite. The discontinuity information recorded at site showed in general joints having partly open to very tight aperture (between 0.5 mm-<0.1mm) with quartz or clayey matrix and roughness varying between undulating rough to planar smooth. The general trend of the discontinuity in this area are given in the following Table 5-23.

Table 5-23: The structural features of the Tonalite at the powerhouse and outlet area

	Strike	Dip
Joint J1	N40E to N70E	40SE to 75SE
Joint J2	N30W to N50W	30SW to 60SW
Joint J3	N20W to N70W	30NE to 70NE

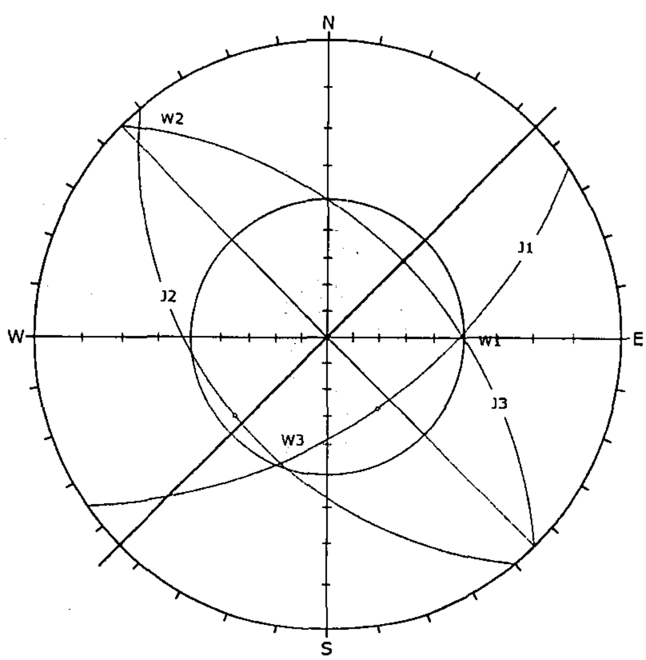


Figure 5-26: Stereographic projection and kinematic analysis of discontinuities measured in the Powerhouse and outlet area

Stereographic projection of major joint sets obtained from statistical analysis of joints is shown in Figure 5-26. Stereographic diagram shows distinct three discontinuity wedges w1, w2 and w3. Assuming friction angle of 40° for rock, the wedges w1 and w3 fall within the failure envelop. These wedges may trigger problem during excavation.

During geotechnical investigation phase three boreholes (BH11, BH12 and BH13) with depth from 40 m to 250 m were drilled at the location between powerhouse and tailrace tunnel.

In total 20 Lugeon tests were carried out in these boreholes (BH11 and BH13) at different depths from 30 m up to 170 m. The summary of these tests are presented in Table 5-24.

The computed Lugeon were all less than 1, which indicates very low permeability of the rock mass. As per the results the rock can be classified as having very low permeability and the corresponding rock discontinuity to be very tight. Apart from this, the pressure flow (p-q) diagram indicated most of the tests to lie in the range of turbulent flow region. As per Houlsby's (1976) interpretation, this indicates that the hydraulic conductivity of the rock mass decreases as the water pressure increases, which characterize the behavior of rock masses exhibiting partly open to moderately wide cracks.

Table 5-24: Summary of Lugeon test in BH11, BH12 and BH13

Bore Hole	Tiefe (m)	Lugeon	hydraulic conductivitiy (Fell et. al., 2005) (m/s)	Interpretation (T. Houlsby 1976)	Classification	Rock discontinuity condition
BH11			< 1E-07			
	33 - 38	0.00	< 1E-07	Turbulent Flow	very low	very tight
	43 - 48	0.00	< 1E-07	Turbulent Flow	very low	very tight
BH13						
	27 - 32	0.02	< 1E-07	Turbulent Flow	very low	very tight
	32 - 37	0.00	< 1E-07	Turbulent Flow	very low	very tight
	37 - 42	0.00	< 1E-07	Turbulent Flow	very low	very tight
	42 - 47	0.00	< 1E-07	Turbulent Flow	very low	very tight
	47 - 52	0.00	< 1E-07	Void Filling	very low	very tight
	52 - 57	0.00	< 1E-07	Void Filling	very low	very tight
	57 - 62	0.00	< 1E-07	Turbulent Flow	very low	very tight
	62 - 67	0.00	< 1E-07	Turbulent Flow	very low	very tight
	120 - 125	0.00	< 1E-07	Turbulent Flow	very low	very tight
	125 - 130	0.00	< 1E-07	Turbulent Flow	very low	very tight
	130 - 135	0.00	< 1E-07	Turbulent Flow	very low	very tight
	135 - 140	0.00	< 1E-07	Turbulent Flow	very low	very tight
	140 - 145	0.00	< 1E-07	Turbulent Flow	very low	very tight
	145 - 150	0.00	< 1E-07	Turbulent Flow	very low	very tight
	150 - 155	0.00	< 1E-07	Turbulent Flow	very low	very tight
	155 - 160	0.00	< 1E-07	Turbulent Flow	very low	very tight
	160 - 165	0.00	< 1E-07	Turbulent Flow	very low	very tight
	165 - 170	0.00	< 1E-07	Turbulent Flow	very low	very tight

Apart from drilling works, several laboratory tests were carried out on the samples taken from these boreholes. The summary of the laboratory tests are presented in Table 5-24.

Table 5-25: Summary of lab tests on samples taken from BH11, BH12 and BH13

Borehole Number	Samples Depth (m)	UCS				Shear Strength		Brasilian Test (Tensile Strength)	Pointload Index		Density	Water content	Porosity	Water absorption	Slake durability index
		σ_c (MPa)	E (GPa)	ν (-)	Deformation (GPa)	φ (°)	c (MPa)	σ_t (MPa)	$I_{s,50}$ (Mpa)	Remarks	ρ (g/cm³)	w_n (%)	n (%)	w_a (%)	s (%)
BH11	11.17 - 11.41										2.79	0.39	0.82	0.29	99.0
	16.21 - 16.44								7.02	Diam.					
	20.08 - 20.34										2.73	0.89	1.72	0.63	99.0
	24.00 - 24.17								7.24	Diam.					
	25.08 - 25.37										2.80	0.43	0.89	0.32	99.1
	28.56 - 28.82								10.53	Diam.					
	29.40 - 29.68										2.79	0.45	0.99	0.36	99.2
	36.02 - 36.28										2.74	0.70	2.30	0.84	99.4
	37.82 - 38.00								7.46	Diam.					
	39.00 - 39.20										2.71	0.54	2.62	0.97	99.6
	43.68 - 43.92								10.09	Diam.					
	45.00 - 45.24										2.76	0.11	1.48	0.54	99.5
	46.77 - 47.00								9.22	Diam.					
	51.62 - 51.93					43.1	1.1	13.24							
	52.71 - 52.97	175.8	17.0		85.1						2.79	0.81	6.74	2.44	99.4
	59.00 - 59.20								6.80	Diam.					
BH12															
	4.20 - 4.42										2.87	0.16	1.42	0.49	99.1
	8.81 - 9.00								9.22	Diam.					
	11.06 - 11.37										2.78	0.30	2.46	0.89	99.3
	14.81 - 15.00								8.34	Diam.					
	15.51 - 15.74										2.79	0.27	1.52	0.54	99.2
	20.23 - 20.44								9.40	Diam.					
	21.00 - 21.23										2.78	0.51	1.12	0.41	99.2
	25.35 - 25.58								7.07	Diam.					
	27.09 - 27.30										2.80	0.28	1.17	0.42	99.1
	29.00 - 29.32								9.22	Diam.	2.77	0.12	1.04	0.37	99.3
	32.78 - 33.00								8.60	Diam.					
	34.17 - 34.38										2.57	0.23	1.79	0.70	99.4
	34.38 - 34.60					44.6	1.1	16.99							
	37.62 - 37.83								6.36	Diam.					
	37.83 - 38.00										2.88	0.43	1.65	0.57	99.3
	39.00 - 39.23	61.1	28.4	14.2				14							
BH13	8.60 - 8.83								8.10	Diam.					
	11.32 - 11.65	95.0	46.9	23.5											
	13.48 - 13.72										2.78	0.32	1.79	0.65	99.6
	16.66 - 16.92								11.63	Diam.					
	21.17 - 21.37								7.68	Diam.					
	22.50 - 22.77										2.83	0.06	1.06	0.37	99.5
	26.60 - 26.81								12.96	Diam.					
	29.53 - 29.74										2.92	0.21	2.18	0.75	99.5
	36.00 - 36.26								7.99	Diam.					
	38.00 - 38.20										2.83	0.19	1.10	0.39	99.4
	50.62 - 50.85								6.76	Diam.					
	53.09 - 53.30														
	54.52 - 54.71										2.68	1.22	3.49	1.32	98.6
	56.05 - 56.38	180.0	12.1		60.4										
	59.51 - 59.70								6.80	Diam.					
	69.48 - 69.71										2.73	0.32	1.41	0.52	99.2
	70.35 - 70.59	120.0	47.2		23.6										
	72.05 - 72.23								5.49	Diam.					
	81.04 - 81.27	193.0	61.8		30.9										
	84.30 - 84.53								10.75	Diam.					
	90.03 - 90.22								9.00	Diam.					
	91.02 - 91.25										2.74	0.66	2.10	0.77	99.3
	94.44 - 94.75	167.0	16.8		83.9										
	96.53 - 96.73								5.05	Diam.					
	104.03 - 104.25										2.88	1.17	6.67	2.34	99.3
	112.26 - 112.52	243.0	65.0		32.5										
	114.00 - 114.20										2.70	0.42	1.32	0.49	99.4
	127.34 - 127.63	49.0	38.7		19.3										
	129.13 - 129.42	134.0	54.1		27.1										
	130.00 - 130.37										2.65	1.05	4.13	1.58	99.1
	146.60 - 146.82	104.0	86.0		43.0						3.24	0.27	2.00	0.62	99.4
	149.65 - 149.92					46.9	2.6	12							
	150.36 - 150.59	202.0	155.0												
	164.70 - 164.93	258.0	86.9		43.0										
	166.00 - 166.35					45.9	2.4	10.65							
	169.16 - 169.45					43.8	2.9		9.60	Diam.	2.61	1.20	3.73	1.45	99.4

In summary, the laboratory test of the rock showed the physical and mechanical parameters with unit weight: 25-29 kN/m³, internal angle of friction: 43°-47°, cohesion: 1.1-2.9 MPa, unconfined compressive strength:40-161 MPa, tensile strength: 49 – 258 MPa, point load strength: 3.7 – 12.9 MPa, young's modulus: 17000 MPa to 86900 Mpa, slake durability > 99% (very high durability, Gamble chart) and water absorption: 0.3 2.4%

Groundwater levels measured in the area is between 15 m and 23 m.

RQD, Q, RMR and GSI values computed for the boreholes BH11, BH12 and BH13 are given in the following
Table 5-26. Value of RQD and Q suggests poor to good quality rock.

Table 5-26: RQD, RMR, Q-value and GSI calculation, BH11, BH12 and BH13

BH11 Depth (m)	RQD	Jn	Jr	Ja	Jw	SRF	RQD + Jn	Jr + Ja	Jw + SRF	Q*	Q	RMR	GSI =RMR-5	Rock Quality based on Q-values
12 - 20	49.75	6	1	1	1	1.5	8.29	1	0.666667	8.29	5.53	59	54	Fair
20 - 30	52.8	4	1	1	1	1.5	13.20	1	0.666667	13.20	8.80	64	59	Fair
30 - 40	54.2	4	1.5	1	0.33	1	13.55	1.5	0.33	20.33	6.71	61	56	Fair
40 - 50	62.9	4	1.5	1	0.33	1	15.73	1.5	0.33	23.59	7.78	62	57	Fair.
50 - 60	73.9	4	1	1	0.33	1	18.48	1	0.33	18.48	6.10	60	55	Fair
60 - 70	39.1	4	1	1	0.33	1	9.78	1	0.33	9.78	3.23	55	50	Poor

BH12 Depth (m)	RQD	Jn	Jr	Ja	Jw	SRF	RQD + Jn	Jr + Ja	Jw + SRF	Q*	Q	RMR	GSI =RMR-5	Rock Quality Based on Q
4 - 10	60	4	1	2	0.66	1	15.00	0.5	0.66	7.50	4.95	58	53	Fair
10 - 20	69	4	1.5	1	0.33	1	17.25	1.5	0.33	25.88	8.54	63	58	Fair
20 - 30	85	4	1.5	1	0.33	1	21.25	1.5	0.33	31.88	10.52	65	60	Good
30 - 40	84	4	1.5	1	0.33	1	21.00	1.5	0.33	31.50	10.40	65	60	Good

BH13 Depth (m)	RQD	Jn	Jr	Ja	Jw	SRF	RQD ÷ Jn	Jr ÷ Ja	Jw + SRF	Q*	Q	RMR	GSI =RMR-5	Rock Quality Based on Q
4 - 11	19	4	1	2	0.66	1.5	4.75	0.5	0.44	2.38	1.05	44	39	Poor
11 - 18	27	4	1	2	0.33	1.5	6.75	0.5	0.22	3.38	0.74	41	36	Very poor
18 - 30	75	4	2	2	0.33	1.5	18.75	1	0.22	18.75	4.13	57	52	Fair
30 - 40	72	4	2	2	0.33	1.5	18.00	1	0.22	18.00	3.96	56	51	Poor
40 - 55	65	4	1	2	0.33	1.5	16.25	0.5	0.22	8.13	1.79	49	44	Poor
55 - 65	42	4	2	1	0.33	1.5	10.50	2	0.22	21.00	4.62	58	53	Fair
65 - 70	70	3	1	1	0.33	1.5	23.33	1	0.22	23.33	5.13	59	54	Fair
70 - 75	85	3	1	1	0.33	1	28.33	1	0.33	28.33	9.35	64	59	Fair
75 - 95	90	3	1	1	0.33	1	30.00	1	0.33	30.00	9.90	65	60	Fair
95 - 100	38	4	1	1	0.33	1	9.50	1	0.33	9.50	3.14	54	49	Poor
100 - 110	82	2	1	1	0.33	1	41.00	1	0.33	41.00	13.53	67	62	Good
110 - 130	92	2	1	1	0.33	1	46.00	1	0.33	46.00	15.18	68	63	Good
130 - 140	37	3	1	1	0.33	1.5	12.33	1	0.22	12.33	2.71	53	48	Poor
140 - 150	82	2	1	1	0.33	1	41.00	1	0.33	41.00	13.53	67	62	Good
150 - 165	73	2	1	1	0.33	1	36.50	1	0.33	36.50	12.05	66	61	Good
165 - 170	86	2	1	1	0.33	1	43.00	1	0.33	43.00	14.19	68	63	Good

No any weak zone is seen or traced in the area although the project site lies in the region of active seismicity. Although no any shear and crossed rock masses is seen on the surface mapping and detail geotechnical/geological and geophysical investigation works, it may encounter weak rock masses that is sheared and crushed during excavation which needs to be considered.

5.3.8 Geotechnical studies of underground structures

Geotechnical studies of proposed underground structures i.e. headrace tunnel, pressure tunnel, vertical shaft, power cavern and tailrace include establishment of geotechnical parameters to know an interaction between existing ground condition and structure that will be built on it. Data required for geotechnical studies of underground structures are acquired from geological and engineering geological mapping, field test, laboratory test and empirical techniques. Since all necessary parameters for geotechnical studies are not available at present level of study, those parameters which are most essence for geotechnical studies are determined using empirical relationships. Geotechnical studies include preliminary stress analysis and rock excavation support design along underground structures.

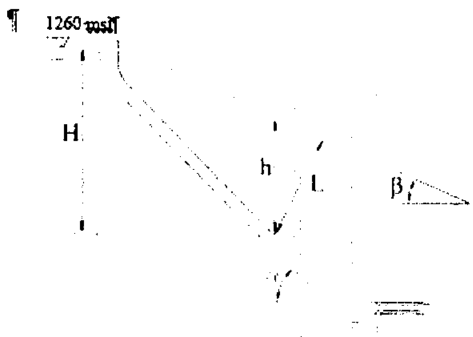
5.3.8.1 Stress analysis along the tunnel

An attempt is made for analysis of stress condition produced by overburden rock body along underground structures using RMR, GSI and Q which are extracted from surface mapping and other values obtained from different empirical methods. This includes determination of rock cover analysis, in-situ stress deformation modulus (Em), in-situ stress condition, elastic and plastic behaviour and failure criteria. However, at this stage of study all data like in-situ stress and elastic and plastic parameters are difficult to acquire, therefore some assumptions are made for stress parameters. They are as follows.

- It is assumed that major principal stress ($\sigma 1$) is oriented along vertical direction and minor principal stress ($\sigma 3$) is oriented along horizontal direction and intermediate principal stress axis ($\sigma 2$) is oriented in longer axis of an underground opening.
- Elastic and plastic parameters are obtained from empirical relation proposed by Hoek et al. (1995).

5.3.8.1.1 Rock cover analysis

At an initial stage of analysis, a minimum depth of rock cover above the tunnel is taken from the thumb rule as per Norwegian criteria (see Figure 5-27), where rock cover is selected to hydrostatic head of water flow within the tunnel. A minimum safety factor of required and computed are put together in Table 5-27. For the computation, the location of the boreholes from respective sections of diversion tunnel, headrace tunnel, vertical shaft and tailrace tunnel are selected and required parameters for computation are measured. Water head within these underground structures gives maximum internal hydrostatic pressure is low compared to pressure applied by rock cover. Thus, governing parameter for stability of rock mass around these structures is the rock cover.



(Broch 1982)

$$h' > \frac{H}{\rho \cos \alpha}$$

$$\rho = \frac{\rho_r}{\rho_w}$$

Bergh-Christensen and Dannevig (1971)

$$L > \frac{H}{\rho \cos \beta}$$

Figure 5-27: Topographic requirement in Norwegian Rule of thumbs

Table 5-27: Minimum Rock cover analysis as per Norwegian criteria

Location	Location	H m	pr t/m³	h' m	α °	β °	L' m	L'/H -	h m	L m	FoS1 (h'/h)	FoS2 (L'/L)	Remarks
Diversion tunnel	BH10	21.7	2.6	29.92	0	19.5	26.32	1.21	8.35	8.85	3.58	2.97	>1.3-1.5
	BH1	35.48	2.6	43	0	25.88	46.59	1.31	13.65	15.17	3.15	3.07	>1.3-1.5
	BH9	27	2.6	27	0	20.67	25.27	0.94	10.38	11.10	2.60	2.28	>1.3-1.5
Intake tunnel	BH19	40.6	2.6	50.86	0	10	55.36	1.36	15.62	15.86	3.26	3.49	>1.3-1.5
	BH16	42.31	2.6	52.56	0	10	67.32	1.59	16.27	16.52	3.23	4.07	>1.3-1.5
	BH15	43.95	2.6	84.53	0	15.24	80.26	1.83	16.90	17.52	5.00	4.58	>1.3-1.5
vertical Shaft (BH14)	Top	81.015	2.6	90.26	0	20.55	90.41	1.12	31.16	33.28	2.90	2.72	>2.0-2.2
	Middle	193.575	2.6	164.1285	0	12.09	164.13	0.85	74.45	76.14	2.20	2.16	>2.0-2.2
	Bottom	306	2.6	240.25	0	20.55	237.06	0.77	117.69	125.69	2.04	1.89	>2.0-2.2
Tail race	BH11	45.03	2.6	55.2	0	31.45	51.3	1.14	17.32	20.30	3.19	2.53	>1.3-1.5
	BH12	50	2.6	34.75	0	22.69	39.13	0.78	19.23	20.84	1.81	1.88	>1.3-1.5
	BH13	144.55	2.6	148.8	0	20.83	149.8	1.04	55.60	59.48	2.68	2.52	>1.3-1.5

The rock cover criterion shows that these structures are deemed safe throughout its whole length.

5.3.8.1.2 Estimation of in-situ deformation modulus

In-situ deformation modulus (Em) of a rock mass is a significant parameter in any form of numerical analysis related to stability of rock masses, but this parameter is difficult and expensive to determine in field and are not generally determined at this study level. However, this parameter can be determined empirically using rock mass classification RMR and Q. In-situ deformation modulus (Em) obtained along underground structures is presented in Table x. Its distribution along the tunnel alignment from the headrace inlet portal to the tailrace construction is shown in Table 5-28.

Table 5-28: Estimation of in-situ deformation modulus (Em) along underground structures

Structure	Location	Rock type	H (m)	Q	RMR	GSI	Em $= 10^{(RMR-10)/40}$ (Gpa)	Em $= 25\log_{10}Q$ (Gpa)
Diversion tunnel	BH10	quartz mica Gneiss	29.92	0.55	38.6	34	5.19	
	BH1	quartz mica Gneiss	43.00	8.27	63.0	58	21.15	22.94
	BH9	quartz mica Gneiss	27.00	0.29	32.9	28	3.73	
Intake tunnel	BH19	Tonalite	50.86	14.19	67.9	63	27.98	28.80
	BH16	Tonalite	52.56	10.1	64.8	60	23.46	25.11
	BH15	Tonalite	84.53	7.49	62.1	57	20.09	21.86
vertical Shaft (BH14)	Top	Tonalite	90.26	2.46	52.1	47	11.29	9.77
	Middle	Tonalite	164.13	8.8	63.6	59	21.84	23.61
	Bottom	Tonalite	240.25	14.52	68.1	63	28.31	29.05
Tail race	BH11	Tonalite	55.20	6.1	60.3	55	18.07	19.63
	BH12	Tonalite	34.75	10.4	65.1	60	23.82	25.43
	BH13	Tonalite	148.80	13.53	67.4	62	27.30	28.28

5.3.8.1.3 Insitu stress analysis

Basically, governing parameters for stability of rock inside the tunnel are orientation of joints, their separation and pressure caused by overburden rock. To avoid hydraulic fracturing of rock with consequent opening of existing joints, a minor principal component of in-situ stresses should be higher than an internal hydrostatic pressure in the tunnel. The hydrofracturing and hydraulic jacking onsite tests were proposed and included in the contractor's TOR for computing the minimum stress values in situ. However, these tests were not available in Pakistan and could not be performed.

Therefore, an empirical method is used here for an evaluation of in-situ stress along the tunnel. Analysis using rock cover is a very simplified approximation and a more elaborate method to analyze in-situ stresses. Vertical and horizontal stress as well as horizontal to vertical stress ratio (k) along the tunnel is calculated and presented in Table 5-29. The depth from the crown of the tunnel to the surface level is used as maximum rock cover ignoring depth of overburden cover above the bed. Unit weight of overlying rock (γ) is assumed as 0.026 MN/m^3 and in-situ deformation modulus (Em) is taken from Table 5-28.

Table 5-29: Estimation of in-situ vertical and horizontal stress along underground structures

Structure	Location	Rock type	Rock cover (m)	Unit weight of rock (MN/m^3)	In-situ deformation modulus Em (Gpa)	In-situ vertical stress (σv) (Mpa)	Ratio of horizontal to vertica'stress (Gpa)	In-situ horizontal stress (σH) (Mpa)
Diversion tunnel	BH10	quartz mica Gneiss	29.92	0.026	5.2	0.78	3.84	2.99
	BH1	quartz mica Gneiss	43.00	0.026	21.2	1.12	2.83	3.16
	BH9	quartz mica Gneiss	27.00	0.026	3.7	0.70	4.20	2.95
Intake tunnel	BH19	Tonalite	50.86	0.026	28.0	1.32	2.47	3.26
	BH16	Tonalite	52.56	0.026	23.5	1.37	2.40	3.28
	BH15	Tonalite	84.53	0.026	20.1	2.20	1.68	3.70
vertical Shaft (BH14)	Top	Tonalite	90.26	0.026	11.3	2.35	1.61	3.77
	Middle	Tonalite	164.13	0.026	21.8	4.27	1.11	4.73
	Bottom	Tonalite	240.25	0.026	28.3	6.25	0.92	5.72
Tail race	BH11	Tonalite	55.20	0.026	18.1	1.44	2.31	3.32
	BH12	Tonalite	34.75	0.026	23.8	0.90	3.38	3.05
	BH13	Tonalite	148.80	0.026	27.3	3.87	1.17	4.53

5.3.8.1.4 Determination of elastic and/or plastic behaviour of rock

Different stress parameters like vertical stress, maximum tangential boundary stress, in-situ deformation modulus and ratio of horizontal to vertical stress are used here to find out elastic and/ or plastic behaviour of rock along the tunnel. Ratio of maximum tangential boundary stress to unconfined compressive strength of rock mass is referred as damage index (Di). If Di<0.4, rock behaves as elastic and if Di >0.4 rock behaves as plastic. Damage index for the tunnel is estimated and presented in Table 5-30. Vertical stress and horizontal to vertical stress ratio is taken from Table 5-29 and unconfined compressive strength (UCS) obtained from laboratory is used for determination of damage index.

Table 5-30: Damage index of rock mass along underground structures

Structure	Location	In-situ vertical stress (σv) (Mpa)	Ratio of horizontal to verticalstress (k)	unconfined compressive strength (σc) (Mpa)	Maximum tangential boundary stress	Damage index (D)
Diversion tunnel	BH10	0.78	3.84	18.45	11.18	0.61
	BH1	1.12	2.83	47.3	11.52	0.24
	BH9	0.70	4.20	21.6	11.10	0.51
Intake tunnel	BH19	1.32	2.47	108	11.72	0.11
	BH16	1.37	2.40	106	11.77	0.11
	BH15	2.20	1.68	51.3	12.60	0.25
vertical Shaft (BH14)	Top	2.35	1.61	66	12.75	0.19
	Middle	4.27	1.11	66	14.67	0.22
	Bottom	6.25	0.92	140	16.65	0.12
Tail race	BH11	1.44	2.31	175.8	11.84	0.07
	BH12	0.90	3.38	61.1	11.30	0.19
	BH13	3.87	1.17	202	14.27	0.07

As mentioned earlier, for damage index Di 0.4, rock mass behaves as an elastic condition and no visible damage occurs. Some of calculated value of Di along the tunnel is more than 0.4. Although rock masses may not show plastic behaves but the value suggests plastic behaviour due to some anomalous results and data. The anomalous result is seen on topography having least overburden on the tunnel.

5.3.8.1.5 Determination of rock mass strength

Rock mass properties are assumed to be adequately characterized by the biaxial failure criteria developed by Hoek and Brown (1980). In order to determine rock mass strength parameters, mb and s, GSI calculated and tabulated in Table 5-28 is taken. Determined strength parameter is tabulated in Table 5-31.

Table 5-31: Strength parameters of rock

Structure	Location	Rock type	Rock cover (m)	GSI	m_i	m_b	s
Diversion tunnel	BH10	quartz mica Gneiss	29.92	34	28	2.616	0.0006
	BH1	quartz mica Gneiss	43.00	58	28	6.25	0.0094
	BH9	quartz mica Gneiss	27.00	28	28	2.129	0.0003
Intake tunnel	BH19	Tonalite	50.86	63	28	7.7	0.0162
	BH16	Tonalite	52.56	60	28	6.903	0.0115
	BH15	Tonalite	84.53	57	29	6.271	0.0085
vertical Shaft (BH14)	Top	Tonalite	90.26	47	29	4.384	0.0028
	Middle	Tonalite	164.13	59	29	6.604	0.01
	Bottom	Tonalite	240.25	63	29	7.758	0.0165
Tail race	BH11	Tonalite	55.20	55	29	5.87	0.0069
	BH12	Tonalite	34.75	60	29	6.97	0.0118
	BH13	Tonalite	148.80	62	29	7.583	0.0154

By using rock mass strength parameters tabulated in Table 5-31 and standards set up on software itself, Hoek-Brown classification, Hoek-Brown criterion, failure envelope range, Mohr-Coulomb fit and rock mass parameters are obtained using the RocLab software developed by Rocscience. Input parameters used for stress analysis on the software are intact uniaxial compressive strength (σc), mi geological strength index (GSI) and disturbance factor (D), modulus ratio (mr), unit weight of rock (γ) and tunnel depth (z).

Among these parameters, σc, mi GSI and z are taken from Table 5-31. Disturbance factor is considered as zero considering blasting results minimal disturbance to surrounding rock mass. mr for different rocks is used from a table given in the software. mr is used to calculate intact modulus (Er = mrσc). γ is considered as 0.026 MN/m^3.

Table 5-32: Analysis of rock strength by using RocLab software along underground structures

Structure		Diversion tunnel			Intake tunnel			Vertical Shaft (BH14)			Tail race		
Location		BH10	BH1	BH9	BH19	BH16	BH15	Top	Middle	Bottom	BH11	BH12	BH13
Rock type		quartz mica Gneiss	quartz mica Gneiss	quartz mica Gneiss	Tonalite	Tonalite	Tonalite	Tonalite	Tonalite	Tonalite	Tonalite	Tonalite	Tonalite
Hoek Brown Classification	Intact uniaxial compressive strength (σ_i, MPa)	18.45	47.30	21.60	108.00	106.00	51.30	66.00	66.00	140.00	175.80	61.10	202.00
	GSI	33.62	58.01	27.86	62.87	59.81	57.12	47.10	58.57	63.08	55.27	60.08	62.44
	m_i	28	28	28	29	29	29	29	29	29	29	29	29
	Disturbance factor (D)	0	0	0	0	0	0	0	0	0	0	0	0
	Intact modulus (E_i MPa)	99700	62500	63100	43000	52100	17100	43000	44900	61400	17000	28400	155000
Hoek Brown Criterion	m_b	2.6160	6.2500	2.1290	7.7000	6.9030	6.2710	4.3840	6.6040	7.7580	5.8700	6.9700	7.5830
	s	0.0006	0.0094	0.0003	0.0162	0.0115	0.0085	0.0028	0.0100	0.0165	0.0069	0.0118	0.0154
	a	0.52	0.50	0.53	0.50	0.50	0.50	0.51	0.50	0.50	0.50	0.50	0.50
	Application	Tunnel											
	σ_{3max} (MPa)	0.40	0.62	0.37	0.76	0.78	1.17	1.27	2.23	3.55	0.90	0.51	2.31
	Unit Weight (MN/m³)	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026
	Tunnel Depth (m)	29.92	43	27	50.86	52.56	84.53	90.26	164.13	240.25	55.2	34.75	148.8
Mohr-Coulomb Fit	Cohesion (MPa)	0.18	0.59	0.16	1.30	1.13	0.82	0.76	1.37	2.69	1.35	0.69	2.71
	Friction angle (deg.)	53.92	63.12	53.86	66.97	66.43	59.92	58.83	57.73	60.86	67.73	65.88	65.04
Rock Mass Parameters	Tensile strength (MPa)	-0.004	-0.071	-0.003	-0.227	-0.177	-0.070	-0.042	-0.100	-0.298	-0.207	-0.104	-0.410
	Uniaxial compressive strength (MPa)	0.406	4.520	0.319	13.597	11.221	4.659	3.352	6.511	17.836	14.297	6.568	24.820
	Global strength (MPa)	3.754	15.964	3.825	40.909	37.730	17.299	18.210	22.913	53.259	56.868	21.867	75.845
	Deformation modulus (MPa)	10,300.2	29,681.0	4,485.5	25,149.0	26,867.0	7,779.1	11,023.6	21,890.8	36,198.1	7,040.2	14,819.6	89,160.4

5.3.8.2 Rock excavation support design

5.3.8.2.1 General

Proposed rock excavation support design is based on today's common practice on tunnel construction. Rock support design is mainly based on classification of rock mass quality along the tunnel. Input parameters are provided by engineering geological surface mapping, field investigations and laboratory testing of rock samples. RMR and Q rules are adopted for present design.

Combinations of rock bolts, fibre-reinforced and steel mesh-reinforced shotcrete and cast concrete lining can be used as dictated by rock mass quality encountered under excavation. A recommended principle for rock support is to require equal quality for an initial support as for permanent support. This requirement intends to incorporate an initial support as part of a total requirement for the permanent support. Experience indicates that 50%-80% of total required support is performed successively during excavation and remaining 20%-50% later, thus leaving a relatively small volume of work to be executed after excavation is finished. Adequate support is assessed based on relevant properties of rock mass and support materials (shotcrete, rock bolts etc.). The support is installed and monitored as required and if necessary strengthened by an additional support.
During tunnel excavation, types and quantities of rock support defined in the present design can used as a menu for detailed design on construction phase. Detailed design is adapted to in-situ geological conditions either as registered at excavation front or as indicated by investigations ahead of face on logging of geological conditions. Additional information for the design is obtained by testing of materials sampled from the tunnel and from direct

registration of in-situ stress conditions and behaviour of proposed support; i.e. by measuring deformation (convergence) of the tunnel cross section. Thus, type and extent of support needed is finalized only during excavation. Study concludes that support system based on either of system i.e. RMR and Q system can be adopted for rock support patterns.

The preliminary support design based on the geotechnical investigation has been dealt in detail in the following headings which is based on the computed RQD, RMR and Q-values at each site.

5.3.8.2.2 Diversion tunnel

The excavation of the diversion tunnel mainly takes place in quartz mica gneiss with different grads of weathering and. Rock bolting, and support with reinforced shortcrete and steel ribs might be expected to be required in the tunnel. The final application of the support measures should follow the NATM approach and should finally be defined in the detailed design. Partially also a pipe roof/ umbrella might be required; this circumstance should also be considered for the detailed design of tunnel portals.

Here as an initial approach for preliminary stage design based on the Q-value and NGI (Norwegian geotechnical institute) method has been used. For the Diversion tunnel based on the borelogs from BH1, BH9 and BH10 RQD and Q values were computed. RQD value calculated based on boreholes core recovery at site at the depth of planned tunnel vary between 10% and 37%. This value suggests very poor to fair rock quality. Furthermore, the computed Q-value for these boreholes vary between 0.55 and 7, which represents very poor to fair rock class (class E and C).

The diversion tunnel is presumed to be D-shaped with 7.5 m diameter half circle on top of 7.5m width and 3.75 m height rectangular opening. As per the chart Figure 5-28 from the Norwegian geotechnical institute (NGI), considering the total height of planned tunnel (H) as 7.5 m and Excavation support ratio (ESR) for water tunnels as 1.6, the tunnel support design for the tunnel roof considering the Q values of 0.55 and 7 and the tunnel side as shown in Table 5-33.

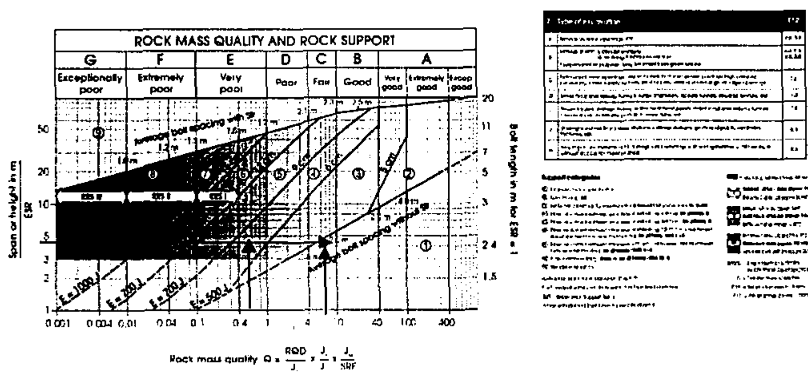


Figure 5-28: Q-System chart for Rock support estimate, Norwegian Geotechnical Institute (NGI, 2014)

Table 5-33: Tunnel support design diversion tunnel

Q	ESR	Span (m)	Span/ESR	Support Class/ Category	Description
Roof design					
0.55	1.6	7.5	4.86	E/ 4	Syst. bolting 1.0 m x 1.0m 8 cm fibercrete
7.0	1.6	7.5	4.86	C/ 1	Unsupported or spot bolting 2.0 m x 2.0 m
Wall, for $0.1<Q<10$, $Q_{wall}= 2.5Q_{Roof}$					
1.375	1.6	7.5	4.86	D/ 3	Syst. bolting 1.4 m x 1.4m 6 cm fibercrete
17.5	1.6	7.5	4.86	B/ 1	Unsupported or spot bolting 2.5 m x 2.5 m

ESR = excavation support ratio

Typical sections for support at the diversion tunnel are presented in Figure 5-29.

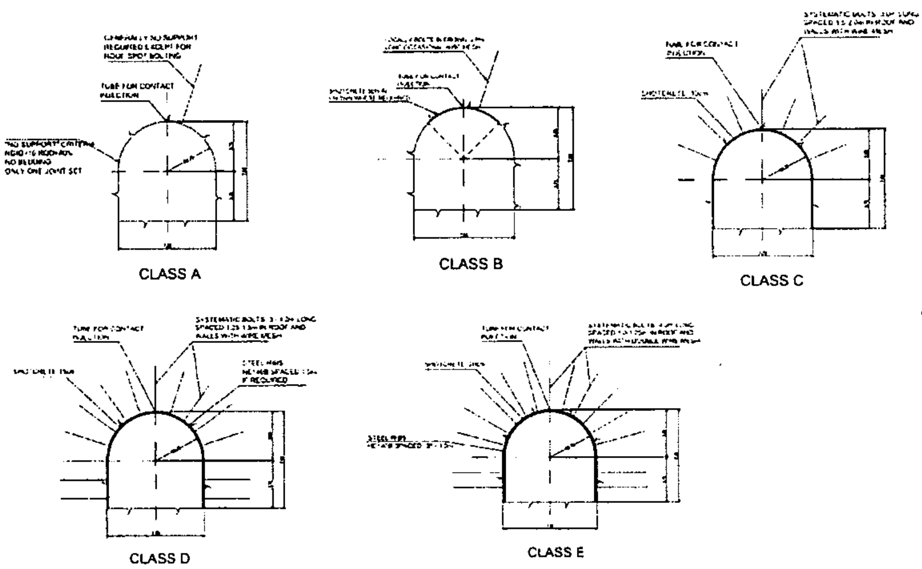


Figure 5-29: Typical support sections for the diversion tunnel

Groundwater was observed in BH10, BH1 and BH9, therefore during exaction and construction of the tunnel and associated structures (inlet and bottom outlet), groundwater management/ control must be considered.

5.3.8.2.3 Head race tunnel

The excavation of the diversion tunnel mainly takes place starting with quartz mica gneiss of metamorphic origin and then around half a way it will make a transition to the tonalite rock of igneous origin. Thus for the advancement of the tunnels, different tunnel rock qualities will be found along the alignment. Adapted to the site conditions, following the NATM approach, different support measures should be considered during the detailed design; amongst others rock bolting, support with reinforced shotcrete, steel ribs or partially also the use of an pipe roof/umbrella. Care shall be taken during the excavation with respect to rock burst.

Here in this report as a preliminary approach, the support calculation has been calculated based on calculated RQD and Q values and NGI chart. The measured RQD values lie between 82%-88% and Q-values between 10 and 14 respectively. Value of RQD sand Q values suggests good rock quality.

As per the chart Figure 5-28 from the Norwegian geotechnical institute (NGI), considering the total height of planned tunnel (H) as 7.5 m and Excavation support ratio (ESR) for water tunnels as 1.6, the tunnel support design for the tunnel roof considering the Q values of 10 and 14 and the tunnel side as shown in Table 5-34.

Table 5-34: Tunnel support design headrace tunnel

Q	ESR	Span (m)	Span/ESR	Support	
				Class/ Category	Description
Roof design					
10	1.6	7.5	4.86	B/ 1	Unsupported or spot bolting 2.0 m x 2.0 m
14	1.6	7.5	4.86	B/ 1	Unsupported or spot bolting 2.1 m x 2.1 m
Wall, for 0.1<Q<10, Q_{wall}= 2.5Q_{Roof}					
1.375	1.6	7.5	4.86	D/ 1	Unsupported or spot bolting 2.5 m x 2.5 m
17.5	1.6	7.5	4.86	B/ 1	Unsupported or spot bolting 3.0 m x 3.0 m

Typical sections for support at the headrace tunnel are presented in Figure 5-30.

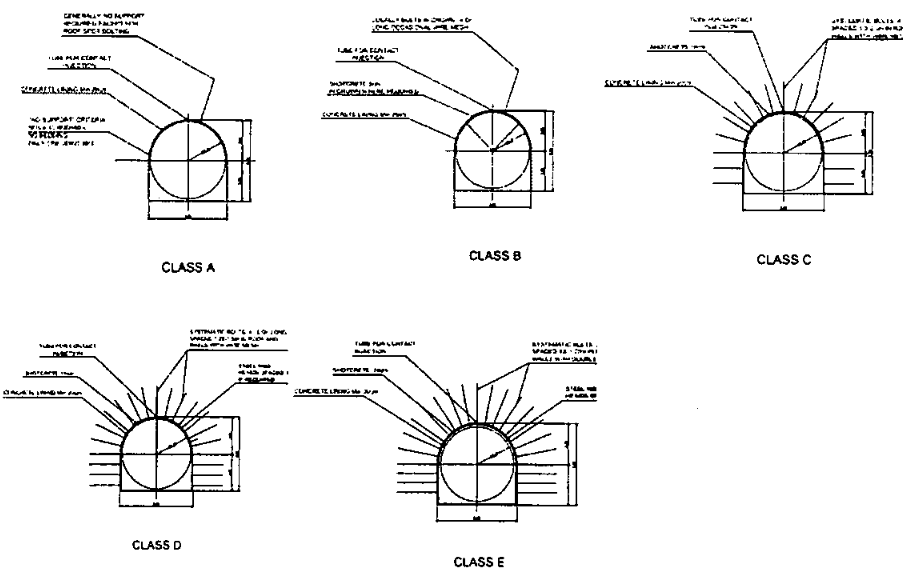


Figure 5-30: Typical support sections for the headrace tunnel

Groundwater was observed in BH16 and BH19, therefore during exaction and construction of the tunnel and associated structures, groundwater management/ control must be considered.

5.3.8.2.4 Surge tank and vertical shaft

The surge tank is located at the end of the headrace tunnel. For the excavation of the surge tank, most part will be in competent tonalite rock which is overed by thick overburden boulders of varying size. The excavation in rock requires blasting. For temporary support of the shaft walls rock support should be foreseen as the quality of the rock varies with depth. Rock bolting, anchoring and support with reinforced shotcrete might be required which must be defined in the detailed design. Groundwater and rainwater management must be considered i.e. an adequate drainage or pumping system to evacuate water from the excavation.

A comparable geotechnical situation is observed in the pressure shaft located in a short distance, approximately half way to the powerhouse.

Here as a preliminary approach, based on borehole datas, computed RQD and Q-values along with NGI chart have been used for the design of the support. The computed RQD and Q value are between 25%-90% and 0.34-15.5 at vertical shaft/ power house area and 45% to 70% and 4.7 to 11.6 at surge tank area respectively. Value of RQD and Q suggests very Poor to good quality rock.

As per the chart Figure 5-28 from the Norwegian geotechnical institute (NGI), considering the total height of planned tunnel (H) as 7.5 m and Excavation support ratio (ESR) for water tunnels as 1.6, the tunnel support design for the tunnel roof considering the Q values at surge tank area and vertical shaft area are as shown in Table 5-35.

Table 5-35: Tunnel support design Surge tank area and vertical shaft

Q	ESR	Span (m)	Span/ESR	Support	
				Class/ Category	Description
Surge tank					
0.34	1.6	7.5	4.86	E/ 5	Bolting 1.0 m x 1.0 m and fibrecrete 9-12 cm thick
15.5	1.6	7.5	4.86	B/ 1	Unsupported or spot bolting 2.0 m x 2.0 m
Vertical shaft					
4.7	1.6	7.5	4.86	C/ 3	Syst. bolting 1.6 m x 1.6 m Fibrecrete 5-6 cm thick
11.6	1.6	7.5	4.86	B/ 1	Unsupported or spot bolting 3.0 m x 3.0 m

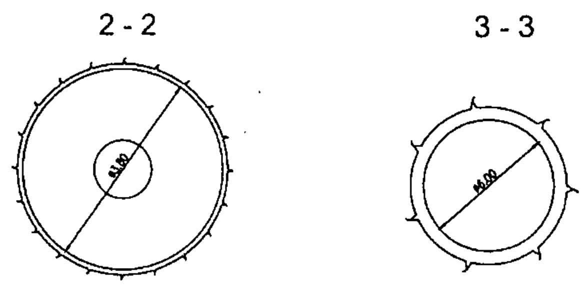


Figure 5-31: Planned Surge tank and vertical shaft dimension

Groundwater was observed in BH14, therefore during exaction and construction of the tunnel and associated structures, groundwater management/ control must be considered.

5.3.8.2.5 Powerhouse and tail race tunnel

The geological information about the powerhouse location are based on the BH13 and BH14 and tail race on BH 11 and 12, which is drilled at vertical shaft location. The excavation of the powerhouse and tailrace is mostly in tonalite type rock of igneous origin. Temporary rock support should be foreseen as the quality of the rock varies with depth and zones of weathering during excavation. Rock bolting, anchoring and support with reinforced shotcrete might be required- this must be defined in the detailed design.

Here as a preliminary approach, calculated RQD, Q-values in Boreholes at different depth along with NGI has been used for the computation of the required support.

As per the chart Figure 5-28 from the Norwegian geotechnical institute (NGI), considering the total height of planned tunnel (H) as 10 m and Excavation support ratio (ESR) for water tunnels as 1.6, the tunnel support design for the tunnel roof considering the Q values of 6 and 14 and the tunnel side as shown in Table 5-37. Similarly, for powerhouse cavern with planned height of around 70 m with Q value as 10, the results are shown in Table 5-36.

Table 5-36: Tunnel support design powerhouse

Q	ESR	Span (m)	Span/ESR	Support	
				Class/ Category	Description
Powerhouse, roof					
10	1.6	70	43.75	B/ 5	bolting 2.0 m x 2.0 m fibrecrete 9-12 cm thick (E700)
Wall, for 0.1<Q<10, Q_{wall}= 2.5Q_{Roof}					
25	1.6	70	43.75	B/ 5	bolting 2.5 m x 2.5 m fibrecrete 9-12 cm thick (E700)

Table 5-37: Tunnel support design tail race

Q	ESR	Span (m)	Span/ESR	Support	
				Class/ Category	Description
tailrace, roof					
6	1.6	10	6.25	C/ 3	bolting 1.7 m x 1.7 m fibrecrete 5-6 cm thick
Tailrace, side wall, for 0.1<Q<10, Q_{wall}= 2.5Q_{Roof}					
15	1.6	10	6.25	B/ 3	Unsupported or spot bolting 2.0 m x 2.0 m

Typical sections for support at the diversion tunnel are presented in Figure 5-32.

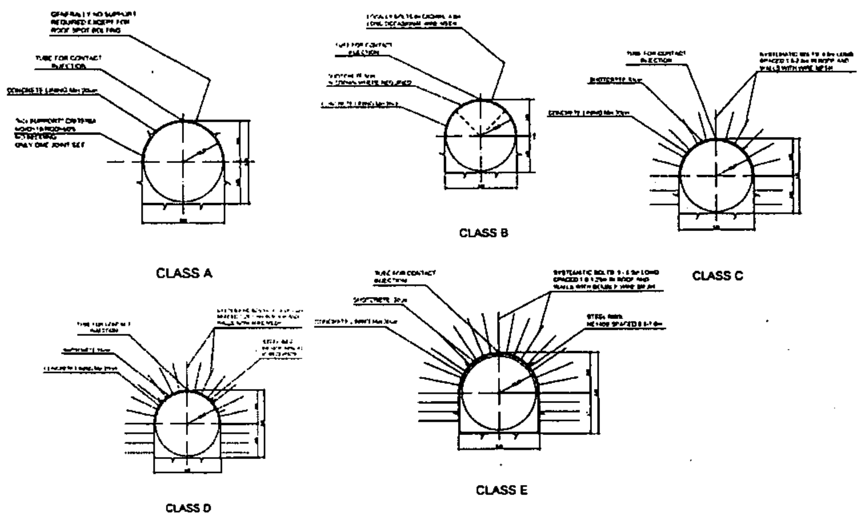


Figure 5-32: Typical tailrace tunnel support

Groundwater was observed in BH11 and BH13, therefore during exaction and construction of the tunnel, Powerhouse and associated structures, groundwater management/ control must be considered.

In order to get more detailed geological information and to reduce uncertainties at the powerhouse area, further core drilling is recommendable for the detailed design and construction.

5.3.9 Rock mass classification

5.3.9.1 Diversion tunnel

Table 5-38: Percentage distribution of Rock classes in Diversion tunnel

Class	[%]
A	0-5
B	15-25
C	60-70
D	10-20
E	5-10

5.3.9.2 Portal structures of tunnels

Table 5-39: Percentage distribution of Rock classes in Portal of Diversion tunnel

Class	[%] Inlet	[%] Outlet
A	0-5	0-5
B	15-20	45-60
C	50-60	30-40
D	20-25	10-15
E	5-15	5-10

Table 5-40: Percentage distribution of Rock classes in Portal of Headrace tunnel

Class	[%]
A	0-5
B	20-25
C	50-60
D	10-20
E	5-15

5.3.9.3 Headrace tunnel

Table 5-41: Percentage distribution of Rock classes in Headrace tunnel

Class	[%]
A	0-10
B	20-25
C	50-60
D	10-20
E	5-15

5.3.9.4 Surge shaft

Table 5-42: Percentage distribution of Rock classes in Diversion tunnel

Class	[%]
A	0-5
B	15-25
C	50-60
D	10-20
E	10-20

5.3.9.5 Vertical shaft

Table 5-43: Percentage distribution of Rock classes in Headrace tunnel

Class	[%]
A	0-5
B	10-20
C	50-60
D	10-20
E	10-20

5.3.9.6 Powerhouse, transformer cavern and downstream surge tank

Table 5-44: Percentage distribution of Rock classes in Powerhouse

Class	[%]
A	0-5
B	20-30
C	50-60
D	10-20
E	10-20

5.3.9.7 Tailrace tunnel

Table 5-45: Percentage distribution of Rock classes in Tailrace

Class	[%]
A	0-5
B	40-50
C	40-50
D	5-10
E	5-10

5.4 Seismicity

The Seismicity of the project site is covered in detail in the Seismic Hazard analysis report attached as an Annex 06 to this report.

As an excerpt to the seismic hazard analysis of the project site following summary can be drawn:

The level for MCE and OBE for the dam site cannot be determined by scientific criteria only, but rather by socio-political decisions. The higher the selected level, the lower will be the accepted residual risk.

The recommended annual probability of exceedance for Maximum credible earthquake (MCE) is 1 / 10,000. This value is compatible with the recommendations of ICOLD and is commonly used in civil engineering practice. Lower values are only recommended if a risk assessment for the downstream area shows only marginal risks in case of a dam failure.

For Operating basis earthquake (OBE), considerably lower values for the annual probability of exceedance are recommended in the ICOLD Bulletin 148 (2016) and 72 (ICOLD 1989), in the order of 1 / 145 years.

The selection of an annual probability of exceedance for the OBE level depends on the residual risk that the client is willing to take. The OBE level is not safety-related as it is in the MCE case, but determines the functionality of the structure after an earthquake event. So the selection of the OBE level is a management decision which can be taken by the utility. In this report, two possibilities are shown:

- "OBE 1" level corresponding to an annual probability of exceedance of 1 / 475, thus corresponding to the earthquake action on structures of in modern building codes
- "OBE 2" level corresponding to an annual probability of exceedance of 1 / 145, thus accepting a higher residual risk compared to the OBE 1 level.

It is recommended in taking the "OBE 1" level as the relevant level. It must be pointed out that the earthquake actions on structures in modern building codes are based on an annual probability of 1 / 475. At this level, substantial damage is allowed for structures of low importance (e.g. importance class

III and IV in Eurocode 8, Part 1). In contrast, important or hazardous structures have to remain functional, that means only minor damage is allowed.

Design spectra derived from attenuation laws for different spectral periods Given the annual probabilities of exceedance for MCE (1 / 10,000) and OBE 1 / OBE 2 (1 / 475 or 1 / 145), the relevant spectral accelerations obtained from this study are the weighted values corresponding to Figure 5-33 (for MCE) and Figure 5-34 (for OBE).

Vertical spectra

Vertical spectral values are selected as 2/3 of the horizontal values, as is common international practice if not determined otherwise.

Final horizontal and vertical spectra

The final spectra for MCE, OBE 1 and OBE 2 are given in Figure 5-33 to Figure 5-36, as well as in Table 5-46 and Table 5-47.

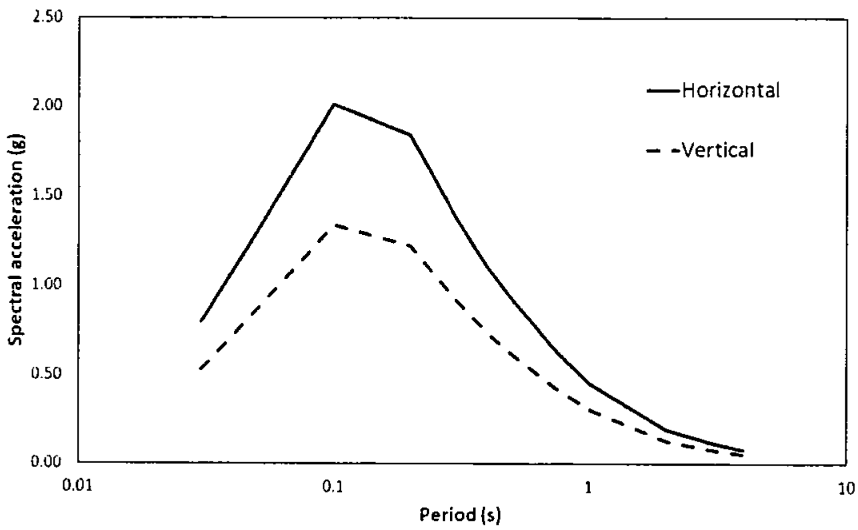


Figure 5-33: Acceleration response spectra for horizontal and vertical components of MCE (with 1 / 10,000 annual probability of exceedance) for SHPP (5% damping)

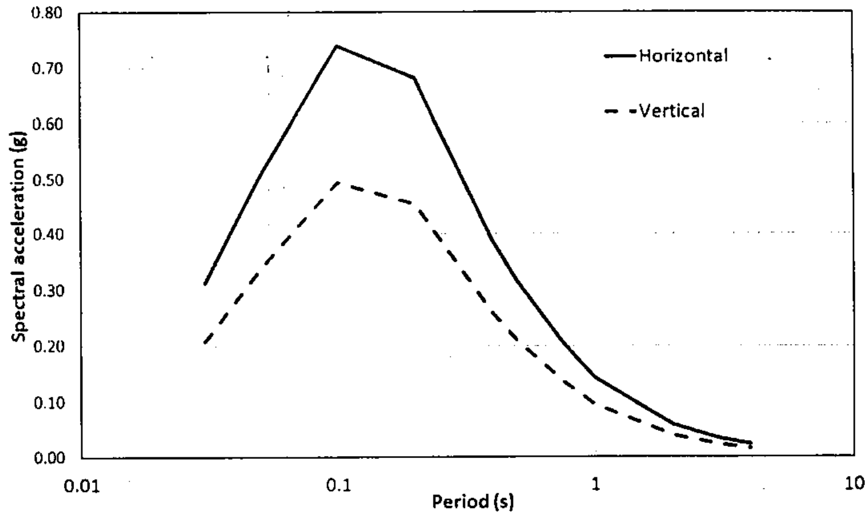


Figure 5-34: Acceleration response spectra for horizontal and vertical components of OBE 1 (with 1 / 475 annual probability of exceedance) for SHPP (5% damping)

Additionally, design spectra is given for 975 and 2474.9 years return periods in the following Figure 5-35 and Figure 5-36 and Table 5-46 and Table 5-47.

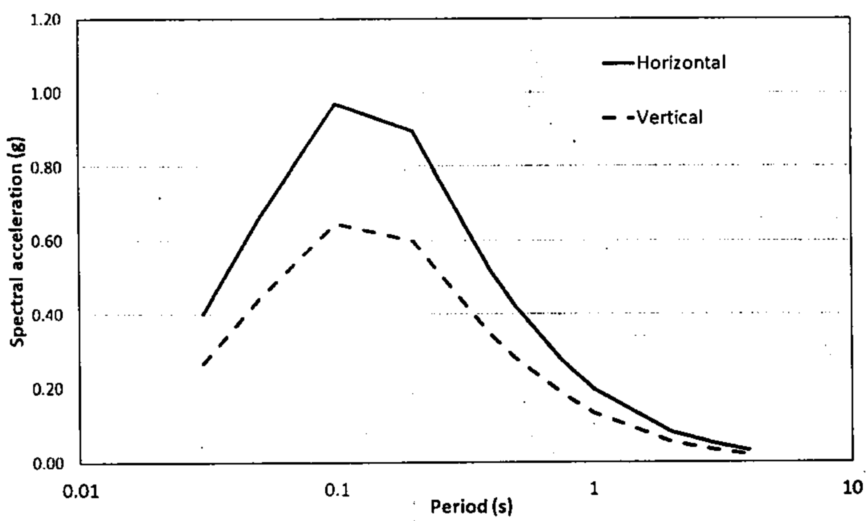


Figure 5-35: Acceleration response spectra for horizontal and vertical components with 1 / 975 annual probability of exceedance for SHPP (5% damping)

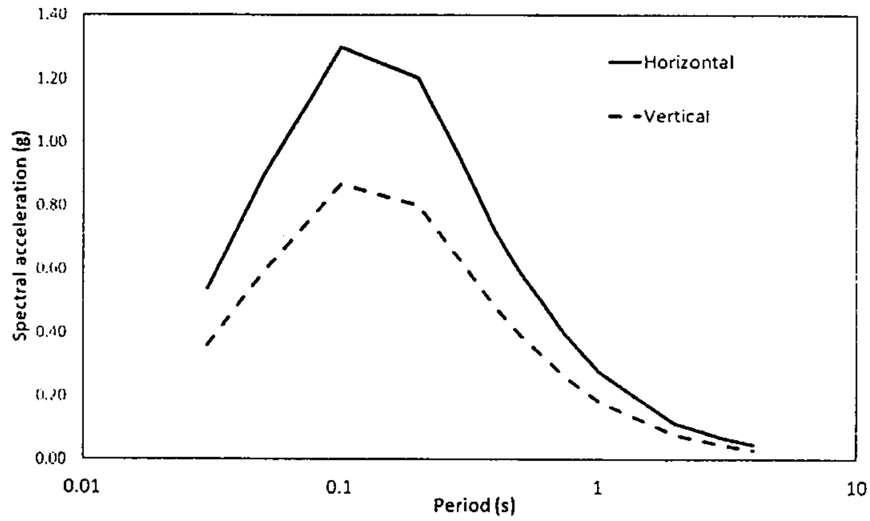


Figure 5-36: Acceleration response spectra for horizontal and vertical components with 1 / 2475 annual probability of exceedance for SHPP (5% damping)

The final spectra are given in Table 5-46 and Table 5-47, as fraction of the gravitational acceleration g for different spectral periods.

Table 5-46: Design Spectra for MCE

	MCE spectrum (an. prob. of exceedance 1 / 10,000)	
Period [s]	horizontal [g]	vertical [g]
0.01 (PGA)	0.795	0.530
0.05	1.301	0.867
0.10	2.012	1.341
0.20	1.842	1.228
0.30	1.392	0.928
0.40	1.105	0.737
0.50	0.925	0.617
0.75	0.630	0.420
1.00	0.457	0.305
2.00	0.191	0.127
3.00	0.115	0.076
4.00	0.078	0.052

Table 5-47: Design Spectra for OBE

	OBE 1 spectrum (annual probability of exceedance 1 / 475)		OBE 2 spectrum (annual probability of exceedance 1 / 145)	
Period [s]	horizontal [g]	vertical [g]	horizontal [g]	vertical [g]
0.01 (PGA)	0.311	0.207	0.191	0.127
0.05	0.506	0.337	0.308	0.205
0.10	0.740	0.493	0.444	0.296
0.20	0.681	0.454	0.405	0.270
0.30	0.511	0.340	0.302	0.201
0.40	0.390	0.260	0.228	0.152
0.50	0.316	0.211	0.184	0.123
0.75	0.206	0.137	0.118	0.079
1.00	0.143	0.096	0.083	0.055
2.00	0.059	0.039	0.034	0.022
3.00	0.034	0.023	0.020	0.013
4.00	0.023	0.015	0.013	0.009

The PGA values for design are summarized as follows in Table 5-48:

Table 5-48: PGA values for different return period

Return Period	Horizontal acceleration (g)	Vertical acceleration (g)
145 years (OBE)	0.191	0.127
475 years (OBE)	0.311	0.207
2475 years (SEE)	0.275	0.183
10000 years (MCE)	0.795	0.530

5.5 Construction Material Survey

Construction material survey encompasses identification of borrow areas by visual estimation and sampling for lab tests. Different construction materials, their description and calculation of tentative volume were carried out. The detail results of the lab tests are attached as Annex 04.

5.5.1 Construction Material Study

The construction material study was carried out for following purposes.

- Potential site identification and estimation of fine aggregates quantity and its quality in vicinity of construction sites.
- Site identification and determination of relevant properties of materials available along river beds that can be used as concrete aggregates.

- Identification of quarry sites as an alternate source of concrete aggregates.
- Field investigations include test pitting and sampling in the borrow area and collection of quarry materials to determine their suitability as concrete aggregates.
- Sample collection and test in laboratory to determine their properties.

5.5.2 Field investigations

Field investigations were carried out to locate borrow and quarry sites for construction materials. Quarry areas are primarily identified for sand material and coarse aggregates. A short description of borrow area and quarry site is given following sections.

5.5.2.1 Rock quarry site

For the rock quarry, different locations were identified and samples were extracted. The extracted samples were sent to the Lab for the chemical, physical and mechanical suitability tests. The coordinates and other details of the rock quarry sites are tabulated as follows:

Table 5-49: Location of Quarry

Sample No.	Site	Easting	Northing	Location	Sample type
1	1a	226330	3867735	Toormang (Lower Dir) River Panjkora	Silty Sandy gravely Cobbles, boulders
2	1b	226632	3867812	Toormang (Lower Dir) River Panjkora	Silty Sandy gravely Cobbles
3	2a	227438	3869529	Khakgram (Upper Dir) River Panjkora	Silty Sandy gravely Cobbles
4	2b	227474	3869791	Khakgram(Upper Dir) River Panjkora	Silty Sandy gravely Cobbles
5	3a	772450	3884100	Sahib Abad (Upper Dir) River Panjkora	Silty Sandy gravely Cobbles
6	3b	773231	3883583	Sahib Abad (Upper Dir) River Panjkora	Silty Sandy gravely Cobbles
7	4a	772067	3888285	Darora (Dir Upper) River Panjkora	Silty Sandy gravely Cobbles
8	4b	771942	3887385	Darora (Upper Dir) River Panjkora	Silty Sandy gravely Cobbles
9		769730	3897711	Intake Area	Quartz Mica Gneiss
10		772450	3889669	Surge Tank & PH Area	Tonalite

Sample No.	Site	Easting	Northing	Location	Sample type
11		771743	3889183	Confluence of the rivers	Granite

Following tests were carried out on the collected samples at the CMTL lab:

Table 5-50: Lab test performed on the quarry samples

Item	Works Description	Standards
1	Definitions of constituents and Aggregate constituents (Petrographic examination)- qualitative test	ASTM C294/ C125 / ASTM C40/ C87/ C117/ C123/ C142/ C295
2	Potential Alkali Aggregate Reactivity- quantitative test	ASTM C1260
3	Grading	ASTM C136
4	Los Angeles abrasion test	ASTM C131
5	Soundness test	ASTM C88-05
6	Specific gravity, Water absorption and surface moisture	ASTM C128
7	Bulk density	ASTM C29

As per petrographic analysis, which is qualitative analysis for the aggregate, apart from samples from site 3a, 3b and 4a, other samples are prone to potential alkali aggregate reactivity. Therefore, the other samples cannot be used directly for construction purpose without any additional measures for e.g.: using low alkali cement, slag cement or using additives with prior testing. However, the 16 days quantitative alkali reactivity test carried out on these samples based on ASTM C136, all of these samples were within the specified limit.

Table 5-51: Summary of laboratory test results on samples for quarry

Sample No.	Site	Sp. Gravity	Absorption %	Bulk density g/cm³	LA %	Grain size distribution %		
						Gravel	Sand	Fines
1	1a	2.79	0.64	3.01		100	0	0
2	1b	2.85	0.51	2.78		100	0	0
3	2a	2.80	0.56	2.79		100	0	0
4	2b			2.74		100	0	0
5	3a	2.79	0.64	2.77	25			
6	3b	2.80	0.65	3.04	25			
7	4a	2.85	0.52	2.79	26			
8	4b	2.77	0.51	2.93	31			
9				2.74	25			
10				2.74	36			
11				1.99	24			

The results show that the samples from site 3a, 3b and 4a are chemically, mechanically and physically suitable for the construction purpose. However, the samples from the other sites do not fulfill the petrographic criteria related to the potential aggregate reactivity.

5.5.2.2 Disposal sites

There are many possible areas along the river terraces of the Panjkhora River to be used as muck disposal sites. However, final arrangements and location of disposal site needs to be finalized during construction.

5.6 Conclusions

Based on geological, engineering geological and geotechnical studies, their analysis and interpretation following conclusions are made.

- The project area lies in the Kohistan magmatic arc of northern Pakistan
- Surface geological mapping, Geotechnical investigation in the form of core drillings (17 drill holes), Lugeon tests and laboratory tests and geophysical seismic refraction study were performed for this feasibility study.
- Rock types of the project area are broadly classified into Sub-Phyllites/Slates, Quartz mica Gneiss, Impure marble, Granite, Tonalite, Epidosite, greenish grey Epidote, Quartz Mica Epidote Gneiss and Para-Amphibilite
- Quartz mica Gneiss is abundant at Dam site, Diversion tunnel and about half of the head race tunnel alignment.
- The quartz mica Gneiss rock is light gray to dark gray, medium-banded to massive, strong and moderately weathered with random joints and fractures. The quartz mica gneiss measured shows a foliation and in general three joint sets.
- The rocks encountered in bore holes shows in general joints having partly open to very tight aperture (between 0.5 mm-<0.1 mm) with quartz or clayey matrix and roughness varying between undulating rough to planar smooth.
- From half of the tunnel alignment till the surge tank, vertical shaft, power house and tail race, the surface mapping showed majority of presence of light gray Tonalite. The discontinuity information recorded at site showed in general 3 joint sets having partly open to very tight aperture (between 0.5 mm-<0.1 mm) with quartz or clayey matrix and roughness varying between undulating rough to planar smooth.
- No major structural disturbances such as fault and major shear zones were observed in and around the vicinity of major structures
- Assessment of stability of an existing slope reveals no evidence of slope failure or fault identification in vicinity. However, in due course of construction planning and detail design works the area needs to be investigated thoroughly and Precaution measures should be adopted on civil works. Due to the location of the site in tectonic region, shear zones may encounter in the area although they are not exposed in the surface.

- The planned concrete dam has its foundation in overburden. The thickness of the overburden as per borelogs and geophysical studies varies from 6 m-10 m along the left bank up to 20 m-25 m in the middle of the river and 15 m-20 m to the right bank
- To capsule the permeable openings of overburden at dam site foundation, grout curtain is recommended. This however needs to be verified during the time of construction based on the test grouting and permeability tests at these sections. In case of high permeability of the rock, second row of grout curtain should be applied on the same plane as the first one.
- In total 57 Lugeon tests were carried out in the drill holes BH1 to BH19 to determine the permeability of the rock mass in situ. The majority of the Lugeon value carried out at different depths except showed values less than 3. As per the results the rock can be classified as having very low permeability and the corresponding rock discontinuity to be very tight. Apart from this, the pressure flow (p-q) diagram indicated most of the tests to lie in the range of turbulent flow region. As per Houlsby's (1976) interpretation, this indicates that the hydraulic conductivity of the rock mass decreases as the water pressure increases, which characterize the behavior of rock masses exhibiting partly open to moderately wide cracks.
- The rock cover analysis shows that for all tunnel sections, the factor of safety greater than required.
- As per the computed RQD, Q-value, RMR and GSI values, all of the rocks in general as NGI (Norwegian geotechnical institute) can be categorized in the range from very poor to good quality.
- Preliminary support design for tunnel has been done based on the RQD, RMR and Q-values and the recommendation chart as per NGI.
- Shear zones if present are usually characterized by shear/crushed rock with bands different rock soil units with certain trend of foliation plane. These shear zones are usually the result of the tectonic activity. Although, the project region lies in the vicinity between the two-major tectonic thrust zones namely MKT and MMT, no such marked shear zones were identified during the course of geological and geotechnical investigation of SHPP. However, the presence of shear zones might be possible.
- Due to difficult terrain and security situation, the tunnel portion between the intake and the surge tank (about 5-6 km) could not be investigated in detail. Therefore, for this section the study is based mostly on the available investigation in the vicinity and the available literature. In due course of construction or further investigations, the geology of this area needs to be investigated in detail.
- In order to obtain rock parameters like minimum principal stress values which are very important for the design and analysis of tunnel, in situ hydro fracturing/ hydro jacking/ 3D-overcoring tests were recommended from the very beginning. However, the tests could not be performed owing to its unavailability in Pakistan. Therefore, simple assumptions were made for the computation of the stresses needed to acquire the rock mass stress and strength values.

- For the seismic studies, both deterministic and probabilistic studies were carried out and horizontal and vertical seismic parameters for the OBE and MCE were recommended together with the spectral curves.
- The petrographic studies (qualitative studies) carried out on the quarry material obtained from the existing SHPP site revealed that that the samples are not suitable for the construction based on potential alkali/silica aggresivity. However, the quantitative 16 day ASTM 1260 tests showed that these samples have no alkali aggresivity potential.
- As per CMTL lab, Pakistan based on their petrographic study only the samples from obtained from Sahib Abad (Upper Dir) River Panjkora and a par of Darora (Dir Upper) River Panjkora are suitable as quarry for the concreting works.

6. Layout Optimization

6.1 Objective

This section summarizes the findings of the analyses performed in order to identify the optimum project configuration under the prevailing boundary conditions.

The discussion on the general layout for Sharmai project was partially presented in the Inception stage. The present section provides results of the additional and more accurate analyses, based on the detailed topographic maps, geological, hydrological and sediment studies and preliminary design considerations. The analyses reported under this chapter are accorded to the conclusions and constraints as per the Phase 1 ESIA Report, prepared by Hagler Bailly, the ESIA consultant.

The Optimization analyses are intended to identify the comparatively best project configuration, whereas the optimization of the design of the individual project components will be performed for the selected conceptual layout.

6.2 Design Criteria

6.2.1 General

The purpose of this chapter is to define the basis for feasibility-level design of all major project components:

- Dam
- Spillway and energy dissipater
- Power waterways
- Powerhouse
- River diversion works
- Miscellaneous other works.

6.2.2 Codes, Standards and References

The following codes, standards and references were applied during elaboration of design.

6.2.2.1 USACE Engineering Manuals

Hydrology

- EM 1110-2-1415 Hydrologic Frequency Analysis
- EM 1110-2-1417 Flood Run-off Analysis
- EM 1110-2-1419 Hydrologic Engineering Requirements for Flood
- Damage Reduction Studies

- EM 1110-2-1420 Hydrologic Engineering Requirements for Reservoirs
- ER 1110-8-2(FR) Inflow Design Floods for Dams and Reservoirs

Dam, Spillway and Energy Dissipation
- EM 1110-2-1603 Hydraulic Design of Spillways
- EM 1110-2-2000 Standard Practice for Concrete for Civil Works Structures
- EM 1110-2-2100 Stability Analysis of Concrete Structures
- EM 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures
- EM 1110-2-2300 Engineering and Design – General Design and Construction Considerations for Earth & Rock-Fill Dams.
- EM 1110-2-1902 Slope Stability.
- DIN 19700-11:2004-07 Dam plants - part 11: Dams
- USACE Engineering Manual EM 1110-2-1902 Slope Stability.
- USACE Engineering Manual EM 1110-1-2908 Rock Foundations
- USACE Engineering Manual EM 1110-2-2300 Engineering and Design – General Design and Construction Considerations for Earth & Rock-Fill Dams.

Outlets and Ancillary Structures
- EM 1110-1-1905 Bearing Capacity of Soils
- EM 1110-1-2908 Rock Foundations
- EM 1110-2-1601 Hydraulic Design of Flood Control Channels
- EM 1110-2-1602 Hydraulic Design of Reservoir Outlet Works
- EM 1110-2-2000 Standard Practice for Concrete for Civil Works Structures
- EM 1110-2-2100 Stability Analysis of Concrete Structures
- EM 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures
- EM 1110-2-2400 Structural Design and Evaluation of Outlet Works.

6.2.2.2 ICOLD Bulletins

- No. 46 – Seismicity and Dam Design
- No. 49 – Operations of Hydraulic Structures of Dams
- No. 58 – Spillways for Dams
- No. 67 – Sedimentation Control of Reservoirs
- No. 72 – Selecting Seismic Parameters for Large Dams
- No. 81 – Spillways – Shockwaves and Air-Entrainment: Review and Recommendations
- No. 82 – Selection of Design Flood
- No. 88 – Rock Foundations for Dams
- No. 92 - Rock Materials for Rockfill Dams - Review and recommendations, 1993.
- No.141 - Concrete Face Rockfill Dams: Concepts for design and construction, 2010.

6.2.2.3 Other references

- Smoltczyk et al. (Editors). Geotechnical Engineering Handbook. Volume 1-3. Ernst & Sohn, 1990.
- Fell, Robin; Macgregor, Patrick; Stapledon, David & Graeme Bell (2005): "Geotechnical Engineering of Dams".A. A Balkema Publishers.
- Fell, Robin; Macgregor, Patrick; Stapledon, David; Graeme, Bell and Foster, Mark (2015): "Geotechnical Engineering of Dams - Second Edition".A. A Balkema Publishers.
- Hynes-Griffin, M.E. & Franklin, A.G. (1984): Rationalizing the Seismic Coefficient Method. Department of the Army.Waterways Experiment Station, Corps of Engineers. Prepared for the USACE.

6.2.3 Design criteria

For the feasibility design the following hydraulic and civil design criteria have been established:

6.2.3.1 Design floods

ICOLD Bulletin 82 "Selection of Design Flood" discusses methods used in the calculation of floods, and discusses factors in the selection of an appropriate "design flood". It also describes the methods and criteria used in different countries. It is currently accepted practice in many countries (including USA, Australia, India, UK), that large dams classified as "high-hazard" i.e. where human life would be lost as a result of dam failure, should be designed on the basis of a flood with a high return period (1,000 year flood, 10,000 year flood, or even the Probable Maximum Flood (PMF)).

Design floods are defined for the following purposes and structures:

Weir – Spillway:	Design Flood	(according to ICOLD)
	Safety Check Flood	(according to ICOLD)
Powerhouse:	Design Flood	
	Design Discharge at Rated Operation Conditions	
Diversion Floods:	Design Flood	

6.2.3.2 Spillway capacity

In view of the size of the weir structure and consequences of potential failure the following design floods are considered adequate as a conservative approach in accordance with the recommendations of ICOLD-Bulletin 82: "*Selection of Design Flood – Current Methods*".

For details regarding the methodology applied to the calculation of the design floods reference is given to the report on hydrology:

Design Flood:		$HQ_{10,000}$, with all gates open and provision of flood freeboard
	or	
		$HQ_{1,000}$, with *n-1* gates open and provision of flood freeboard
Safety Check Flood:		HQ_{PMF}, with all gates open and without provision of flood freeboard

The Design Flood represents the inflow which must be discharged under regular operating conditions with a safety margin provided by freeboard. In the event of the design flood the reservoir is assumed to be at normal operation water level (full reservoir) with the stipulation for conserving freeboard. Also, it is assumed that one spillway gate is jammed in closed position (n-1 condition) during routing of the design flood.

The Safety Check Flood is the flow for which the crest structure, waterway and the energy dissipater may be on verge of failure, but exhibit marginally safe performance characteristics for such flood condition.

The ratio of maximum head to spillway design head shall not exceed 1.3 in order to limit negative pressure on the ogee according to ASCE Design Guidelines.

6.2.3.3 Powerhouse operation design flood

Design of the powerhouse structure shall ensure the operational ability of the equipment and structures up to the 1,000-year flood event. Erection bay will be elevated above the tailwater level which corresponds to the 1,000y flood, augmented by a necessary freeboard margin (provisionally set to 1 m).

In the event of higher floods the gates at the power intake shall be closed. Powerhouse discharge shall not be considered as separately released through the units during the occurrence of a flood event, i.e. the flood control structures will be designed as if the powerhouse is out of operation at the time of a flood event.

6.2.3.4 Construction period flood

During the construction period, safe evacuation of floods up to the river diversion design flood shall be guaranteed and the diversion waterways shall be designed accordingly. Selection of the diversion design flood is to be made in conjunction with the principle of river diversion applicable to the prevailing topographic, hydrological, geological and other related conditions. The magnitude of the diversion design flood shall consider adequately potential risks and damages for life and goods which may result in the event of a flood of a greater magnitude than the diversion design flood.

The UK Institution of Civil Engineers (ICE) publication "Floods and Reservoir Safety" (3rd Edition, 1996) gives guidance on flood risk during dam construction: "Where an arbitrary criterion is appropriate, it should be acceptable to design for the flood that has only a 10% chance of being exceeded during the critical period of diversion work. The percentage probability of risk (Pr) that a flood peak with a return period of T years would occur within the period of risk (r), i.e. the construction period in years, is given as:

$$Pr/100=1-[1-(1/T)]r$$

For Sharmai HPP dam/headworks, envisioned to be constructed within the period of about 2 years, the design of diversion works shall be such to cope with, not less than:

- a 20-year return period flood.

6.2.3.5 Freeboard

As defined by ICOLD, freeboard is "the vertical distance between a stated water level and the top of a dam". Flood freeboard is the vertical distance between the maximum flood level and the top of the dam. Freeboard is a safety provision, in order to avoid overtopping of the dam by waves. There can be other reasons for providing freeboard, but it is normally only provided as protection against wind-generated waves.

For calculation of freeboard we adopt criteria given in the ''Hydraulic Structures'', Smith, 1995. This publication also includes guidance on determination of wave surcharge. Wave surcharge shall be determined as follows:

$$h_t = 0.00513 {V_w}^{1.06} (KL_0)^{0.47}$$
$$L_t = 0.187 {V_w}^{0.88} (KL_0)^{0.56}$$
$$h_{wave} = 2.5\ x\ h_t$$

where

ht - wave height
Vw - referent wind velocity
K - coefficient, in function of fetch length and width of the reservoir
Lo - maximum fetch length
Hwave - wave upsurge

6.2.3.6 Spillway design

As described in ICOLD Bulletin 58 "Spillways for Dams", there are two broad classes of spillways: surface spillways and submerged spillways. The main factors in the choice of spillway type, for a given project, include:

(a) Reliability and accuracy of flood prediction;
(b) Topography and geology; and
(c) Dam type.

According to ASCE design guidelines the shape of the spillway ogee can be defined for a design head less than the maximum head H_0. Selection of the design head shall account for a possible additional head of up to 30% in case of the Safety Check Flood, however, sub-atmospheric pressure on the ogee shall not fall below -4 m water column.

The design of the spillway downstream of the crest shall comply with international standards such as e.g. ASCE. The thickness of piers shall be selected to safely transfer forces (e.g. gate trunnion forces) into the main dam body.

The pier shape shall be selected in a way to avoid separation of flow and to guarantee minimum head losses.

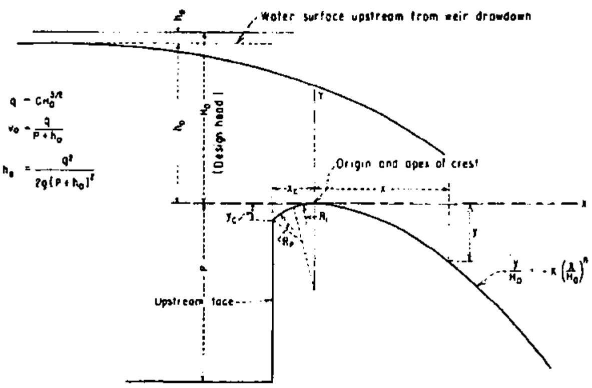


Figure 6-1: Design Chart for Spillway Ogee

The ogee crest structure is designed applying WES standard profile as defined by the Hydraulic Design Charts by USACE for the equation downstream of the crest axis.

$$\frac{Y}{H_d} = K\left(\frac{X}{H_d}\right)^n$$

Where,

X	=	horizontal distance in downstream direction
Y	=	vertical distance from crest level
H_d	=	Spillway Design Head
K,n	=	Factors defining the nappe-shape of crest
K	=	Variable depending upon upstream slope, 0.5
n	=	Variable depending upon upstream slope, 1.835 in this case

By putting the variables in above equation; $X^{1.835} = 2.0 \times H_d^{0.835} \times Y$

The shape of the spillway pier upstream and downstream faces is to be selected to guarantee a high discharge capacity and limitation of the height of downstream rooster (shock) waves. For the design of the spillway crest structure and calculation of the discharge capacity the following effect shall be taken adequately into account:

- effect of head on overfall coefficient and hydraulic effective width
- effect of abutment and pier shape on hydraulic effective width.

The spillway discharge capacity is calculated applying the following standard formula:

$$Q = CB'\sqrt{2gH_e^{\,3}}$$

Where,	C	=	Discharge coefficient (0.48)
	B'	=	Effective width
	H_e	=	Head over the crest

The effect of piers and abutments on the hydraulically effective crest width and thus on the spillway discharge capacity is estimated using the following relationship:

$$B = B' - 2(n \times k_p + k_a) \times H_e$$

Where,	B'	=	Effective width
	B	=	Clear waterway width
	n	=	Number of piers
	k_p	=	Pier contraction coefficient
	k_a	=	Abutment contraction coefficient
	H_e	=	Head above crest level

The piers and abutments cause side contraction of the overflow, thus reducing the effective width of the spillway bay, also depending on the head over the crest. The contraction coefficients K_p and K_a are affected by the shape of the pier nose and abutment shape respectively. Rounded pier nose shape is recommended at this stage of design, whereas the radius of the upstream pier curve equals to 50% of the pier's width. The coefficients K_p=0.01 and K_a=0.1 apply.

6.2.3.7 Design of energy dissipation structure

The riverbed of Panjkora River consists of large scale boulders and rock outcrops, not prone to erosion. Nevertheless, a stilling basin is envisioned as the structure to dissipate the energy and limit the extents of the possibly excessive riverbed and bank erosion.

Energy dissipation at the toe of the concrete ogee shall be effected by means of hydraulic jump basins. Standard USBR design for dimensioning is adopted, as summarized in the table below:

Table 6-1: Standard USBR stilling basin types

Froude number, F	Velocity (m/s)	Stilling Basin Type
F < 1.7	-	No basin required
1.7 < F < 2.5	-	USBR Type I
2.5 < F < 4.5	-	USBR Type IV
F > 4.5	V < 15 m/s	USBR Type III
F > 4.5	15 m/s < V < 46 m/s	USBR Type II

$$F = v/\sqrt{gh1}$$

Where,

F	=	Froude number
v	=	velocity at the entrance, m/s
h1	=	first conjugated depth, m

To ensure that the hydraulic jump is maintained within the stilling basin for the entire range of river discharges the elevation of the end sill is selected with an additional submergence safety factor of 1.05.

The length of the stilling basin shall be designed according to common design approaches such as that reported by USBR. Accordingly the minimum length of the stilling basin is to be calculated as a linear function of the second conjugated depth:

$$L_{SB} = Lc \times h2$$

Where,

L_{SB}	=	Length of stilling basin, m
Lc	=	coefficient; depending on the selected basin type and the Froude number at the entrance
h_2	=	Conjugated depth at end of hydraulic jump, m

Alternatively, for the conventional stilling basin without baffles, Lc = 5.5 shall be considered. The friction losses along the chute will not be considered for design.

6.2.3.8 Reservoir flushing and drawdown

Provisions shall be made that the reservoir flushing can be conducted to remove:

a) floating debris from the top reservoir layers and
b) large depositions of silt and sand fractions during the high flow season, if required.

Reservoir flushing shall be conducted through the operation of the spillway gates and the bottom outlet facility, without the necessity to drawdown the reservoir.

Reservoir flushing facilities will be designed to enable the drawdown of the water level in the reservoir under the average inflow conditions within 5 days if required (e.g. due to safety- or inspection-related reasons).

6.2.3.9 Vortex prevention at the power intake

In order to avoid vortex formation at intakes the submergence must be sufficient. Gordon [Water Power, April 1970] has assembled comprehensive field data and prepared an empirical formula according to the definition given in the figure below for the required minimum submergence.

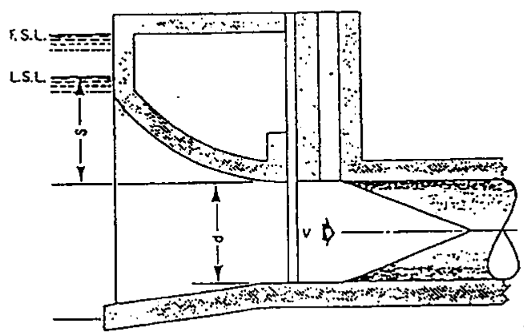


Figure 6-2: General configuration of power intake

Gordon's formula defines: $s = c * v\sqrt{d}$

Where:

s	=	submergence, (m)
c		coefficient; 0.72 for non-symmetrical and 0.54 for symmetrical approach
d	=	diameter of intake section, (m)
v	=	intake flow velocity, (m/s)

The adopted submergence shall be at least 1 m below the level calculated by the Gordon's formula.

6.2.3.10 Desander

To avoid wear and frequent maintenance of the hydraulic steel structures and turbines due to sediment in discharge it is necessary to implement a desander. From the Intake the water is conveyed to the desander. Within the desander the reduction of the flow velocity by increasing the cross-section leads to the settling of gravel and sand.

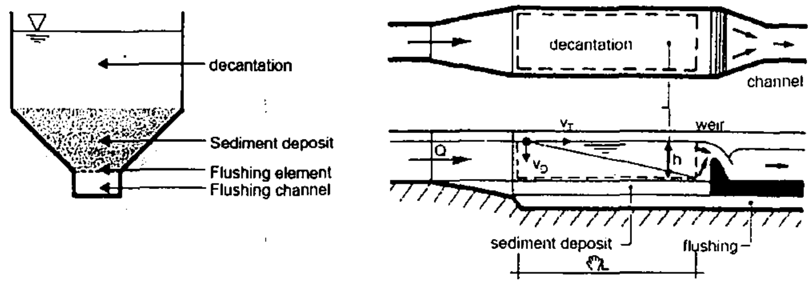


Figure 6-3: Design of Desander, Source: Layman's Guidebook

For the design of a desander the following criteria was applied:

Design Grain Diameter: Critical Sediment Grain Size

Head	20 - 50 m	D = 0.30 mm
Head	50 - 100 m	D = 0.25 mm
Head	100 - 300 m	D = 0.20 mm

The number of desander chambers shall allow a continuous operation of the plant at the rated discharge, assuming one chamber being flushed.

6.2.3.11 Head losses in waterways

Along the waterways the flow passes conduits of different size and shape and stream lines are deflected by bends, dividing or combining flow, such as at trash racks, branches, bends, enlargements and contractions. These local changes of the stream line direction cause an addition head loss to that resulting from frictional resistance. All head losses involved in each conduit system are individually evaluated according to methods and formulas described subsequently.

6.2.3.11.1 Friction caused head losses

The equation used for the calculation of friction losses in the conduit is the Darcy-Weisbach formula:

$$Hf = f \cdot \frac{L}{D} \cdot \frac{v^2}{2g}$$

Where:

H_f	=	Head loss due to friction, (m)
f	=	Friction factor
L	=	Length of conduit or section (m)
D	=	Diameter of conduit (m)
v	=	Velocity of flow, (m/s)
g	=	Gravity acceleration constant, (m/s²)

$\frac{1}{\sqrt{f}} = -2\log\left(\frac{2.51}{Re\sqrt{f}} + \frac{e/D}{3.71}\right)$The friction factor f is determined by means of the Prandtl-Colebrook formula

Where: Re = $\frac{v.D}{\nu}$ Reynolds number
e = Equivalent sand roughness , (m)
ν = Kinematic viscosity, (m^2/s)

The equivalent wall roughness varies from one type of concrete lining to the other within the given limits. For design purposes the value (within the given range) that results in the more critical condition shall be applied. Energy calculation shall be based on mean roughness coefficients.

6.2.3.11.2 Local hydraulic head losses

Intake Loss

The entrance loss for a pipe or tunnel is defined

$$H_L = K_E \cdot \frac{v^2}{2g}$$

Where:

K_E = entrance loss coefficient
v = velocity in tunnel or pipe, (m/s)

For the level of feasibility design a single head loss coefficients is applied to the entire intake structure taking into account the combined effect of entrance, trashrack, gate slots, gradual constriction and expansion of flow as given by the design charts according to ASCE.

In order to reduce the friction losses at the trashrack, the flow velocity at this section shall be 0.8 m/s.

Expansion Loss

The sudden expansion loss is described by Borda's Formula

$$H_L = \left(1 - \frac{A_2}{A_1}\right)^2 \cdot \frac{v_2^2}{2g}$$

Where:

A_1 = area of cross section flow incoming from, (m^2)
A_2 = area of cross section flow is going to, (m^2)
v_2 = velocity in cross section 2, (m/s)

Bend Losses

The head loss produced by a bend with circular cross section is

$$H_L = K_b \frac{v^2}{2g}$$

The head loss coefficient Kb shall be determined based on well established references. In case of subsequent bends, an adequate reduction of head loss coefficients shall be made to account for bend-bend interaction.

6.2.3.12 Surge tank design

Surge tank structure is required in order to facilitate governing and fast start up of turbines fed by tunnels or penstocks. The acceleration time of water within the relevant water conduit can be used as a first indication whether a surge tank is required or not for stable operation of the plant. The acceleration time should not exceed a value of 3 seconds. If the acceleration time exceeds 3 seconds, the requirement for a surge tank is rather high and needs to be investigated in further detail.

The acceleration time is determined as follows:

$$t_s = {\Sigma(L_i \cdot v_i)}/{(H \cdot g)}$$

where

t_s = acceleration time (s)
L_i = length of individual conduit reach (m)
v_i = flow velocity in the individual conduit reach (m/s)
H = head (m)
g = gravity acceleration (m/s^2)

For stability of governing a minimum cross section area (THOMA-Criterion) of the surge tank is required. The Thoma-Criterion calculates as follows:

$$A_{Thoma} = \frac{L * A_t * v_0^2}{2 * g * h_0 * (H_{g,0} - h_0)}$$

Where:

A_{Thoma}	=	Cross section area surge tank according to Thoma (m)
L	=	Length of headrace tunnel (m)
A_t	=	Cross section area headrace tunnel (m²)
v_0^2	=	Stationary velocity in headrace tunnel (m)
g	=	Gravitational acceleration (m²/s)
h_0	=	Losses in headrace tunnel (m)
$H_{g,0}$	=	Minimum gross head at stationary conditions (m)

In order to provide a sufficient safety margin, the actual cross sectional area of the surge tank shall be between 1.5 and 1.8 times larger than the cross sectional area according to Thoma. The surge tank may be provided with an orifice at its bottom. The most common type is a cylindrical surge tank. Lower or upper expansion chambers may be applied. In the event that an orifice is applied its area shall not be less 50 per cent of the pressure shaft or penstock in order to facilitate total pressure wave reflection

In addition, the volume of the surge tank would need to assure the supply of the required volume in case of the startup of the units, whereas the corresponding downsurge in water level oscillations will define the required elevations and geometry of the structure. Numerical simulation of the transient should be performed as soon as the final characteristics of the turbine are fixed.

6.3 Initial considerations on the conceptual layout

6.3.1 Introduction

There are several promising sites identified during the Inception stage of the project, which satisfy the technical requirements (such as topographical, geological, sediment and flow conditions) and the envisioned peaking operating mode of Sharmai HPP and which are not expected to raise meaningful environmental and social concerns.

The following chapters summarize the initial findings; the technically feasible project concepts are further developed, evaluated and compared, as reported in the following sections.

6.3.2 Description of layout alternatives

Sharmai HPP is envisioned to operate as a run-off river plant, with the possibility to act as a daily peaking plant over the low flow period/winter months. Due to the relevance to the national power system, particular attention in design considerations will be given to the maximization of the installed and guaranteed power under the commercially justifiable conditions.

For the purpose of design and considerations on the installed capacity, the 90% guaranteed availability of the capacity shall be regarded as the minimum requirement from the perspective of the national power system. As per the existing project design documents, the project is envisioned as a (daily) storage plant, with a long low-pressure headrace tunnel and a powerhouse.

As concluded during the site visit, due to the sensitive environmental and social issues at the project area, the design of the project components shall be such to minimize the impact to the existing households and arable lands in vicinity of the structures. The areas particularly affected by the project are the reservoir area and the area envisioned for the powerhouse location.

Conceptual layout of Sharmai HPP has been the subject to various analyses since 1992. The general layout of the project was preliminarily elaborated by GTZ (German Agency of Technical Cooperation) in 1992. Further information on design and salient features is available in the Sharmai HPP Project Profile (prepared by PEDO, 2016). Finally, the Site Investigation

Report, Sinohydro, 2017, provides further information and considerations on the project.

The following sections will summarize and compare the available information on the Sharmai HPP.

6.3.2.1 GTZ Report, 1992

The project concept was presented in the report prepared by GTZ in 1992, which considered different alternative options for the utilization of the hydropower potential of Panjkora River over the considered river stretch.

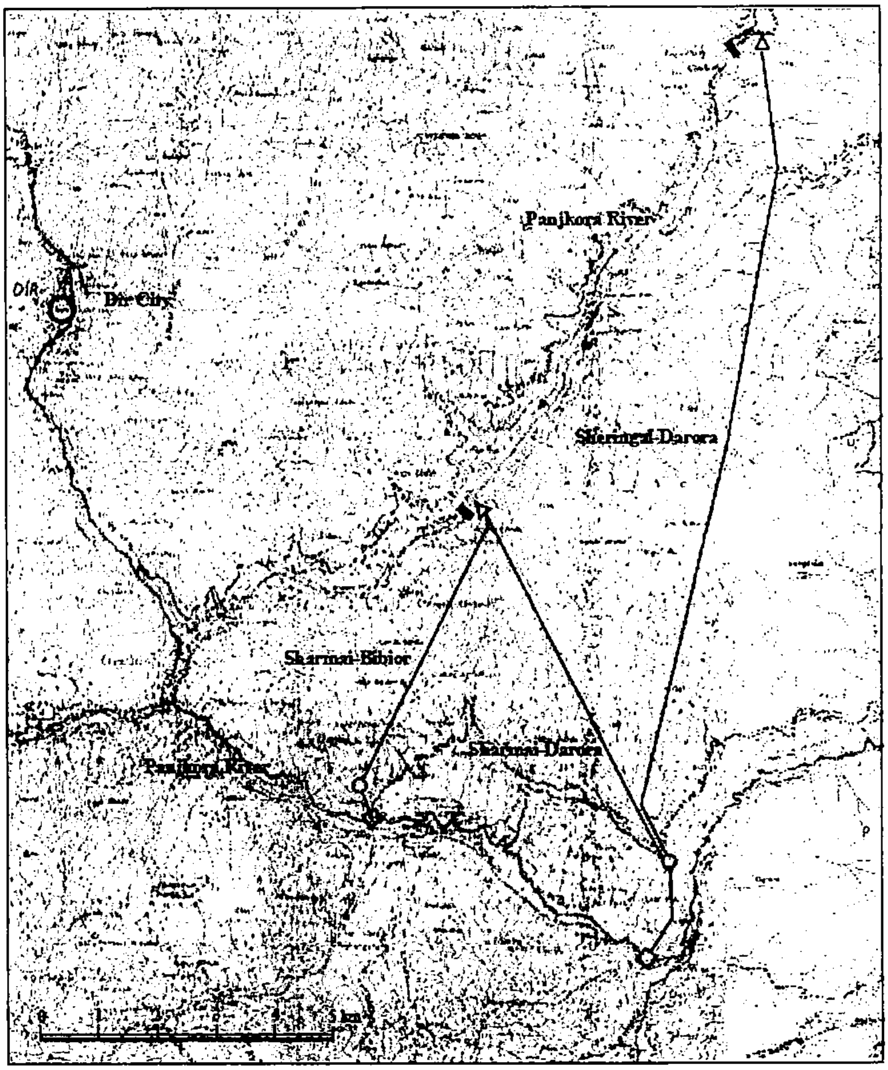


Figure 6-4: Project conceptual layout, source: GTZ Report, 1992

The preliminary design was based on the at the time available topographic maps.

The longitudinal river profile with the considered technical alternatives is shown in the following figure.

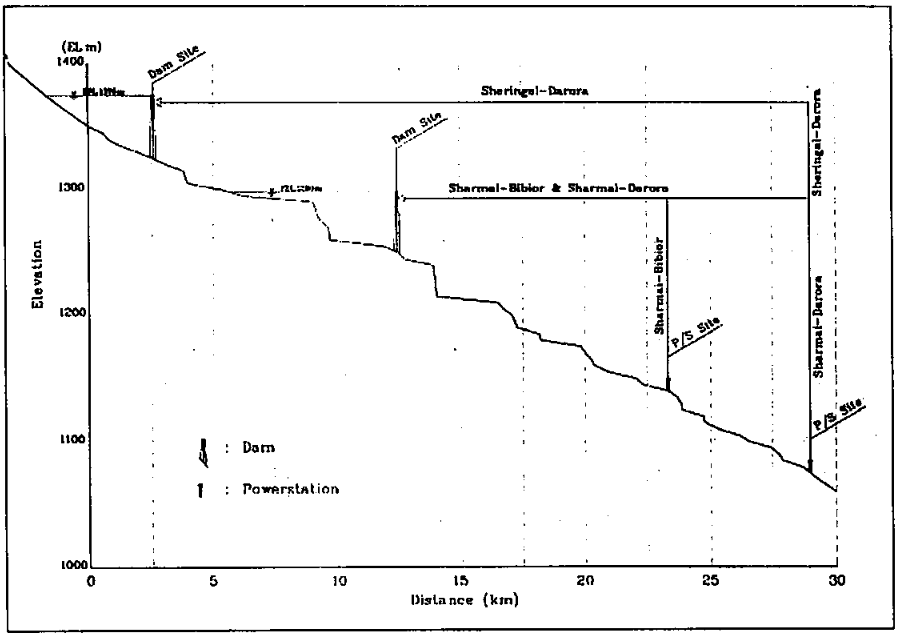


Figure 6-5: Panjkora River longitudinal profile, source: GTZ Report, 1992

A summary table with the project salient features from the GTZ report is given in the table below.

Table 6-2: Project salient features, source: GTZ Report, 1992

Item	Unit	Sheringal-Darora	Sharmai-Bibior	Sharmai-Darora
Energy Generation				
Design Discharge	m³/s	66	69	69
Minimum Discharge	m³/s	37.5	39.1	39.1
Gross Head	m	315	190	220
Minimum Head	m	295	170	200
Net Head	m	290	176	203
Design Capacity	MW	157	102	115
Minimum Availability Capacity	MW	87	52.6	61.6
Mean Energy	GWh	740	475	547
Storage Reservoir				
Catchment Area	km²	1,786	1,915	1,915
Gross Storage Volume	Mil. m³	32.8	8.3	8.3
Effective Storage Volume	Mil. m³	12.0	5.0	5.0
Full Supply Level	EL. m	1,375	1,280	1,280
Minimum Operation Level	El. m	1,355	1,260	1,260
Available Water Depth	m	20	20	20
Surface Area at FSL	ha	219	62	62
Main Dam				
Dam type	-	Rockfill	Rockfill	Rockfill
Crest Length	m	170	115	115
Height of Dam	m	50	40	40
Headrace Tunnel				
Type	-	Concrete Lined	Concrete Lined	Concrete Lined
Number	-	1	1	1
Internal Diameter	m	4.8	4.9	4.9
Length	km	14.1	5.6	6.9
Penstock				
Type	-	Tunnel	Open	Tunnel
Number	-	1	3	1
Internal Diameter	m	4.2	2.5	4.2
Length	km	2.0	0.8	2.0
Powerstation				
Number of Units	-	3	3	3
Type of Turbine	-	Francis	Francis	Francis
Estimated Cost and Economic Indices				
Total Project Cost	Mil. US$	256.0	151.4	161.4
Project Cost per kW	US$/kW	1,631	1,514	1,403
Project Cost per kWh	US$/kWh	0.35	0.32	0.30
EIRR for Alternative Thermal Power	%	13.18	14.03	15.08
Benefit to Cost Ratio	-	1.29	1.37	1.48

The report concluded on the project configuration Sharmai - Darora with the installed capacity of about 115 MW, installed turbine flow of 69 m³/s and the MWL of 1280 masl as the comparatively most promising project layout.

6.3.2.2 PEDO Report, 2016

Sharmai HPP Project Profile (PEDO, 2016) further elaborated the concept as defined in the GTZ report, 1992 and reported on the following project concept and the salient project features:

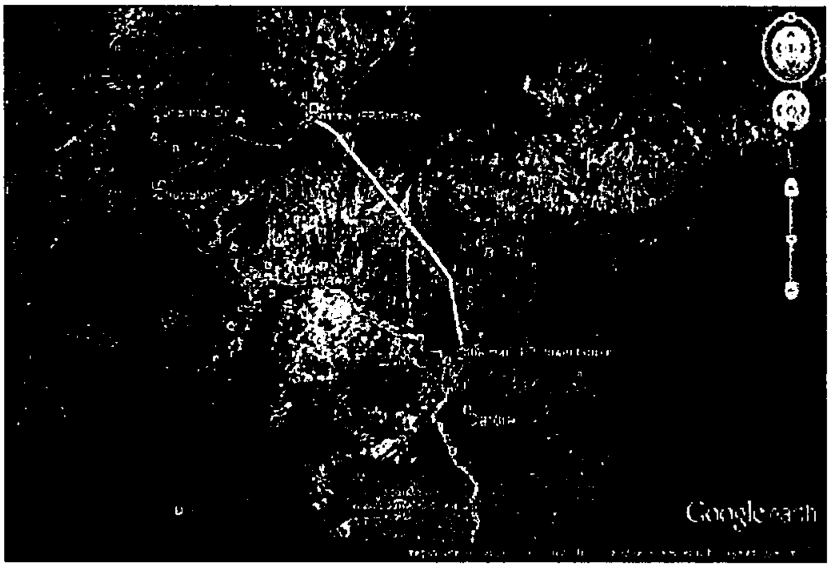


Figure 1: Location Google Map

Source: PEDO Report, 2016

Figure 6-6: Sharmai HPP concept

Table 6-3: Project salient features

Salient Features of the Project

The tentative salient features of the Project are as under: -

General:

- Project Name: Sharmai HPP
- River Name: Panjkora
- Distance: 248 KM From Peshawar
- Status: Raw Site

Technical:

- Capacity: 150 MW
- Design Discharge: 88 Cumecs
- Net Head 193.6 m
- Annual Energy Generation: 682 GWh
- Reservoir Capacity: 32.2 MCM
- Length of Power Tunnel: 7.803 Km
- Catchment Area: 1950 KM^2

Source: PEDO Report, 2016

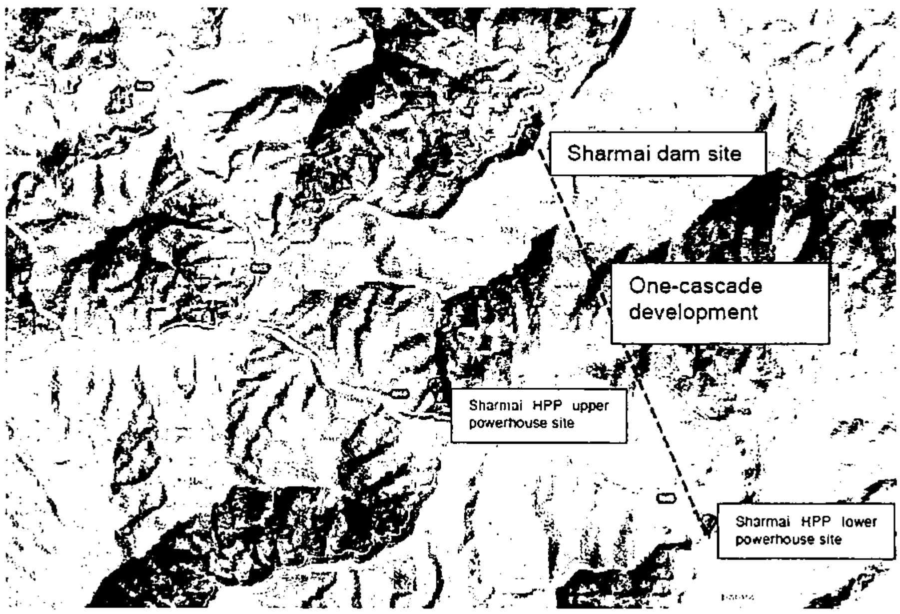
PEDO report concluded on the Sharmai project configuration of about 150 MW as the most viable project layout.

6.3.2.3 Sinohydro Report, 2017

Based on the information available in the Site Investigation Report, Sinohydro, 2017, Sharmai HPP is envisioned with the following general characteristics:

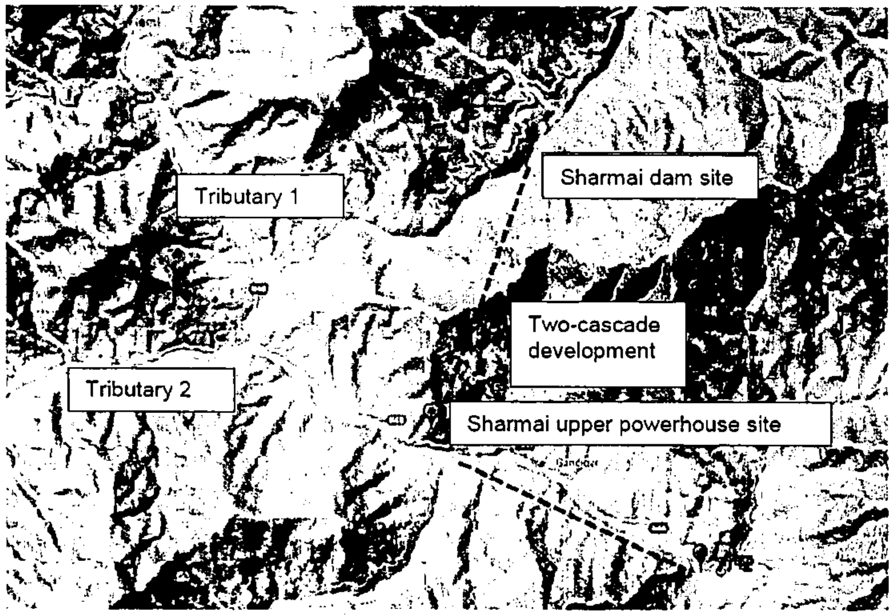
- reservoir with the maximum WL at 1290 masl
- approximately 50 m high dam
- 8 km long headrace tunnel
- surface powerhouse at the confluence.

As a general comment and based on the available topographic data, the elevation of the riverbed at the proposed dam site is about 1220; consequently, the required dam height would be about 75 m from the terrain surface.



Source: Sinohydro Report, 2017

Figure 6-7: Project conceptual layout



Source: Sinohydro Report, 2017

Figure 6-8: Project conceptual layout - cascade alternative

The report summarizes the key features of Sharmai HPP, as presented in the following table.

Table 6-4: Project salient features, source: Sinohydro Report, 2017

Table 5.4 Estimation of installed capacity and energy indexes of the options

Item	Unit	One-cascade development	Two-cascade development	
		Sharmai HPP	Sharmai-1 HPP	Sharmai-2 HPP
Discharge at the dam site	m^3/s	59.5	59.5	82
Normal pool level	m	1290	1290	1117
Minimum operating level	m	1285	1285	1117
Reservoir capacity at normal pool level	$10^6 m^3$	38.43	38.43	-
Reservoir capacity at minimum operating level	$10^6 m^3$	29.74	29.74	-
Normal water level at plant house	m	1060	1117	1060
Effective storage capacity	$10^6 m^3$	8.69	8.69	-
Available head	m	230	173	57
Installed capacity	MW	203	150	70
Number of units	Set	4	4	4
Annual energy output	GWh	801	594	263

6.3.2.4 Summary of existing options

The following table summarizes the key parameters of Sharmai project, referring to the previous studies and analyses.

Table 6-5: Summary of existing project options

Item	Unit	GTZ	PEDO	Sinohydro
Average inflow	[m³/s]	-	-	59.5
Ecological flow	[%]	-	-	-
P90 flow	[m³/s]	-	14.7	-
Installed flow	[m³/s]	69	88	~110
Dam height	[m]	40	53	~50
Reservoir WL	[masl]	1280	1290	1290
Tailwater level	[masl]	1060	1080	1060
Gross head	[m]	220	210	230
Net head	[m]	203	193.6	-
Installed capacity	[MW]	115	150	203
Efficiency*	[-]	0.84	0.90	-
Guaranteed capacity	[MW]	61.6	143	-
Energy production	[GWh/y]	547	682	801
Plant factor	[-]	0.54	0.52	0.45

Although elaborated to a relatively high level of detail, the analyses of the options described above were based on insufficiently accurate topographic data and on general, regional geological information.

A detailed topographic and bathymetric survey mission, followed by the geological field and laboratory investigation campaign was carried out within the scope of the present analyses, in order to bring accuracy and reliability in findings. The key outcomes are summarized in the following chapters.

6.4 Alternatives to the existing project concept

In order to identify the most promising conceptual layout to be further developed, the following key project components of Sharmai HPP were analyzed and compared based on their technical and ESIA performance:

- Dam site
- Dam type
- Powerhouse site
- Powerhouse type.

As the starting point, the dam and powerhouse sites as proposed in the previous project documents were assessed.

Based on the impressions from the site visit and on the conclusions of the Inception stage of the project, fine tuned with the newly obtained topographic data and the conclusions of the Phase 1 ESIA report, the following sites of the individual components and the corresponding conceptual arrangements were identified and further studied and compared:

- 5 dam sites
- 2 powerhouse sites
- 2 powerhouse concepts - surface and cavern.

The following figure illustrates the preliminarily identified options for Sharmai HPP.

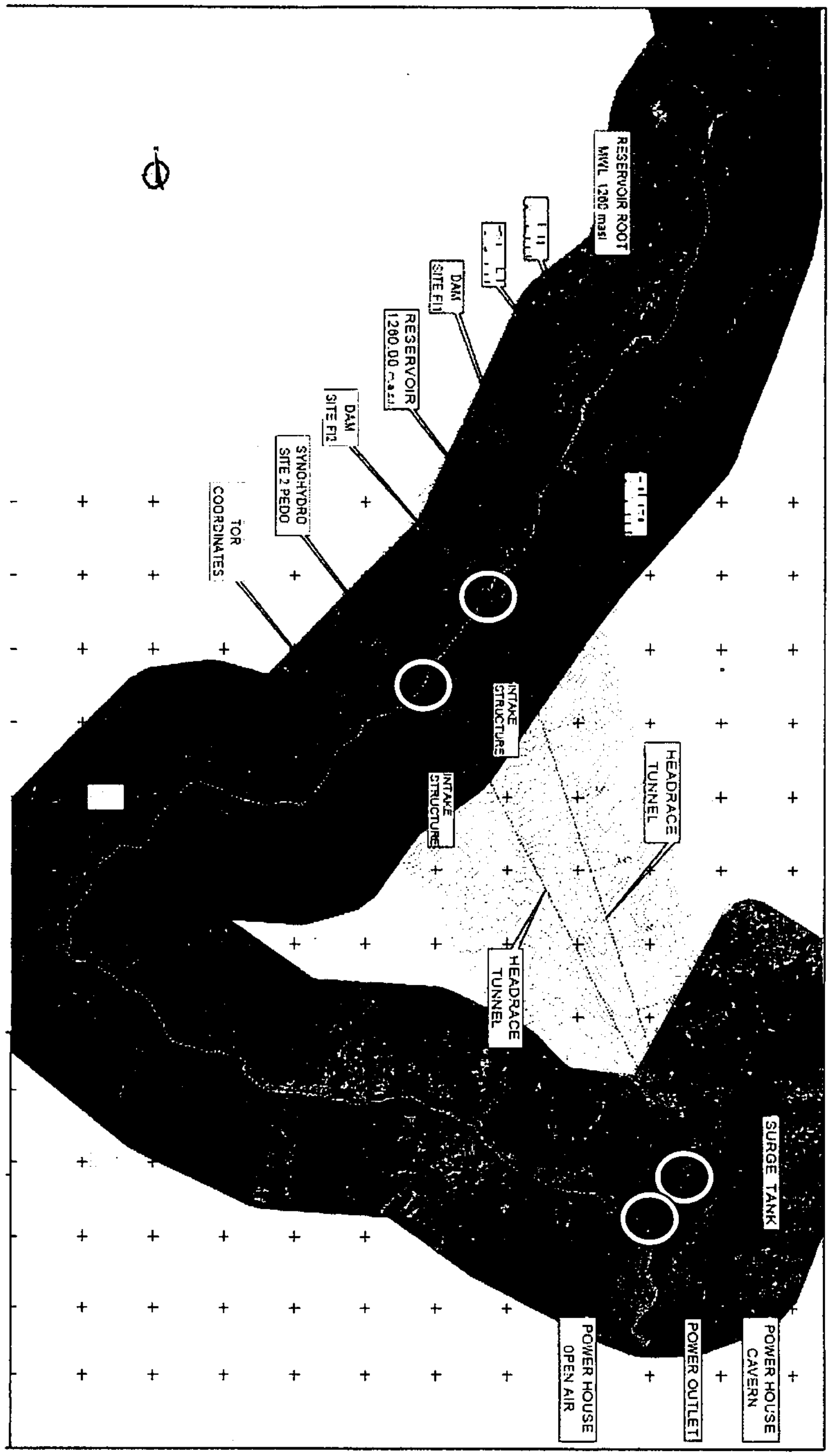


Figure 6-9: Sharmai HPP - sites of interest

6.4.1 Dam site selection

The following dam locations for the dam site were visited and identified during the site visit (in the order from downstream to upstream):

6.4.1.1 Dam site as proposed in the Sharmai HPP Project Profile (PEDO, 2016, TOR Coordinates)

This site is located about 250 m downstream from the existing suspension bridge across Panjkora River. As per the cited document, the coordinates of the dam site are: 35°10'16.28"N, 71°56'33.37"E.

The site is characterized by a wide valley shape with gullies at both river banks and the significant layer of river deposits at the bottom. Neither by terrain morphology nor by the geological features this site may represent a suitable option for further design considerations.

The following figures illustrate the dam site location as proposed in the document referred to above.

Figure 6-10: Dam site as per the coordinates from the Sharmai HPP Project Profile (PEDO, 2016)

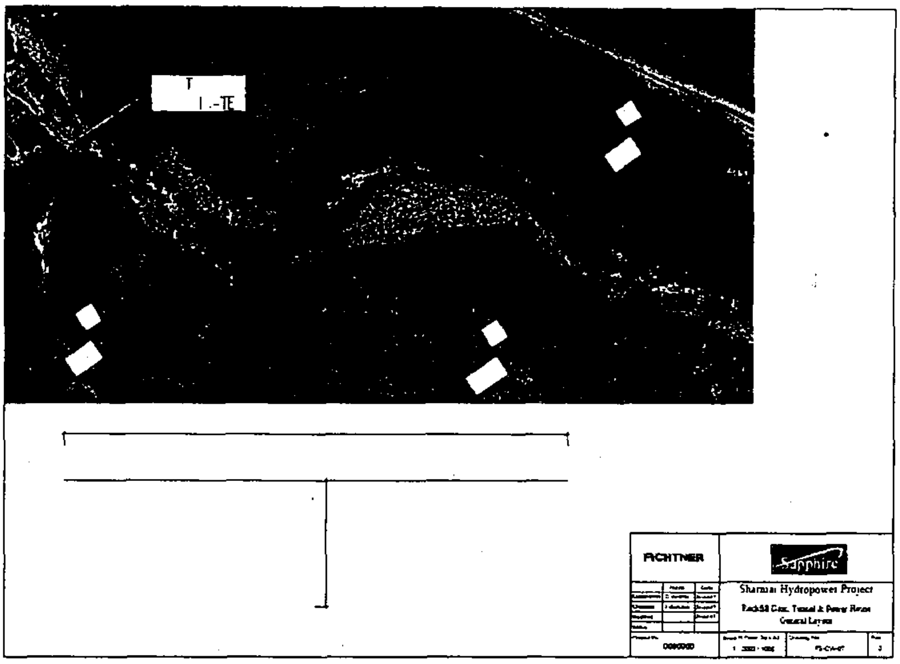


Figure 6-11: Dam site as per the coordinates from the Sharmai HPP Project Profile (PEDO, 2016) (layout and section)

6.4.1.2 Downstream located dam site proposed in the Site Investigation Report (Sinohydro, 2017, Dam Site 1)

This site is located in a narrow part of the river, about 650 m downstream from the suspension bridge.

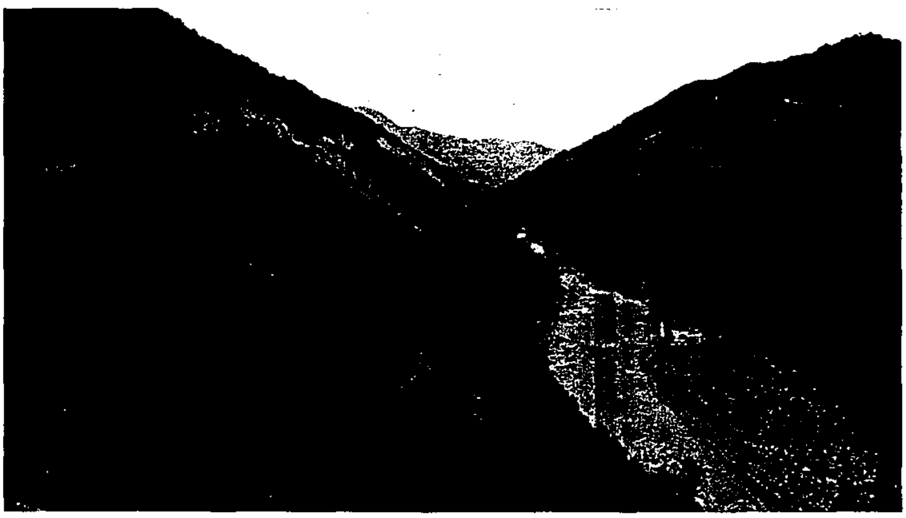


Figure 6-12: Sinohydro dam site 1

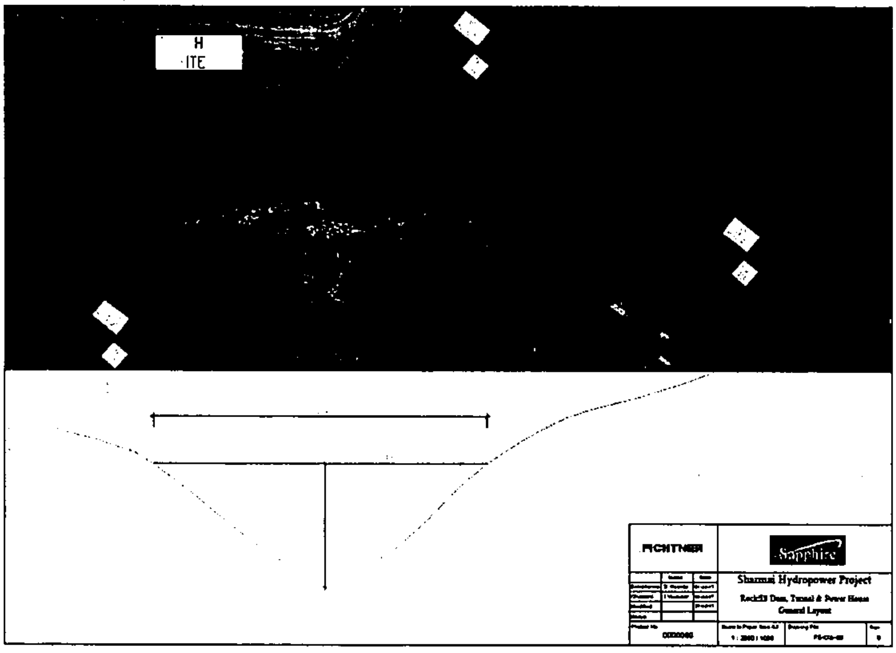


Figure 6-13: Sinohydro dam site 1 (layout and section)

Although that the site conditions seem suitable for construction of the dam, the deposits and the unstable formations at the left bank upstream from the dam site bring in question the suitability of this site for a reservoir with the envisioned prominent daily oscillations in water level. Same conclusion in terms of the suitability of this dam site is arrived to in the cited report.

6.4.1.3 Upstream located dam site proposed in the Site Investigation Report (Sinohydro, 2017, Dam Site 2)

This dam site is located in a narrow part of the river valley, about 750 m upstream from the bridge. This site is at the location of the Sharmai dam site as proposed in GTZ Report, 1992. This site is characterized by a V-shaped valley profile and visible outcrops of competent rock formations at both abutments.

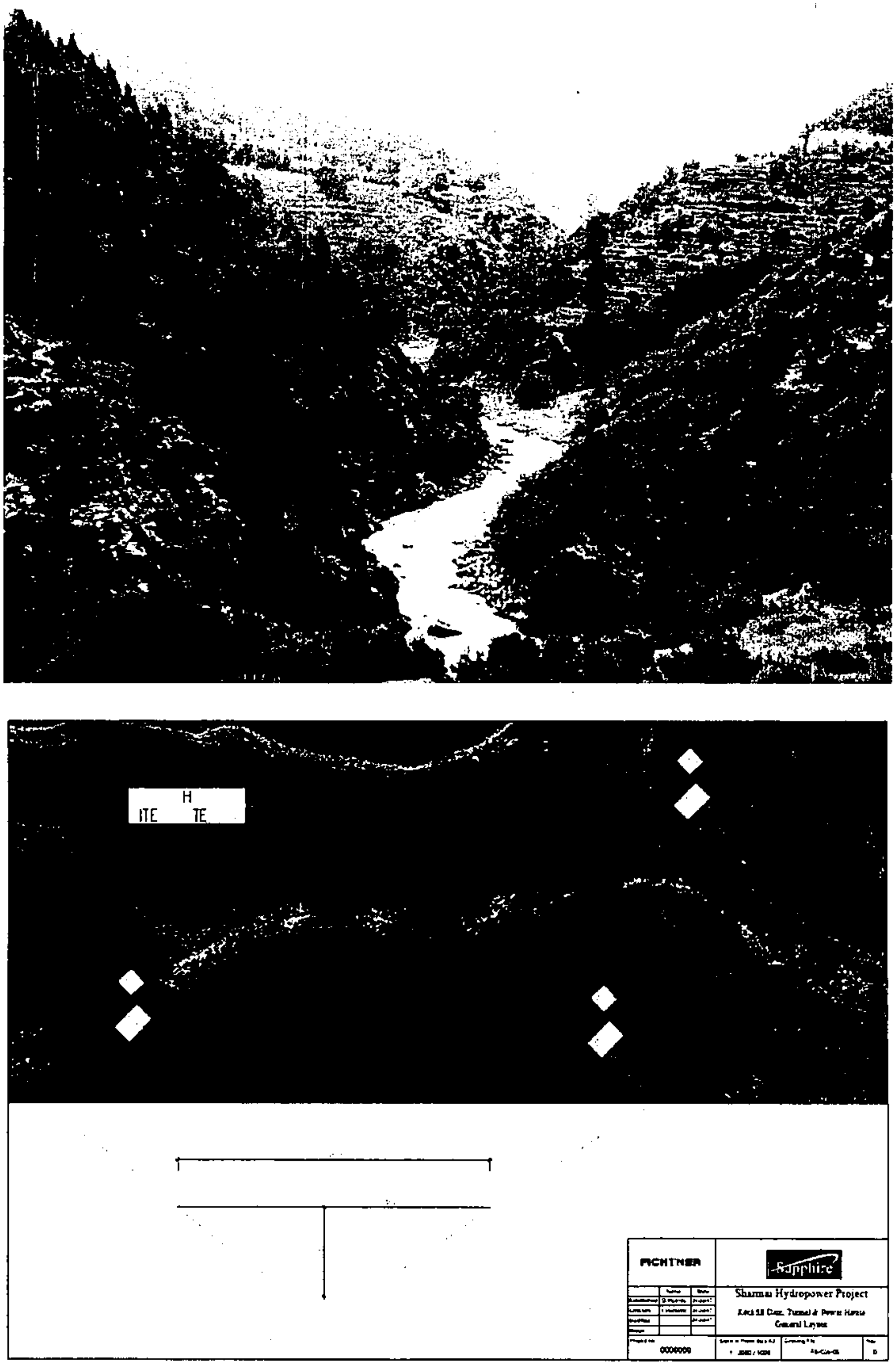


Figure 6-14: Sinohydro dam site 2 / GTZ dam site

This dam site offers suitable conditions in terms of river diversion, spillway and energy dissipation structures and the position of the intake to the power tunnel.

Dam formed at this profile, with the reservoir NOL at 1290, as suggested in the report referred to, would need to be about 100 m high. The accurate topographic and bathymetric measurements carried out at this site show the terrain elevation at about 1200 masl.

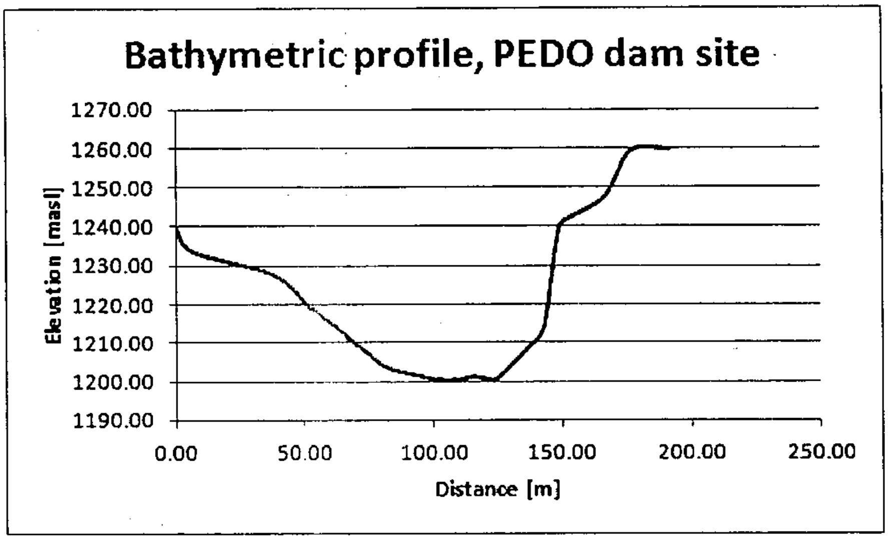


Figure 6-15: Bathymetric profile, PEDO dam site

Alternatively, the reservoir NOL at 1260 would imply the need for a dam approximately 70m high.

6.4.1.4 New dam site - FI 2

This site is located about 1.55 km upstream of the site above. The site is deemed suitable for a daily storage with an approximately 45 m high dam.

Figure 6-16: FI 2 dam site (1)

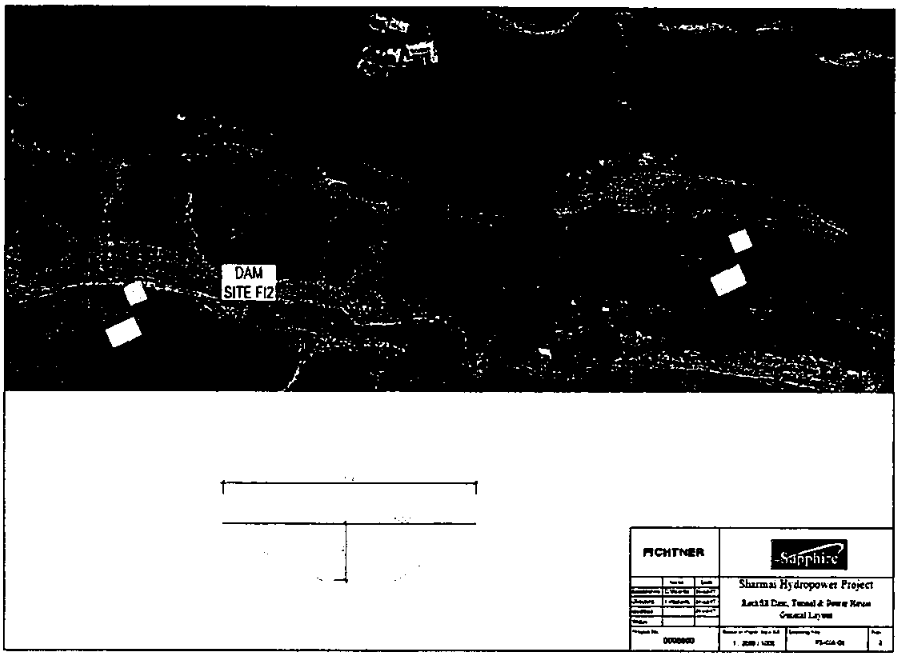


Figure 6-17: FI 2 dam site (2)

This site, however, implies somewhat longer power tunnel and the related friction caused head losses.

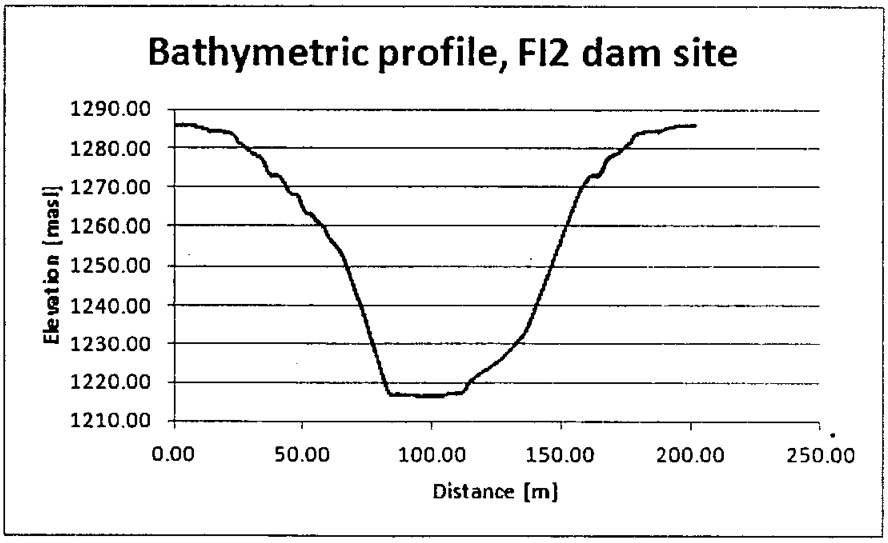


Figure 6-18: Bathymetric profile, FI2 dam site

Dam formed at this profile, with the reservoir NOL at 1260, would be about 45 to 50 m high. The accurate topographic and bathymetric measurements carried out at this site show the terrain elevation at about 1220 masl.

6.4.1.5 New dam site - FI 1

This site is located at the profile of a suspension bridge, some 1.25 km upstream from the site FI 2. This site is suitable for a run-off river type plant, with the possibility to construct a 15m high weir.

Figure 6-19: Suspension bridge at FI 1 dam site

Furthermore, the location of this site would imply the power tunnel length of more than 10 km.

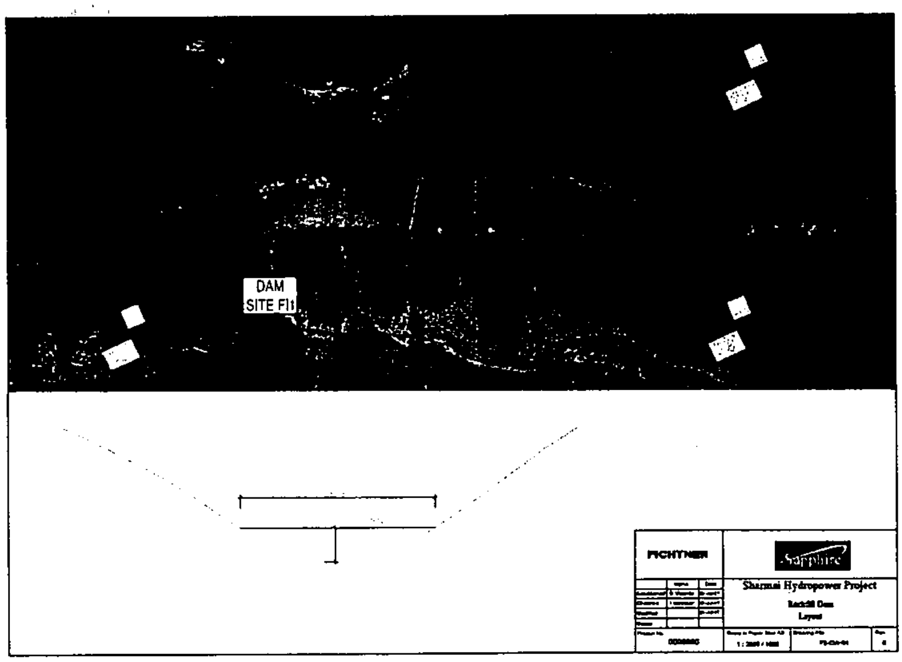


Figure 6-20: FI 1 dam site

This dam site brings concerns in terms of reservoir sustainability. Being exposed to heavy sediment intrusion, frequent outages in operation and additional O&M costs due to the intensive sediment management are deemed likely for this dam site. According to the results of the performed reservoir volume analysis, FI 1 dam site could offer the possibility to be operated as a daily peaking plant only with the reservoir water level of 1270 masl and more. This elevation is deemed to raise significant ESIA concerns, as elaborated in the following chapter. Therefore, this dam site is abandoned from further analyses.

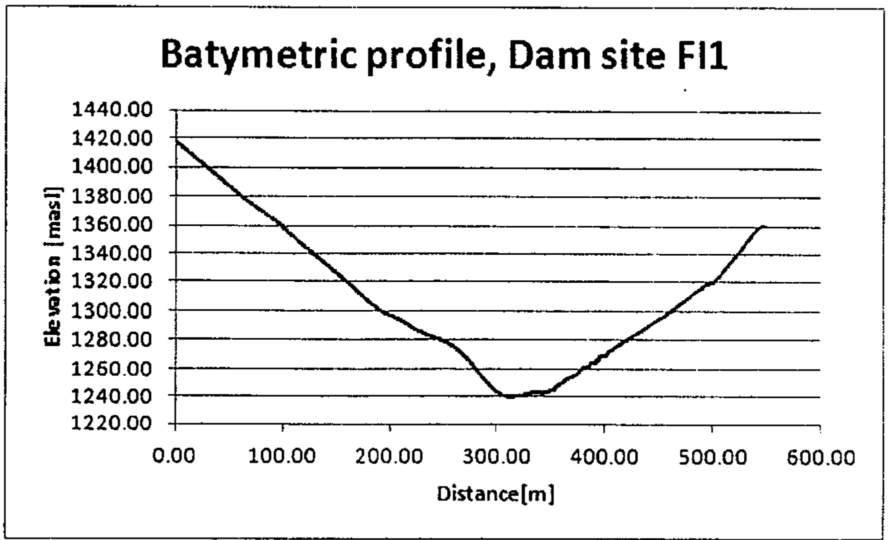


Figure 6-21: Bathymetric profile, FI1 dam site

Dam formed at this profile, with the reservoir NOL at 1260, would be about 25 m high. The accurate topographic and bathymetric measurements carried out at this site show the terrain elevation at about 1240 masl.

6.4.1.6 Conclusion on the dam sites to be further studied

There are 2 main alternatives for the dam site location:

- PEDO Site, the dam site proposed in the Site Investigation Report (Sinohydro, 2017) as the upstream dam site (Dam site 2) and in the GTZ Report (1992); in the following text this site will be referred as D/S (downstream) site, and
- FI 2 site, located 1.55 km upstream; in the following text this site will be referred as U/S (upstream) site.

Both dam sites provide conditions suitable for a plant with a daily reservoir. The key advantage of the downstream site is somewhat shorter power tunnel and the larger size of the active storage. The key advantage of the upstream site is approximately 20 m lower dam. The benefit/cost comparison in terms of project layout and the position of the individual structures is provided in the subsequent sections.

The other inspected sites are not deemed competitive. The two most promising dam sites will be further analyzed and compared within the optimization analyses as reported in the following chapters.

6.4.2 Reservoir water level

As concluded during the site visit, the area close to the reservoir root, presently used as arable land and source of incomes for the local population, would be inundated by the water level (WL) in the reservoir above approximately 1260 masl. The option to raise the reservoir WL above this

elevation, although technically doable, is expected to born severe ESIA-related impacts due to the project.
It is expected that the inundation of the areas above this elevation would create resistance at the local inhabitants and possibly affect the implementation of the project.

Figure 6-22: Arable land; photo taken from the elevation 1275 masl

According to the findings of the initial site visit, the main source of income for the local residents is agriculture, the cultivation of fruit trees, and stock-breeding. Agriculture is very difficult in an environment which is characterized by deep indented valleys with steep hillsides. Therefore terraces have been built at the steep hillsides over many centuries, which now allow the local population to grow staple food.

Wide parts of the steep hillsides, which are not used for agriculture, are covered by trees, which are used by the local people as firewood or as fruit trees (mainly walnut).

Inundation of productive land within the reservoir would lead to a permanent loss of livelihood for the local population and therefore opposition of the local inhabitants against the planned project is expected to be directly connected to the loss of productive land.

In order to conclude on the reservoir WLs, the various possible alternatives were quantified. The water levels between 1,160 and 1,300 masl have been investigated for the two dam site locations "U/S" and „D/S".

For these two dam sites an initial comparison between the various reservoir water levels has been conducted, based on a GIS evaluation of high resolution satellite imagery, and considering the main means of livelihood, agricultural terraces and areas covered by trees.

The following two figures shows the locations of the dam site options and the different water levels investigated.

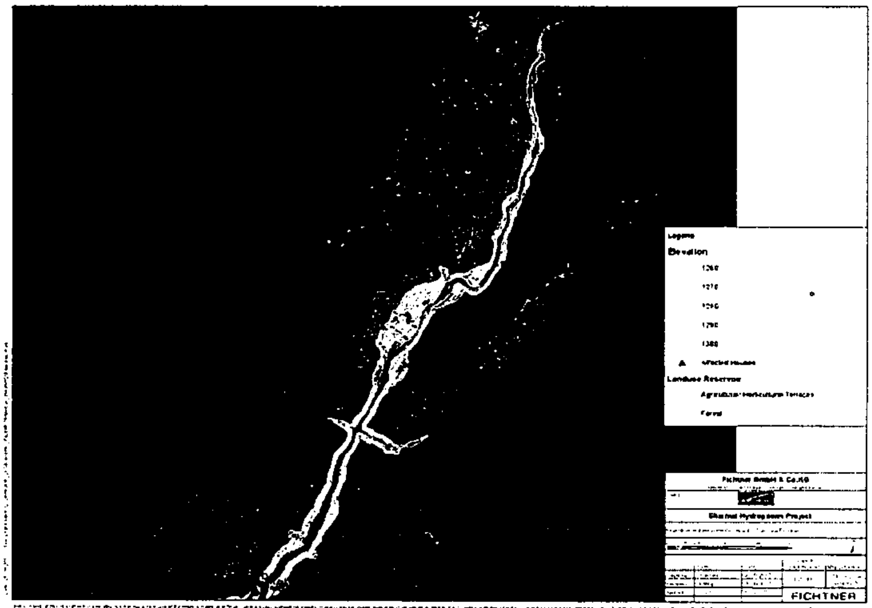


Figure 6-23: Locations of the dam site option "U/S" investigated and the resulting reservoir areas at reservoir levels between 1,260 masl and 1,300 masl

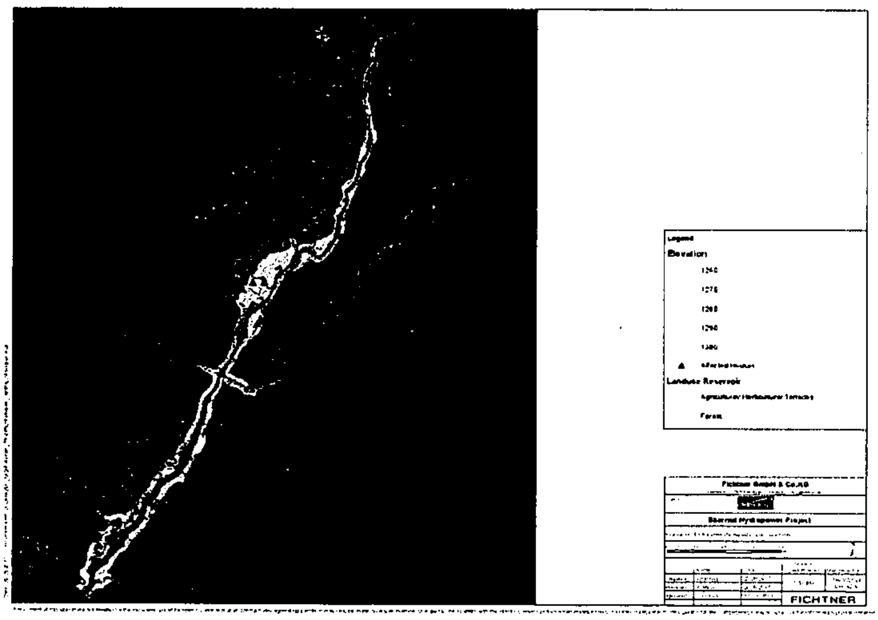


Figure 6-24: Locations of the dam site option "D/S" investigated and the resulting reservoir areas at reservoir levels between 1,260 masl and 1,300 masl.

The following table shows the results of the GIS evaluations of the 2 dam with various full supply water levels.

Table 6-6: **Results from GIS evaluations of high resolution satellite scenes regarding lass of agricultural land and tree areas within the various reservoir options**

Dam site	FSL Reservoir	Loss of houses/compounds (number)	Loss of agricultural terraces (in ha)	Loss of forest / trees (in ha)
Dam site U/S	1.260 masl	0	1,61	3
	1.270 masl	0	9,38	5,47
	1.280 masl	3	20,11	8,31
	1.290 masl	6	35,43	11,87
	1.300 masl	15	48,72	16,9
Dam site D/S	1.260 masl	0	4,49	8,34
	1.270 masl	3	12,88	13,28
	1.280 masl	6	24,40	17,97
	1.290 masl	12	41,51	23,01
	1.300 masl	21	58,95	29,6

Only the Dam site "U/S" at 1,260 masl and 1,270 masl, and dam site "PEDO" do not require physical destruction and relocation of residential houses. These solutions are therefore regarded as the most favourable or preferable options from an environmental and social point of view.

The dam site option "U/S" at 1,280 m would cause a loss of 3 houses/compounds and a high loss of agricultural terraces (ca. 20 ha). Three houses and ca. 13 ha agricultural terraces would also be lost with dam option "D/S" at 1.270 masl and 6 houses and ca. 24 ha agricultural terraces at 1,280 masl. These options are regarded as "medium" or "tolerable" from an environmental and social point of view. However there is still a medium risk that affected property owners may cause problems.

All other options would cause a loss of numerous houses and a significant loss of agricultural areas. These options are therefore regarded as "worst cases" from an environmental and social point of view. In these cases there is a high risk of public opposition against the planned project.

In terms of the impact of the project to the existing infrastructure, the elevation of the deck of the steel bridge in vicinity of the reservoir root, capable to withstand heavy duty traffic is at 1265 masl. It is considered as a minimum to allow 3 m clearance freeboard between the reservoir water / flood level and the bottom of the bridge's deck, in order to ensure passage of floating debris during flood events.

Figure 6-25: Existing bridge at the reservoir's root

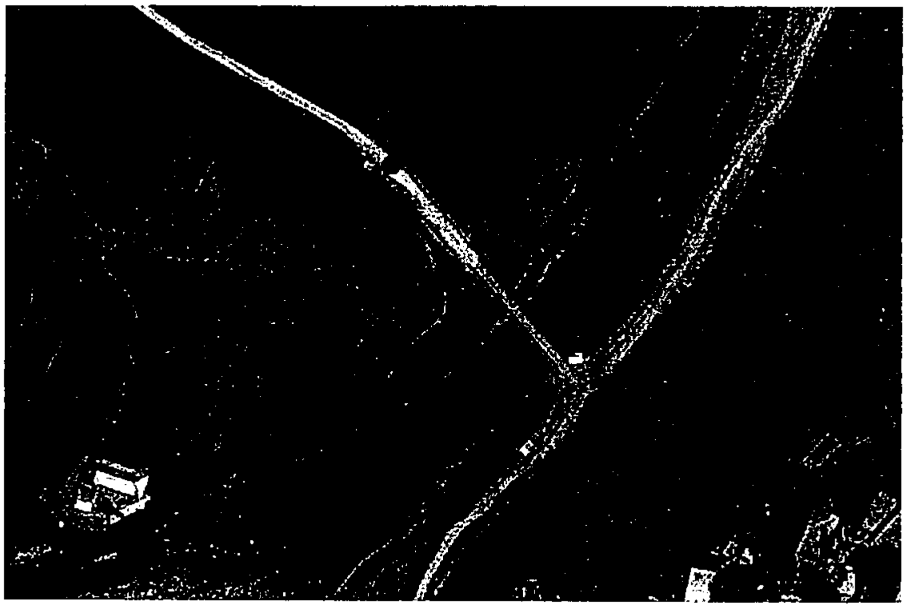


Figure 6-26: Elevation of the steel bridge at the reservoir's root

Therefore, the maximum reservoir level was limited to 1,260 masl, deemed acceptable both from ESIA and infrastructure aspects.

It has to be noted that a higher reservoir WL (e.g. of 1290 masl, as proposed in the previous studies) would cause the inundation of up to 5 km of the regional road to Citral and Dir valleys and require reconstruction of another, newly constructed bridge, which again could lead to resistance at the local population.

Figure 6-27: New bridge under construction (affected by reservoir WL 1290 masl)

According to the preliminary analysis, based on the water availability as presented in the hydrological report and the statement on the environmental minimum release, as per the Phase 1 ESIA report, prepared by Hagler Bailly, the reservoir's active volume of about 650'000 m³ would allow daily peak operation at Sharmai powerplant.

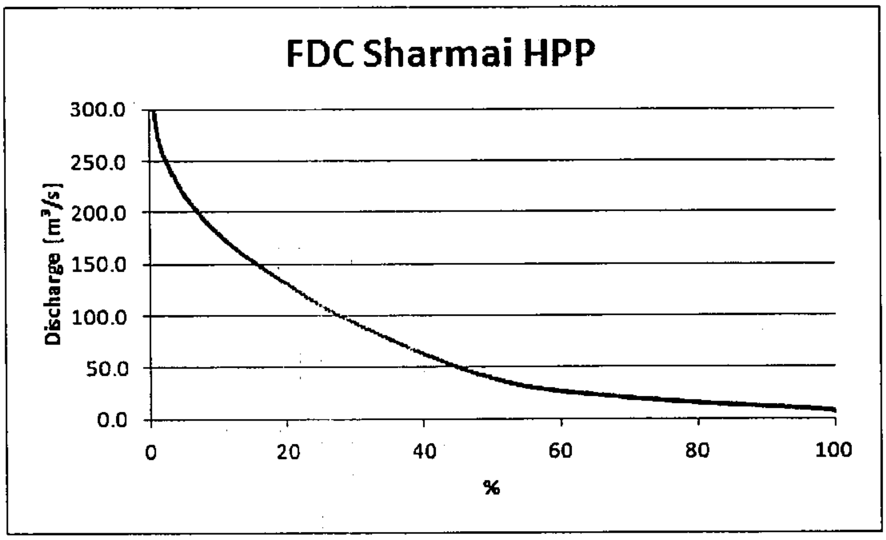


Figure 6-28: Sharmai HPP - FDC

The following figure shows the reservoir volume curves at the two promising sites, as discussed in the previous chapter.

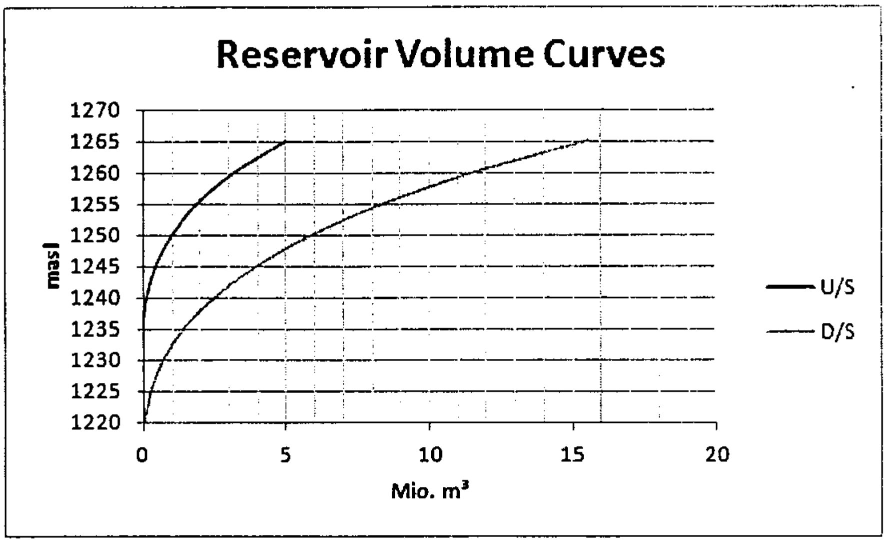


Figure 6-29: Sharmai HPP - reservoir volume curves for the preselected dam sites

6.4.3 Dam type selection

Considerations on the dam sites and the reservoir levels imply the dam heights of 45 m (U/S site) and 65 m (D/S site), roughly. The reservoir size / volume is relatively modest, due to the terrain morphology. In order to cope with the heavy sediment load, as per the results of the sediment analysis, flushing of the reservoir seems as one of few options which could guarantee sustainability of the reservoir active storage over its envisioned lifetime. As concluded in the Report on Hydrology and Sediments, the design of the Sharmai headworks would need to accommodate for a sediment tank as well.

According to the results of the hydrological analysis, the order of magnitude of flows during exceptional / design flood conditions is several thousands m^3/s, which implies the need for a spillway structure of considerable size. Assuming a high unit discharge of 120 $m^3/s/m$ (ref. chapter 2), the required width of the spillway structure could be more than 70m, including piers. This width corresponds to more than 50% of the dam crest lengths at the considered dam sites.

The prevailing site conditions imply the necessity for a reservoir flushing device / bottom outlet, required to remove the deposits accumulated below the spillway crest elevation. As such, the sedimentation of the reservoir would be prevented by sluicing over the spillway gates and by flushing through the bottom outlet. The following figure shows the size and layout of the openings deemed necessary at the dam.

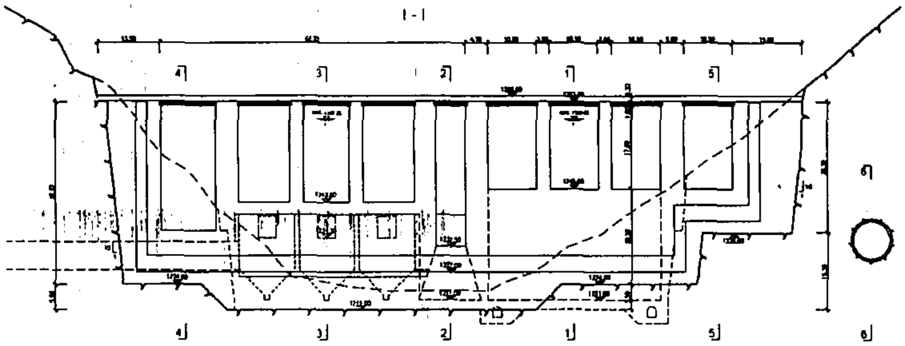


Figure 6-30: Required openings at Sharmai dam

Such site conditions and the particulars related to the design qualify a concrete gravity dam structure, equipped with openings for sediment tank, reservoir flushing and release of high flows as a promising dam type option. A concrete dam offers the possibility for a two-stage river diversion during construction, which could further reduce the efforts and costs related to dam and is deemed less sensitive to overtopping.

An RCC dam is deemed less attractive, due to the relatively low total volume of dam material and the configuration of the dam, which assumes numerous openings and thus, application of conventional, structural concrete.

Although that a rockfill dam option may be more suitable in terms of foundation conditions and the locally available construction material, the necessity for an integrated facility for reservoir flushing, the uncertainty in terms of hydrological/flood data and the inevitable existence of the large spillway and sediment tank blocks disqualify the rockfill dam type as a competitive option, as the said concrete blocks would need to be of the same size as for the concrete dam option.

6.4.4 Powerhouse site selection

The two powerhouse sites proposed in the available documents were generally confirmed during the site visit, with certain adjustments to reflect the conditions as encountered on site. There are two powerhouse sites:

- Downstream PH site, close to Darora and
- Upstream PH site, close to Bibyawor.

6.4.4.1 Downstream PH site

Powerhouse site 1, as referred to in the Site Investigation Report, Sinohydro, 2017 is shifted to Panjkora River (in some of the available project documents it was positioned at the left tributary, which has a much steeper gradient). This powerhouse site is located in a densely populated area.

Surface powerhouse concept is deemed to cause considerable social impacts, due to extensive excavation required to place the powerhouse building as a surface structure. Same applies for the alignment of the penstocks, envisioned as surface structures as well.

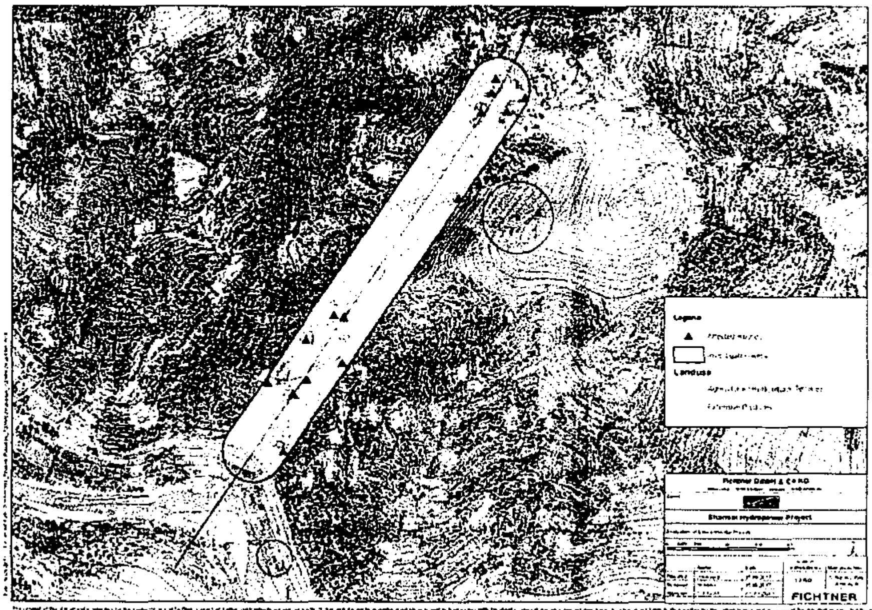


Figure 6-31: Area affected by surge tank, surface penstock and surface power house

Besides the obvious disadvantages in terms of ESIA, a surface powerhouse concept would negatively affect the existing regional road, which would need to be locally relocated.

Instead, the option with a cavern powerhouse with the outlet of the tailrace tunnel placed at the visible rock formations located somewhat upstream from the confluence is seen as a promising option.

Figure 6-32: Downstream located powerhouse site/site of the outlet from tailrace tunnel

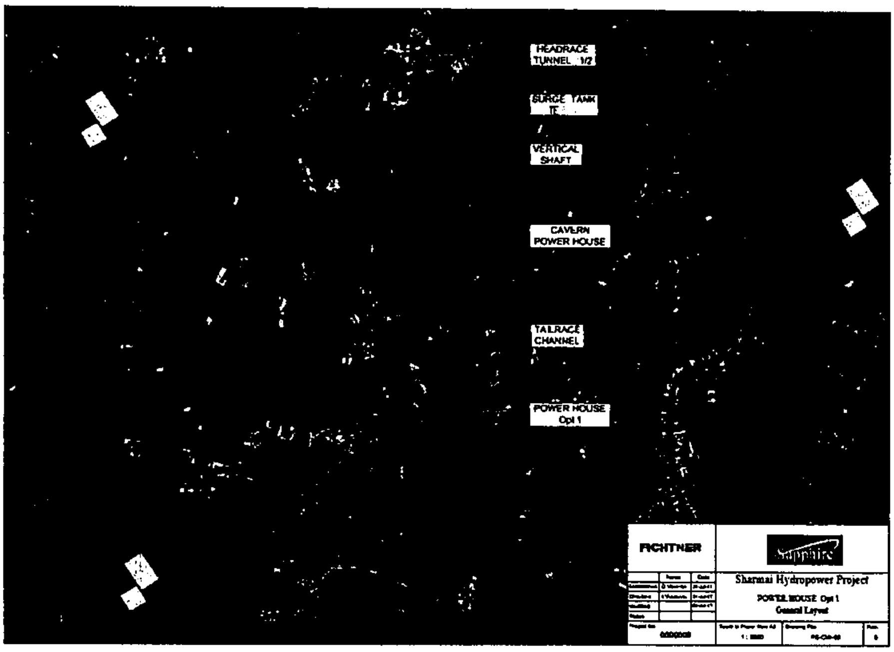


Figure 6-33: Downstream located powerhouse site - conceptual layout

6.4.4.2 Upstream PH site

Powerhouse site 2, as referred to in the Site Investigation Report, Sinohydro, 2017 is located close to Bibyawor at a narrow section of Panjkora River, upstream of the confluence with the right tributary. A surface powerhouse would imply considerable excavation works, due to the relatively steep terrain. The possible layout with the surface penstock would further affect the N-45 road and imply the need for its partial relocation.

Figure 6-34: Upstream located powerhouse site, view from the road above

Figure 6-35: Upstream located powerhouse site, view to downstream

Again, a cavern type powerhouse is regarded as a promising alternative to the surface option concept.

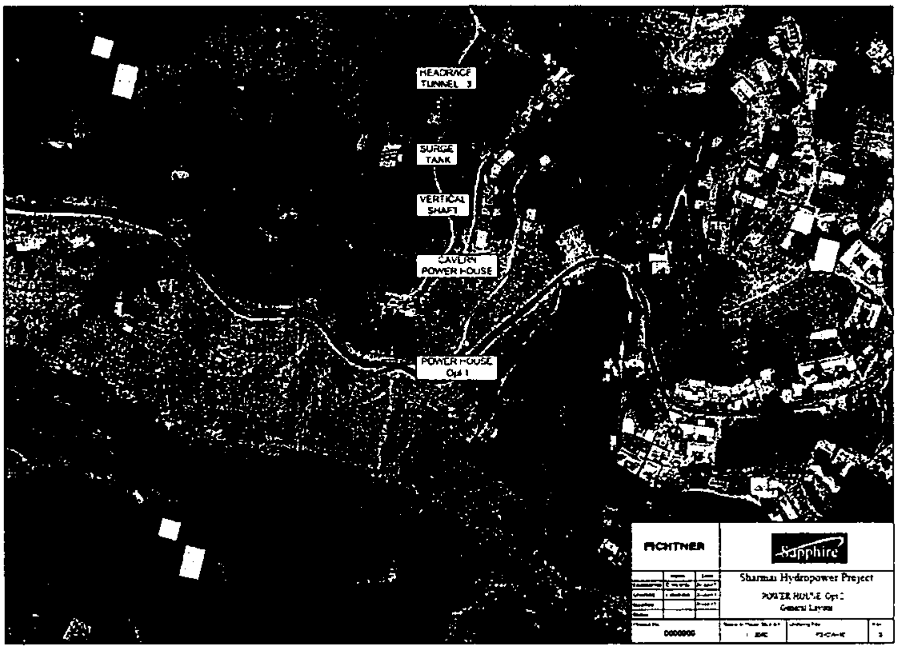


Figure 6-36: Upstream located powerhouse site - conceptual layout

The difference in head between the two powerhouse sites is roughly 50 m (i.e. about 30% of the available head / installed capacity). The key advantage of the upstream located powerhouse is somewhat shorter power tunnel. These facts disqualify the upstream powerhouse location as a not-competitive for further development.

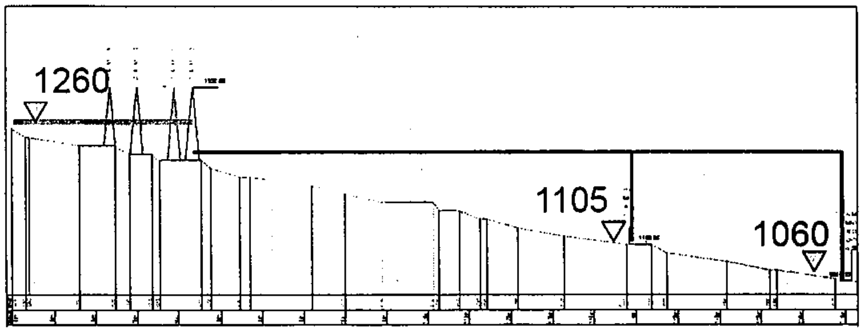


Figure 6-37: Section through the riverbed - conceptual layout

The downstream powerhouse location utilizes the maximum hydropower potential of Panjkora River over the considered stretch, which is deemed optimum under the given energy and peak power supply conditions in Pakistan.

The option to develop the Sharmai HPP as a concept of two cascade projects, as discussed in the Site Investigation Report, Sinohydro, 2017 is not found suitable.

6.4.5 Layout of power waterways and Powerhouse arrangement

6.4.5.1 Route of the headrace tunnel

After the dam and powerhouse sites were defined, the layout of the waterways was adjusted accordingly. Depending on the particular combination of the individual components, the length and the layout of the power tunnel changes: U/S dam site requires a tunnel almost 1km longer than if the dam would be placed at D/S site.

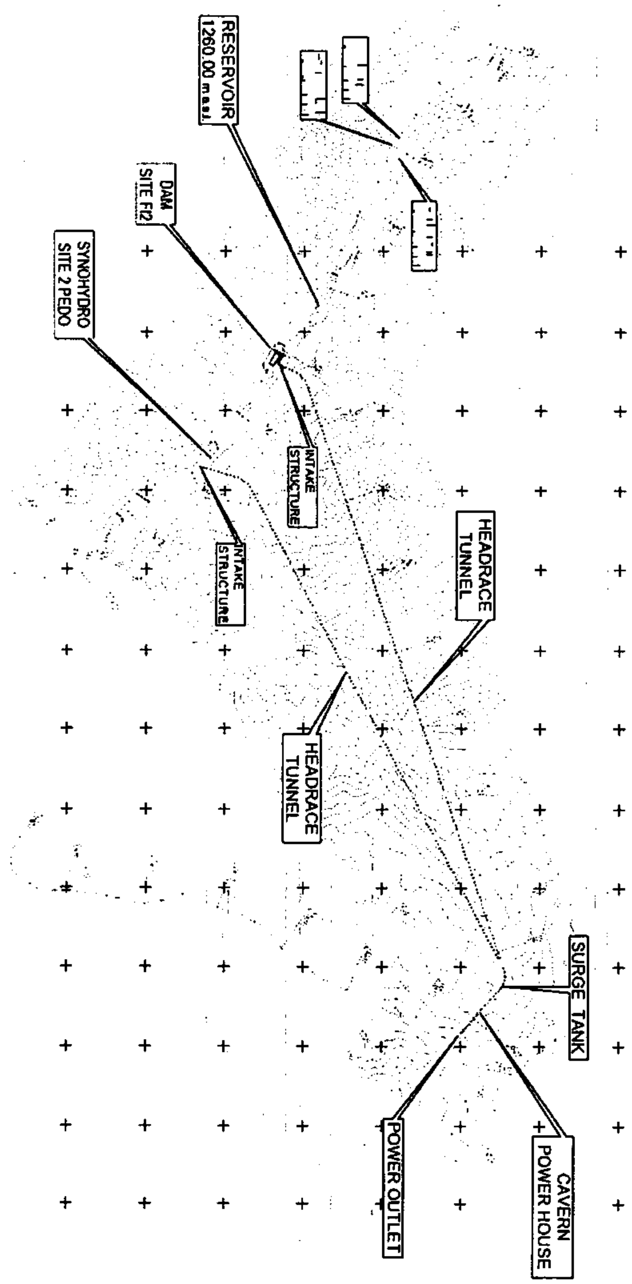


Figure 6-38: Alternative routes of the headrace tunnel

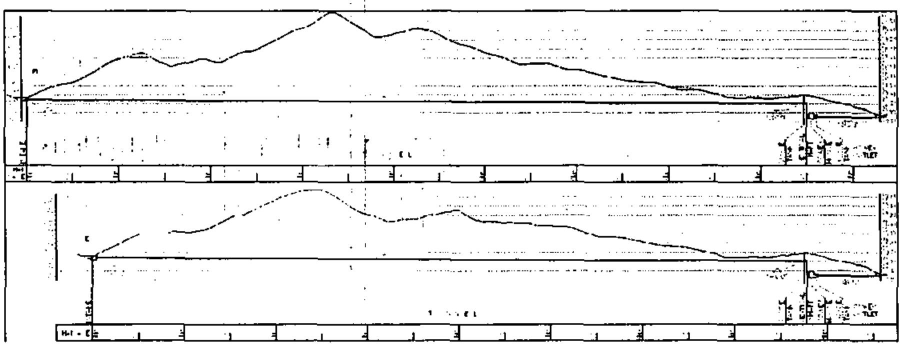


Figure 6-39: Sections through the alternative headrace tunnel routes

6.4.5.2 Headrace tunnel diameter

In course of the elaboration of the Feasibility Study, least-cost optimization analysis focusing on the diameter of the power tunnel was performed. It considered the direct, construction costs and the losses in energy generation over a time horizon of 30 years, assuming different design discharges, ranging from 60 to 100 m³/s.

The following figure shows the summary of the results obtained through the analysis.

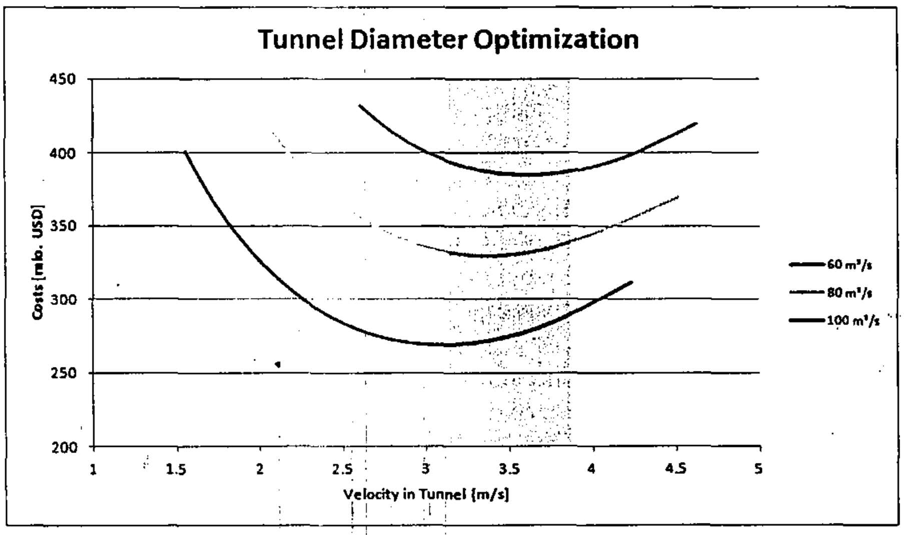


Figure 6-40: Tunnel diameter optimization

As per the results of the analysis, the optimum tunnel design is achieved with the flow velocities ranging from 3.25 to 3.75 m/s. For the subsequent optimization analyses focusing on selection of the installed discharge, the design flow velocity in the power tunnel of 3.5 m/s was adopted.

6.4.5.3 Surge tank

Another criterion for the layout of the power tunnels is the necessity for and the position of the surge tank. Although relatively obvious, the necessity for the surge tank is confirmed on account of the regulation capability of the plant.

One method to verify the regulation capability of a hydropower scheme is the comparison between the expected machine starting time T_m and the acceleration time of the closed water column (between free water surfaces) in the waterways T_w. This method is applied for the Sharmai HPP.

The machine starting time T_m is a characteristic of the generating units, used in the hydropower plant. The water acceleration time T_w defines the time for accelerating the water column in the waterways from standstill until steady state conditions and therefore depends on the waterway layout.

An indication for an acceptable regulation capability is a ratio between the machine starting time T_m and the water acceleration time T_w of minimum 3.5:

$$\frac{T_m}{T_w} \geq 3.5$$

The machine starting time T_m can be estimated based on the output of the generating unit, the rotational speed and the moment of inertia of the rotating mass (which is mainly determined by the generator). For determination of these input values, required to estimate the machine starting time, a preliminary layout of the generating equipment for the given conditions and the plant discharges is necessary. For definition of the overall power plant scheme a plant discharge between 60 m³/s and 100 m³/s is considered. The results and the related machine starting time depending on the plant discharge are shown in the table hereunder. For all considered plant discharges, a two-unit-arrangement is considered at this stage.

Applying the minimum ratio between the machine starting time and the water acceleration time of 3.5, with the received machine starting times, the related maximum water acceleration times can be calculated as:

$$T_w \leq \frac{T_m}{3.5}$$

Based on this criterion and the preliminary machine starting times, the maximum permissible water acceleration times to ensure sufficient regulation capability are derived as given below. These values shall be used to define and verify the design of the waterway layout of Sharmai HPP.

Table 6-7: Machine starting time Tm, Maximum permissible water starting time Tw,max

Plant Discharge	Q	m³/s	60	80	100
Unit #	-	-	2	2	2
Approx. Unit Capacity	Pu	MW	51	67	84
Prelim. Synchr. Speed	n	rpm	428.6	375.0	333.3
Machine Starting Time	**Tm**	**s**	**6.0**	**6.3**	**6.6**
Maximum Water Starting Time	**Tw,max**	**s**	**1.7**	**1.8**	**1.9**

The water acceleration time is further defined as:

$$T_w = \frac{\sum(L_i * v_i)}{(H * g)}$$

L_i Length of waterway section i [m]
v_i Velocity in waterway section i [m/s]
H Available head [m]
g Gravitational constant [m/s²]

The actual water acceleration time $T_{w,act}$ for the headrace tunnel options (7500 and 8500 m long) amount to about 14 to 16 s.

Table 6-8: Actual water starting time Tw,act

Section	Length L [m]	Velocity v [m/s]	Head H [m]	L x v / g x H [s]
Headrace Tunnel	7,500	3.5	185	14.5
Headrace Tunnel	8,500	3.5	185	16.4

Although based on the preliminary values for the velocities in the power waterways, it is evident that an upstream surge tank is required.

The site for the surge tank for the option with the downstream located powerhouse is relatively challenging, due to the unfavorable terrain morphology.

For the option with the downstream located powerhouse, the surge tank site identified in the previous project studies seems as the only feasible location. The surge tank shall be located at the existing ridge, as close as possible to the riverbed.

Figure 6-41: Photo from the surge tank site

The position of the surge tank implies relatively long waterways to reach Panjkora River. Horizontal distance to the tailrace outlet is about 800 m.

Due to the terrain morphology, the power tunnel in the option with the downstream located powerhouse will have a curvature on its downstream end. The location of the surge tank was selected with respect to the hydraulic transient calculations and fine tuned with respect to the existing infrastructure and settlements, as illustrated in the figure below.

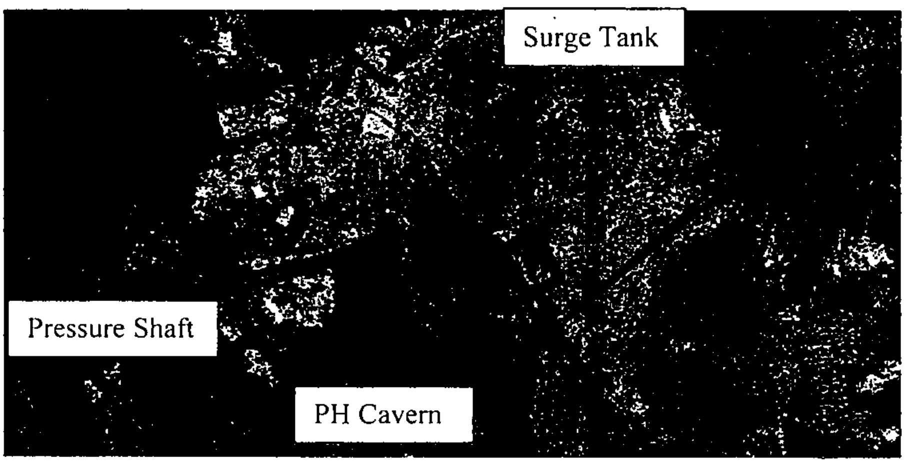


Figure 6-42: Power waterways - location of the surge tank

6.4.5.4 Alignment of pressure waterways

As concluded in chapter 6.4.4, layout of the pressure waterways is predominantly governed by the ESIA impact the different possible configurations cause. The settlement structures in the project affected area are generally farm compounds, which are usually inhabited by one family. Beside the residential house these compounds also comprise other buildings like storage houses and stables. These compounds are surrounded by agricultural terraces.

The options with surface and underground pressure waterways are compared. The results based on an evaluation of satellite maps (subject to confirmation in the further ESIA process) are presented in the following table.

Table 6-9: Pressure waterways: resettlement and loss of arable land

Option	Loss of compounds/ houses (number)	Loss of agricultural & horticultural terraces (in ha)
1. Underground penstock	0	1 ha
2. Surface penstock	8	7,8 ha

For a surface penstock solution a 100 m wide and ca 800 m long open construction strip between surge tank and power house over the whole hillside would be required. Ca. 7,8 ha of agricultural & horticultural terraces would be lost. In this area all structures like houses, agricultural terraces, roads, irrigation channels, etc. would need to be removed.

Considering the population growth and the scarce availability of farm land in the area the loss of farms and farm land, together with the interruption of irrigation channels and the road and path network may not only cause problems regarding compensation issues but may also constitute a strong reason for the local population to oppose the planned project.

Therefore, the option with underground pressure waterways is selected for further design optimization analyses.

6.4.5.5 Powerhouse concept: Surface vs. underground (Cavern)

Two concepts for the powerhouse general arrangement were considered: surface and underground/cavern. The selection of the concept to be further developed is based on technical and financial analyses, as described in the following sections.

Additionally to the ESIA-related considerations on the powerhouse arrangement, as discussed in Chapter 6.4.4, an analysis of the hydraulic transients was performed for both powerhouse concepts: surface and underground, with the target to investigate the technical feasibility for the different options.

Preliminary transient calculations showed that an arrangement with a cavern powerhouse, will either require a tailrace surge tank downstream of the powerhouse cavern or a free flow tailrace tunnel to avoid negative pressures in the turbine draft tubes and tailrace tunnel during sudden load rejections or rapid load changes.

Therefore, the following arrangements were evaluated with regards to the regulation capability:

1. Alternative 1: Surface powerhouse, located at the tailwater at the downstream end of a high-pressure tunnel of approx. 980 m length
2. Alternative 2: Cavern powerhouse with free flow tunnel (cavern powerhouse at the end of the pressure shaft)
3. Alternative 3: Cavern powerhouse with tailrace surge tank (cavern powerhouse at the foot of the pressure shaft), the surge tank is located downstream of the powerhouse cavern.

Table 6-10: Machine Starting Time Tm, actual Water Acceleration Time Tw,act depending on the plant discharge

Plant Discharge	Q	m^3/s	60	80	100
No. Generating Units	-	-	2	2	2
Approx. Unit Capacity	Pu	MW	51	67	84
Prelim. Synchr. Speed	n	rpm	428.6	375.0	333.3
Machine Starting Time	Tm	s	6.0	6.3	6.6
Alternative 1: Surface Powerhouse					
Actual Water Acceleration Time	Tw,act	s	3.0	3.0	3.0
Ratio Tm/Tw	Tm/Tw,act	-	**2.0**	**2.1**	**2.2**
Acceptable Regulation Capability	**Tm/Tw>3.5**	**-**	[redacted]	[redacted]	[redacted]
Alternative 2: Cavern Powerhouse with Free Flow Tunnel					
Actual Water Acceleration Time	Tw,act	s	0.6	0.6	0.6
Ratio Tm/Tw	Tm/Tw,act	-	**10.0**	**10.5**	**11.0**
Acceptable Regulation Capability	**Tm/Tw>3.5**	**-**	**Yes**	**Yes**	**Yes**
Alternative 3: Cavern Powerhouse with Tailrace Surge Tank					
Actual Water Acceleration Time	Tw,act	s	0.6	0.6	0.6
Ratio Tm/Tw	Tm/Tw,act	-	**10.0**	**10.5**	**11.0**
Acceptable Regulation Capability	**Tm/Tw>3.5**	**-**	**Yes**	**Yes**	**Yes**

The comparison between the machine starting times and the water acceleration times shows, that only an arrangement with a cavern powerhouse (with free flow tunnel or tailrace surge tank) will ensure sufficient regulation capability of the power plant. The long waterways for a layout with a surface powerhouse will not allow for proper regulation of the plant and generating units without particular additional measures (such as increase of the rotating mass/inertia at the units, increase in size of the pressure tunnel and/or application of pressure-release valves at the units), which could possibly make the surface powerhouse concept technically feasible.

According to the results of the analyses performed in this respect, the inertia of the rotating shaft would need to be increased by approx. 40%-50% compared to the standard values, resulting in a GD^2 of around $1700tm^2$ to $1800tm^2$. This can be achieved by a larger rotor and thus masses, resulting in a larger diameter to height ratio for the generator.
The cost-implication of the increase in the rotating mass is estimated with 25% increase as compared with the standard equipment/generator configuration. The quotation provided by one of the best world-wide equipment suppliers confirms this statement.

The possible additional measures would turn the solution with the open air powerhouse into a custom, non-standard design and add further costs to this option. Following the approach described in detail in Chapter 6, a preliminary cost comparison is performed for the variety of the installed discharges, assuming open air and cavern powerhouse concepts. The following table summarizes the direct costs related to the two alternatives.

Table 6-11: Preliminary cost comparison: open-air vs. cavern type powerhouse (Francis configuration)

Sharmai HPP U/S Site - Optimization and Ranking / Preliminary Cost Estimate (Open Air)												
Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Design discharge	[m³/s]	55	60	65	70	75	80	85	90	95	100	105
Installed power	[MW]	89.2	98.0	106.3	114.7	124.0	133.3	141.9	150.7	158.9	167.1	175.4
Installed power (at busbars)	[MW]	86.5	95.1	103.1	111.3	120.3	129.3	137.7	146.2	154.2	162.1	170.1
Pressure shaft	[mUSD]	9.5	10.1	10.7	11.4	12.1	12.8	13.3	14.0	14.5	15.0	15.5
Powerhouse - Civil Works	[mUSD]	2.8	3.4	3.5	3.6	4.3	4.4	4.5	4.7	5.4	5.5	5.6
Powerhouse - HSS	[mUSD]	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.6
Powerhouse - HM	[mUSD]	12.7	14.1	14.8	15.0	15.1	15.4	16.6	16.7	18.6	19.1	19.3
*Powerhouse - EM**	[mUSD]	15.5	16.6	17.3	18.1	19.2	19.9	20.6	21.3	22.4	23.0	23.6
Tailrace channel	[mUSD]	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
ESIA costs, road relocation	[mUSD]	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Sub Total Direct Costs	[mUSD]	42.1	45.8	48.0	49.7	52.4	54.3	56.8	58.5	62.7	64.5	66.0
** 25% cost increase due to not-conventional design*												
Sharmai HPP U/S Site - Optimization and Ranking / Preliminary Cost Estimate (Cavern)												
Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Pressure shaft	[mUSD]	2.4	2.6	2.8	2.9	3.1	3.3	3.4	3.6	3.7	3.9	4.0
Powerhouse - Civil Works	[mUSD]	8.9	9.9	10.0	10.4	11.6	11.8	12.2	12.3	13.7	13.8	14.4
Powerhouse - HSS	[mUSD]	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.6
Powerhouse - HM	[mUSD]	12.7	14.1	14.8	15.0	15.1	15.4	16.6	16.7	18.6	19.1	19.3
Powerhouse - EM	[mUSD]	12.4	13.3	13.9	14.4	15.3	16.0	16.5	17.0	17.9	18.4	18.9
Tailrace channel	[mUSD]	4.7	5.0	5.3	5.8	6.2	6.6	7.0	7.4	7.6	7.9	8.1
Downstream Surge Tank	[mUSD]	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6
Sub Total Direct Costs	[mUSD]	41.9	45.8	47.7	49.6	52.3	54.2	56.8	58.2	62.7	64.2	65.9

It can be concluded that the costs for the two options are similar, though slightly in favor of the cavern-type powerhouse type option.

The following figures illustrate the impact an open-air powerhouse would cause to the existing terraces, road and infrastructure. It is to be expected that an open-air powerhouse option may trigger resistance with the local population.

Figure 6-43: Position of an open-air powerhouse and the existing road infrastructure

Figure 6-44: Arable land at the open-air powerhouse location

The two alternatives are compared using the SWOT analysis approach:

Accessibility
Independent construction sequence

Complex construction pit prone to flooding
No standard technology
No cost advantage compared to Cavern
Low cost certainty for E&M
Limited options for Switchyard location

Open-air

Higher cost certainty for civil works

Technical challenges in design and operation
Limited number of suppliers
Higher costs for equipment (25+%)
Impact on traffic for Dir and Citral valleys
Resettlement required
Higher risk of interruption during construction due to local population
Risk of flooding during construction

Figure 6-45: SWOT analysis: Open-air powerhouse

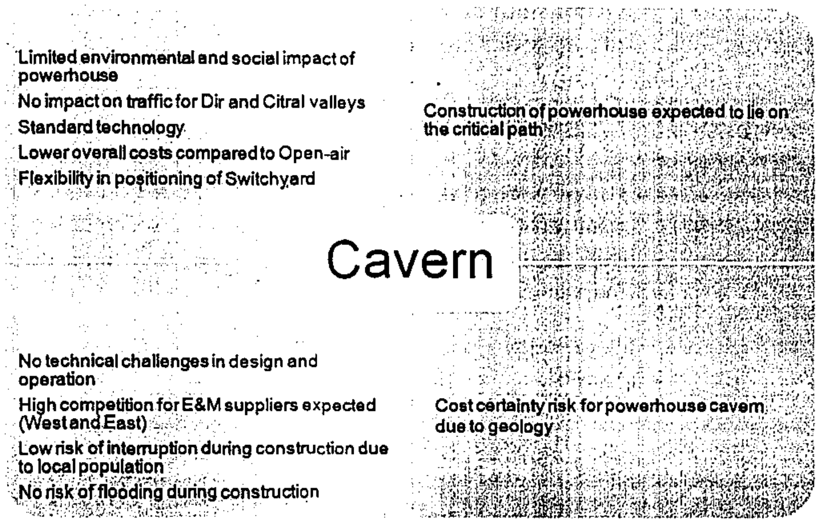


Figure 6-46: SWOT analysis: Cavern powerhouse

Due to the advantages in terms of operation of the units and due to the obvious ESIA- and infrastructure-related downsides the option with the open-air powerhouse option born, the option with the cavern-type powerhouse is adopted to be further developed. The open-air powerhouse option cannot be regarded advantageous from the cost-perspective either.

6.4.5.6 Position of the powerhouse cavern

In order to conclude on the most promising position for the PH cavern, a preliminary analysis was performed, giving focus to the investment costs of the pressure and tailrace tunnels and the hydraulic losses in function of the distance of the powerhouse cavern from the position of the vertical shaft. The results for the range of the installed flows between 60 and 100 m³/s are presented in the graph below.

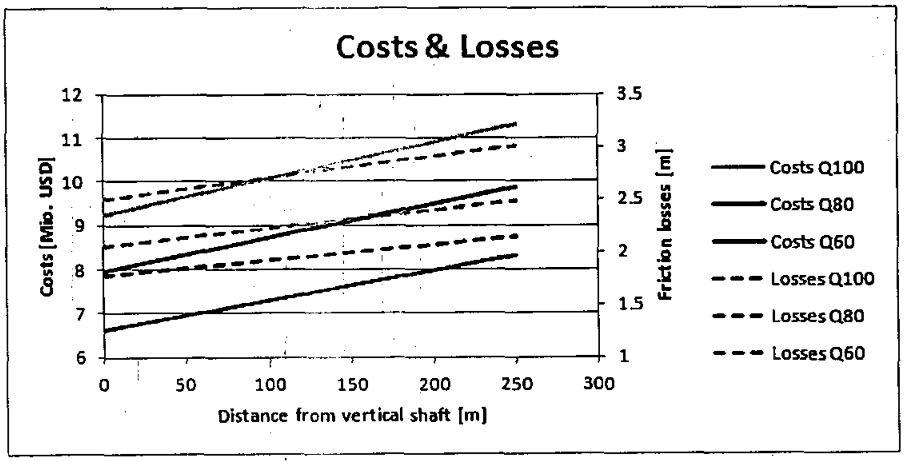


Figure 6-47: Position of the PH cavern - costs and losses

It can be concluded that both costs and friction losses rise with the shifting of the powerhouse cavern towards downstream. Therefore, the powerhouse cavern shall be positioned as close as possible to the vertical shaft, taking into consideration the geological and construction-related constraints.

6.4.5.7 Type and number of units

The project configuration offers the net head of about 170 m, whereas the installed flows range from 60 to 100 m³/s. Such a layout promotes Francis units as the most suitable turbine type. A typical turbine type application chart is given in figure below.

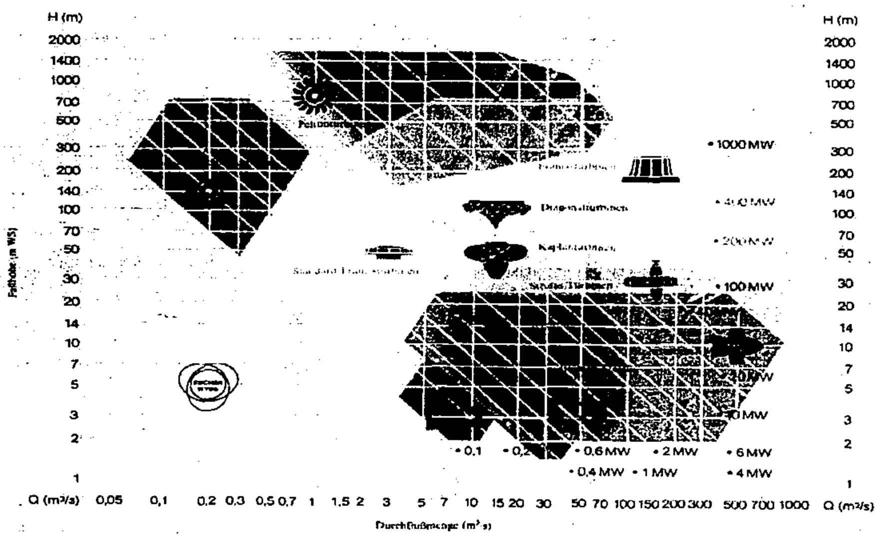


Figure 6-48: Standard range for application of different unit type

As per the developer's express request, the possibility for application of Pelton turbines was assessed. It can be concluded that the Pelton units could possibly be applied with the relatively low installed flow per unit, but that the available head is not within the range for which this type of units is typically considered as a competitive option. A configuration with Pelton turbines would therefore require either a greater number of units in Sharmai powerhouse, or a solution with the vertical arrangement and a large number of nozzles.

As per the quotation received from one of the worldwide leading turbine suppliers, the design of vertical Pelton units for the given range of heads and flows would require a 6 nozzle configuration and is on the verge of applicability in terms of standard solutions. As such, the Pelton option is deemed to trigger cost-related repercussions, as the number of possible suppliers with the respective references is limited.

Therefore, although deemed technically possible, the configuration with Pelton units is not deemed optimal, particularly considering the expected high sediment content, which could cause extensive wear and tear at the nozzles.

It can be concluded that Pelton option can be regarded as competitive to Francis only on account of substantially lower investment costs or higher energy generation.

In order to quantify the difference between the alternative turbine configurations, a cost and energy production analyses for the two options (Francis vs. Pelton) were performed in line with the procedure described in the chapter 7 below. The results are presented in the following table.

Table 6-12: Preliminary cost comparison: Francis vs. Pelton powerhouse

Sharmai HPP U/S Site - Optimization and [illegible]

Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Design discharge	[m³/s]	55	60	65	70	75	80	85	90	95	100	105
Powerhouse - Civil Works	[mUSD]	[illegible]	9.9	10.0	10.4	11.6	11.8	12.2	12.3	13.7	13.8	14.4
Powerhouse - HSS	[mUSD]	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.6
Powerhouse - HM	[mUSD]	12.7	[illegible]	14.8	15.0	15.1	15.4	16.6	16.7	18.6	19.1	19.3
*Powerhouse - EM**	[mUSD]	12.5	13.4	14.0	14.5	15.4	16.0	16.6	17.1	18.0	18.5	18.9
Downstream Surge Tank	[mUSD]	4.7	5.0	5.3	5.8	6.2	6.6	7.0	7.4	7.6	7.9	8.1
Tailrace Tunnel	[mUSD]	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6
Sub Total Direct Costs	[mUSD]	39.6	43.3	45.0	46.7	49.3	51.0	53.4	54.7	59.0	60.4	62.0

Sharmai HPP U/S Site - Optimization and [illegible]

Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Powerhouse - Civil Works	[mUSD]	10.0	11.1	11.9	12.8	13.3	14.3	14.6	15.8	16.2	17.3	18.4
Powerhouse - HSS	[mUSD]	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.6	0.7
Powerhouse - HM	[mUSD]	9.0	9.4	10.3	11.1	11.5	12.3	12.7	13.2	13.4	15.0	16.0
*Powerhouse - EM**	[mUSD]	14.7	15.0	16.0	17.0	17.7	18.8	19.5	20.6	21.2	22.3	23.4
Tailrace Tunnel	[mUSD]	5.9	6.5	6.9	7.2	7.8	8.3	9.0	9.3	9.7	10.1	10.3
Sub Total Direct Costs	[mUSD]	39.6	42.4	45.6	48.7	50.9	54.4	56.3	59.6	61.1	65.2	68.7

Comparison of direct costs shows marginal to substantial cost-advantage for the option with Francis units.

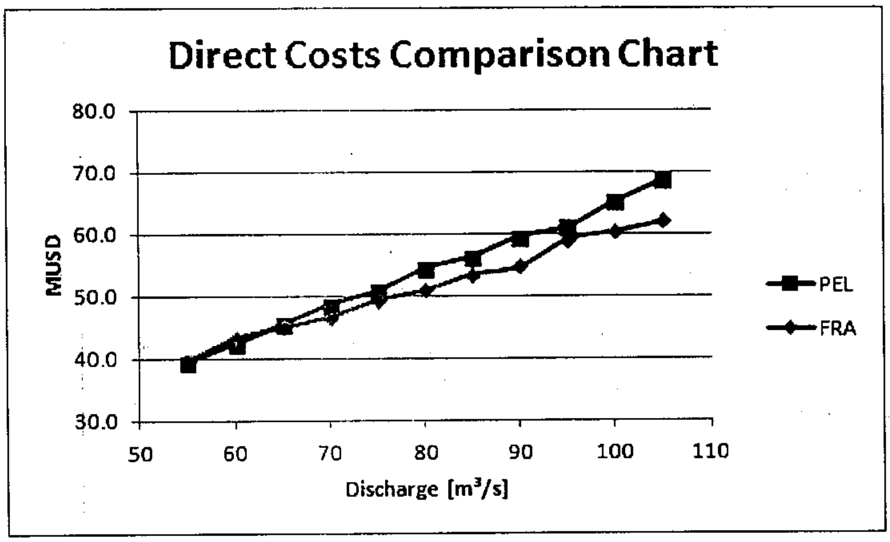


Figure 6-49: Francis vs. Pelton Powerhouse - comparison of direct costs

Nevertheless, the key advantage of Francis turbines lies in somewhat higher efficiency and in the lower head losses due to negative submergence (as opposed to Pelton), as shown in the table below.

Table 6-13: Preliminary energy production comparison: Francis vs. Pelton powerhouse

Sharmai HPP FI2 Site - Optimization and Ranking: Preliminary Energy Production (Francis)												
Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Design discharge	[m³/s]	55	60	65	70	75	80	85	90	95	100	105
Installed power	[MW]	86.5	95.1	103.1	111.3	120.3	129.3	137.7	146.2	154.2	162.1	170.1
Epeak	[GWh/y]	126.6	137.2	147.5	157.4	167.0	176.3	185.4	194.2	202.8	211.2	219.3
Ebase	[GWh/y]	356.9	373.1	388.2	402.3	415.5	427.7	439.0	449.4	459.0	467.9	476.0
Etotal	[GWh/y]	483.5	510.3	535.7	559.7	582.5	604.0	624.4	643.6	661.8	679.1	695.3

Sharmai HPP FI2 Site - Optimization and Ranking: Preliminary Energy Production (Pelton)												
Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Design discharge	[m³/s]	55	60	65	70	75	80	85	90	95	100	105
Installed power	[MW]	82.2	89.7	97.2	104.6	112.0	119.2	126.5	133.7	140.9	148.0	155.2
Epeak	[GWh/y]	119.2	129.2	138.8	148.2	157.2	165.9	174.4	182.8	190.8	198.7	206.3
Ebase	[GWh/y]	335.9	351.2	365.4	378.7	391.1	402.6	413.3	423.0	432.1	440.4	448.1
Etotal	[GWh/y]	455.1	480.3	504.2	526.9	548.2	568.5	587.7	606.8	622.9	639.1	654.4

It can be concluded that the Pelton configuration results in 5-10% lower installed capacity for the same discharge and in about 6% lower total energy generation when compared to Francis option. Based on the presented figures, it can be concluded that the configuration with Francis units represents a comparatively better option in terms of financial performance and is deemed to allow greater variety of suppliers capable to offer a competitive bid. Therefore, the configuration with Francis type units is adopted to be further developed.

Finally, a preliminary comparison of costs for the configurations with 2 and 3 Francis units in Sharmai cavern-type powerhouse was performed, following the procedure elaborated in the next chapter. The results are presented in the table below.

Table 6-14: Preliminary cost comparison: 2 vs. 3 Francis units (cavern)

Sharmai HPP U/S Site - Optimization and Ranking: Preliminary Cost Estimate (2 units)												
Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Design discharge	[m³/s]	55	60	65	70	75	80	85	90	95	100	105
Installed power	[MW]	86.5	95.1	103.1	111.3	120.3	129.3	137.7	146.2	154.2	162.1	170.1
Powerhouse - Civil Works	[mUSD]	8.9	9.9	10.0	10.4	11.6	11.8	12.2	12.3	13.7	13.8	14.4
Powerhouse - HSS	[mUSD]	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.6
Powerhouse - HM	[mUSD]	12.7	14.1	14.8	15.0	15.1	15.4	16.6	16.7	18.6	19.1	19.3
Powerhouse - EM	[mUSD]	12.4	13.3	13.9	14.4	15.3	16.0	16.5	17.0	17.9	18.4	18.9
Sub Total Direct Costs	[mUSD]	34.3	37.7	39.1	40.3	42.5	43.7	45.8	46.7	50.8	51.9	53.2

Sharmai HPP U/S Site - Optimization and Ranking: Preliminary Cost Estimate (3 units)												
Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Powerhouse - Civil Works	[mUSD]	8.0	9.0	9.3	9.4	9.6	10.5	11.8	11.9	12.6	12.5	12.6
Powerhouse - HSS	[mUSD]	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6
Powerhouse - HM	[mUSD]	15.1	17.2	17.2	17.5	17.9	18.5	21.3	21.1	21.2	21.8	22.3
Powerhouse - EM	[mUSD]	15.0	16.0	16.7	17.3	18.0	18.7	19.7	20.3	20.9	21.5	22.6
Sub Total Direct Costs	[mUSD]	38.5	42.6	43.6	44.7	46.0	48.3	53.3	53.9	54.7	56.3	57.4

It can be concluded that the configuration with 3 units would trigger the increase of direct costs for the cavern powerhouse of about 10% as compared with the 2 unit configuration. However, higher number of units would allow flexibility in operation during low inflows, provided that the plant is to be operated in a run-off river mode. This operating mode is deemed applicable during off-peak hours. Additionally, as Sharmai HPP will be connected to the national grid at a remote dead-end, higher number of units is deemed appropriate, in order to comply with the deemed challenges at the grid side.

The following figure illustrates the capacity duration curve for off-peak operation of the plant.

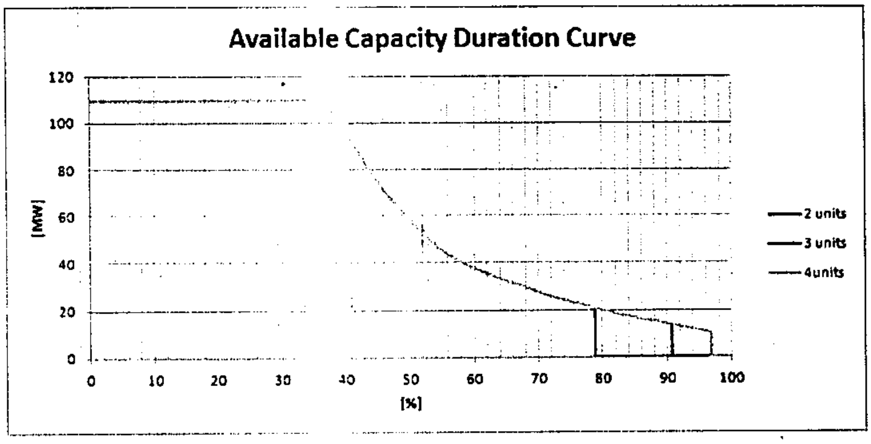


Figure 6-50: Available capacity duration curve during off-peak

It is obvious that the configuration with 3 units would allow for substantially higher flexibility in operation during partial load.

Considering the range of the installed capacities of 80 to 170 MW and the envisioned peaking role of the plant in the national grid/system, the deemed flexibility in operation provided by higher number of units is considered relevant. Therefore, the configuration with 3 vertical Francis units is adopted.

6.4.5.8 Design of Tailrace tunnel

Preliminary calculations for the regulation capability of the plant concluded that the Sharmai powerplant arranged as a cavern requires either a free flow tunnel or a tailrace surge tank in combination with the pressurized tailrace tunnel to avoid critical negative pressures in the turbine draft tubes and tailrace tunnel during sudden load rejections or rapid load changes.

6.4.5.8.1 Free-flow tailrace tunnel

The free-flow tunnel by default implies somewhat higher head losses, as it would need to be designed with a longitudinal slope towards the riverbed. Due to the expected high oscillations in the tailwater levels of about 5 to 10 m due to flood character of flows at Panjkora River with prominent seasonality, the free flow tunnel would need to be positioned at an out let elevation which would allow the free-flow conditions under any flow regime at Panjkora River.

Tunnel outlets below the maximum tailwater level would lead to undefined flow conditions when the water level at the tailwater is in the range of the tunnel outlet. At such conditions, load changes could lead to high fluctuations between pressure and free flow conditions in the tunnel what results in high loads on the tunnel lining and can lead to severe damages. Consequently, positioning of free-flow tailrace tunnel would need to accept the head losses of at least 10 meters attributable to the tailwater level fluctuations, tunnel slope, necessity for an overflow crest at draft tube outlets to ensure sufficient submergence depth and friction caused head losses.

6.4.5.8.2 Pressurized tailrace tunnel

On the costs side, free-flow tunnel implies larger diameter than a pressurized tunnel and, thus higher construction costs. Pressurized tunnel requires a surge tank at its upstream end, as shown in the Table 6-10.

The arrangement with an underground powerhouse has an approx. 780m long tailrace tunnel to the outlet. During fast closure of the guide vanes after a shutdown the pressure in the draft tube will drop significantly and the scheme needs to be safeguarded against undesired water column rupture. The initial analysis has shown that even for relatively moderate, slow guide vane closing times the draft tube pressure drops to absolute vacuum, where water column rupture can occur.

The analyses thus show the necessity for a surge shaft downstream of the turbine draft tubes, either as part of the draft tube gate chambers, or as common shaft after the confluence of the turbines.

By installation of a tailrace surge tank downstream of the powerhouse cavern, the low pressure tailwater tunnel would be decoupled from the waterways and therefore will ensure the regulation capability of the plant and further that the pressure values in the tailrace tunnel and draft tube stay within acceptable ranges. With regards to the regulation capability of the power plant and the construction costs for the underground structures, the tailrace surge tank should be located as close as possible to the powerhouse cavern.

6.4.5.8.3 Conclusion on tailrace tunnel arrangement

An analysis giving focus to the investment costs and the hydraulic losses was performed for the decision on the arrangement of the tailrace tunnel.

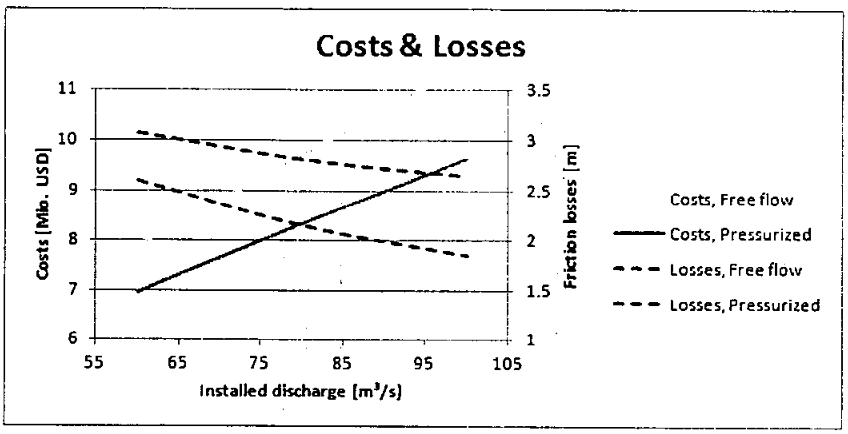


Figure 6-51: Arrangement of the tailrace tunnel - pressurized vs. free-flow

It can be concluded that both investment costs and the head losses rise with the free-flow tailrace tunnel option. Based on the results of these analyses, the tailrace tunnel shall be designed as a pressure tunnel, with a surge chamber at its upstream end.

6.4.6 Definition of the conceptual designs to be further analyzed

Based on the considerations described in the previous sections, the following conceptual layouts were defined. Sharmai HPP comprises the following key components:

- Reservoir, capable to be operated as a daily storage, with the maximum operating level at 1260 masl
- A concrete dam with the inegrated gated spillway and a stilling basin at the two sites: U/S and D/S
- A diversion tunnel at the right abutment
- An open air desander
- System of power waterways
    - power intake
    - low pressure power tunnel
    - surge tank
    - steel lined vertical pressure shaft
- Cavern powerhouse, positioned at the downstream location and equipped with 3 vertical Francis units
- Downstream surge tank
- Pressurized tailrace tunnel
- Access roads.

The following preliminary drawings illustrate the concepts for Sharmai HPP, subject to further optimization analyses.

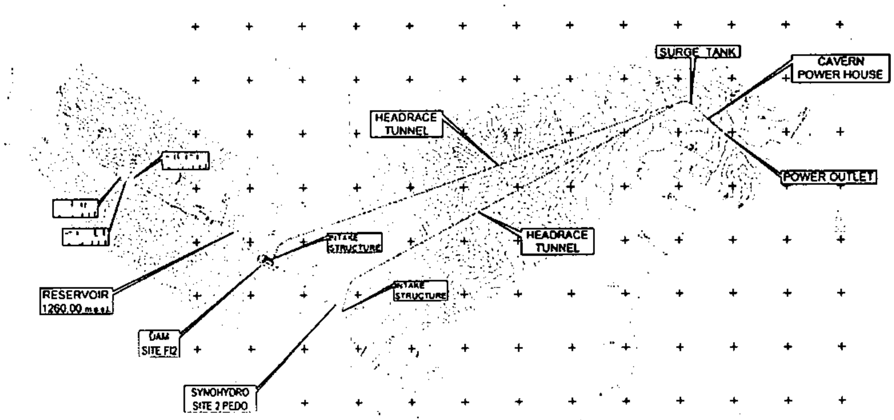


Figure 6-52: Adopted conceptual layout

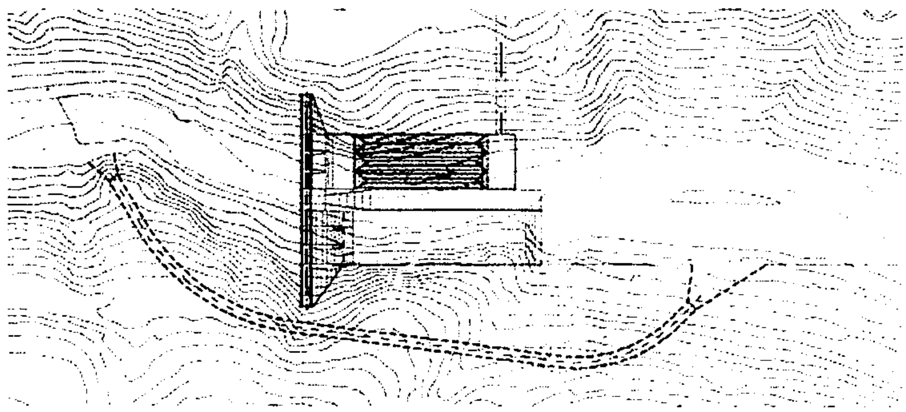


Figure 6-53: Adopted conceptual layout headworks

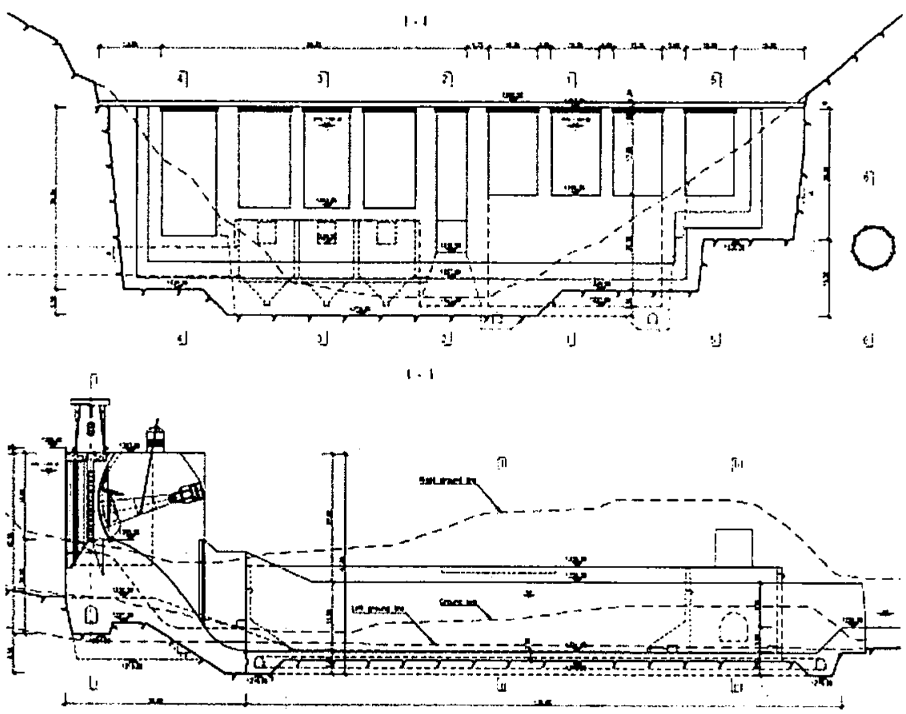


Figure 6-54: Adopted conceptual layout headworks - typical sections

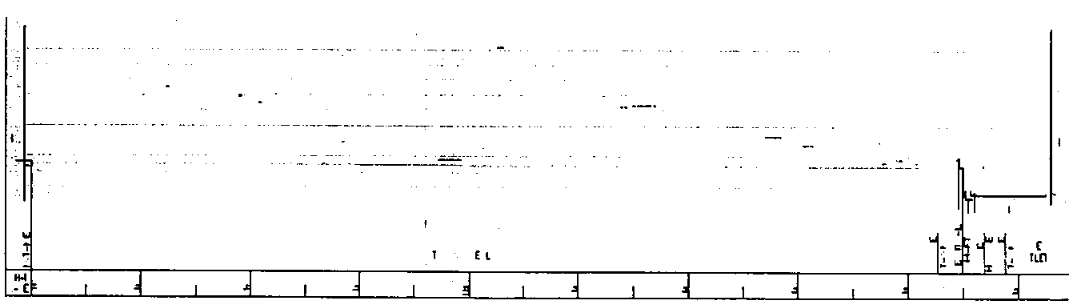


Figure 6-55: Adopted conceptual layout - section through power waterways

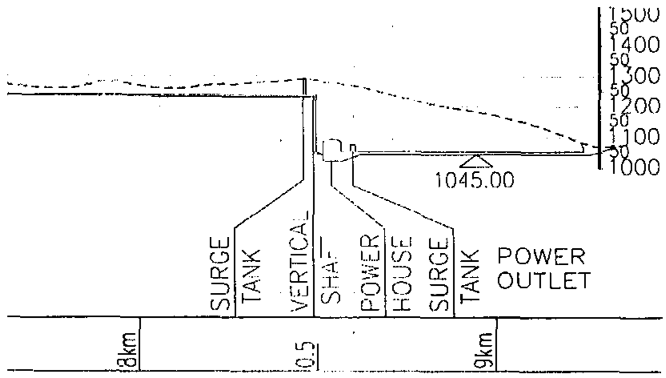


Figure 6-56: Adopted conceptual layout - surge tank, PH cavern and outlet tunnel

The results of the analyses are presented in the following chapter.

6.5 Preliminary Cost Estimate

6.5.1 Introduction

Optimization analysis with the goal to identify the most promising project configuration was performed within the scope of the Feasibility Study. Based on the discussions with PEDO and Sapphire, selection of the installed discharge was based on the maximum technically and financially justifiable capacity of the plant.

As a tool for ranking of the options, the levelized energy cost approach was applied for the comparison between the alternative configurations and compared to the long run marginal cost of Pakistan (LRMC), as discussed in the following chapter.

It has to be emphasized that the purpose of the present analysis was to identify the optimum sizing of Sharmai project / plant, according to the applicable approach and criteria; the optimization analyses of the individual project components are performed for the selected project configuration and presented in a separate report.

The methodology for the cost estimate and the preliminary energy production analysis are summarized below.

6.5.2 Methodology

The preliminary cost estimate is performed for the project layout, optimized and defined as described in the previous chapters. The dimensions (and costs) of various project components such as powerhouse, power waterway system and the generating equipment vary with the installed plant capacity, whereas others, such as the costs of the dam, spillway, river diversion, transmission lines, access roads, for example, have less variation.

The cost function, consequently, might show either a continuous trend, or define the breaking point after which the increment in costs is of a greater order of magnitude than the increment in installed power/energy generation.

In order to provide the basis for the assessment of the project's financial performance, the costs for the adopted project concept were preliminarily estimated. After the most promising conceptual layout is defined, the cost estimate was performed for the variety of the installed discharges, in order to assess the sensitivity of the cost function against the variation in design discharge.

By means of the Consultant's hydropower optimization program HPC (Hydro Power Costing) the design of the project components and the corresponding cost estimate was performed. In order to allow for the comparison between the different considered project configurations, the different dam sites and the operating regimes were considered.

This procedure was applied to powerhouse design discharges in the range from 55 to 105 m^3/s, with 10 m^3/s increments.

For each considered installed discharge at the HPP Sharmai, the design of the project components was adjusted and a bill of quantities was established to estimate the total costs for each alternative. Estimation of costs was performed for the given set of site specific conditions such as topography, hydrology and at the present stage available geological information. The main design parameters, defined in the previous chapters represented the most relevant input to the preliminary cost estimate. These are:

- Water levels (reservoir levels and tailwater curve)
- Reservoir's area and volume curve
- Discharges (powerhouse, diversion flood, spillway design flood)
- Access to the power plant components
- Dam: type, height, inclination of abutments, side slopes, sealing measure, assumed depth of foundation, etc.
- River diversion concept: tunnel geometry, cofferdam heights and types
- Spillway: type, number of gates, discharge capacity, length of chute, etc.
- Energy dissipation structure and the related geometry
- Power intake structure
- Pressurized waterways (tunnels, surge shafts and pressure shafts): length, geometry, design particulars
- Powerhouse (civil works, number and type of units, hydro- and electro-mechanical equipment, hydraulic steel structures)
- Tail race
- Transmission line (25 km, preliminarily estimated)
- Costs for the implementation of mitigation measures for environmental and social impacts
- Indirect costs.

For all components listed above, data collected during field visits and the conclusions derived during the previous project stage were applied; design parameters optimized in the present stage, were considered; accurate topographic maps and the high resolution satellite photos were used; results of the performed hydrological analysis and the geological assessment were applied for the modeling.

For the purpose of the evaluation of the considered conceptual layouts, certain design parameters were taken over from the performed optimization analyses, as presented in the previous chapters and applied for all options, in order to rationalize the number of variables and still to retain a high level of confidence in the results. The following input parameters were considered for the various installed discharges:

- power tunnel was designed with concrete lining and for the flow velocity of 3.5 m/s
- geological conditions along the pressure tunnel assumed as: 10% vary bad, 20% bad, 40% average, 20% good, 10% very good
- velocity in the steel-lined pressure shaft of 5 to 6 m/s
- 2 to 3 units in the powerhouse
- velocity in the tailrace tunnel of 3.5 m/s
- constant, preliminarily estimated ESIA-related costs for all alternatives.

The following table summarizes the key technical inputs to the assessment of the construction costs for the considered alternatives.

Table 6-15: Cost estimate - key inputs; U/S site

Sharmai HPP U/S Site - Optimization and Ranking: Input data table												
Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Design discharge	[m^3/s]	55	60	65	70	75	80	85	90	95	100	105
Dam height	[m]	45										
Dam type	[-]	Concrete Gravity										
Reservoir MWL	[masl]	1260										
Tailwater level	[masl]	1065										
Gross head	[m]	195										
Spillway capacity (10'000y / PMF)	[m^3/s]	5000										
Diversion flood	[m^3/s]	650										
Length of access roads - new construction/upgrade	[m]	3000										
Length of diversion tunnel	[m]	310										
Inner diameter of diversion tunnel	[m]	9										
Number of desander chambers	[-]	3										
Design discharge for desander (n-1; 150% of Q)	[m^3/s]	82.5	90.0	97.5	105.0	112.5	120.0	127.5	135.0	142.5	150.0	157.5
Length of power tunnel	[m]	8500										
Power tunnel inner diameter	[m]	4.50	4.70	4.85	5.00	5.20	5.40	5.55	5.70	5.80	5.90	6.00
Surge tank inner diameter	[m]	15										
Length of pressure shaft	[m]	244										
Pressure shaft inner diameter	[m]	3.40	3.55	3.70	3.85	4.00	4.15	4.25	4.40	4.50	4.60	4.70
Number of units	[-]	3										
Type of units	[-]	Francis, vertical										
Downstream surge tank inner diameter	[m]	10										
Length of the tailrace channel	[m]	750										
Tailrace tunnel inner diameter	[m]	4.50	4.70	4.85	5.00	5.20	5.40	5.55	5.70	5.80	5.90	6.00
Transmission line	[km]	25										
Installed power	[MW]	89.2	98.0	106.3	114.7	124.0	133.3	141.9	150.7	158.9	167.1	175.4
Installed power (at busbars)	[MW]	86.5	95.1	103.1	111.3	120.3	129.3	137.7	146.2	154.2	162.1	170.1

Table 6-16: Cost estimate - key inputs; D/S site

Sharmai HPP D/S Site - Optimization and Ranking; Input data table						
Description	Alternative	1	2	3	4	5
Design discharge	[m³/s]	60	70	80	90	100
Dam height	[m]	65				
Dam type	[-]	Concrete Gravity				
Reservoir MWL	[masl]	1260				
Tailwater level	[masl]	1065				
Gross head	[m]	195				
Spillway capacity (10'000y / PMF)	[m³/s]	5000				
Diversion flood	[m³/s]	650				
Length of access roads - new construction/upgrade	[m]	3000				
Length of diversion tunnel	[m]	360				
Inner diameter of diversion tunnel	[m]	8				
Number of desander chambers	[-]	3				
Design discharge for desander (n-1; 150% of Q)	[m³/s]	90.0	105.0	120.0	135.0	150.0
Length of power tunnel	[m]	7500				
Power tunnel inner diameter	[m]	4.60	5.00	5.30	5.65	6.00
Surge tank inner diameter	[m]	15				
Length of pressure shaft	[m]	244				
Pressure shaft inner diameter	[m]	3.40	3.85	3.70	3.85	4.00
Number of units	[-]	3				
Type of units	[-]	Francis, vertical				
Downstream surge tank inner diameter	[m]	10				
Length of the tailrace channel	[m]	750				
Tailrace tunnel inner diameter	[m]	4.60	5.00	5.30	5.65	6.00
Transmission line	[km]	25				
Installed power	[MW]	95.1	111.3	129.4	147.0	162.1
Guaranteed capacity (P90)	[MW]	94.8	107.2	114.3	114.3	114.3

The present analysis had as the main target the assessment of the robustness of the project's costs against the variation in the design discharge for the variety of options.

6.5.3 Results

The direct costs were estimated by multiplying the calculated quantities with the assumed unit rates for the major construction activities for each component of the project. The unit rates for civil works were derived from tender documents and feasibility studies from similar projects in the region. The corresponding rates were analyzed and escalated from the cost reference dates to the current date.

The costs for hydro- and electro-mechanical equipment and hydraulic steel structures were estimated on the basis of the extensive data from the recent bids for similar size projects. The ESIA-related costs were preliminarily estimated. The final budget for the implementation of the ESIA mitigation measures will be defined at a later stage of project development. Constant ESIA related costs were applied for all options, due to negligible influence of the variation in installed discharge.

Besides the direct cost of the project, indirect costs (mobilization, contingencies - physical and price and engineering and administration costs) were estimated and added as the percentage to the direct costs of the items, due to uncertainties attributable to the preliminary design considerations applied in costing.

The obtained results are intended to enable comparison of the considered project alternatives on equal basis and to support the design optimization process. For the finally selected design discharge, the design will be developed to a full feasibility level and the investment cost will be determined based on actual quantities, derived from the drawings, updated unit rates and costs.

Detailed output tables of the HPC software for all considered project alternatives are available in Annex 1.

The following table summarizes the results obtained through the HPC program.

Table 6-17: Optimization analysis - summary of costs (HPC, U/S site)

Sharmai HPP U/S Site - Optimization and Ranking: Preliminary Cost Estimate												
Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Design discharge	[m³/s]	55	60	65	70	75	80	85	90	95	100	105
Installed power	[MW]	89.2	98.0	106.3	114.7	124.0	133.3	141.9	150.7	158.9	167.1	175.4
Installed power (at busbars)	[MW]	86.5	95.1	103.1	111.3	120.3	129.3	137.7	146.2	154.2	162.1	170.1
Access roads	[mUSD]	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
ESIA measures; Indemnification; Expropriation	[mUSD]	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
Dam and Spillway; Power Intake; River Diversion	[mUSD]	41.9	42.0	42.1	42.3	42.5	42.6	42.7	42.9	43.0	43.1	43.3
Sediment tank	[mUSD]	19.4	20.1	22.0	23.9	26.2	28.2	30.3	32.4	34.6	36.8	39.0
Headrace tunnel	[mUSD]	49.3	53.4	56.6	61.1	65.6	70.3	74.4	78.1	80.9	83.5	86.3
Surge tank upstream	[mUSD]	1.6	1.6	1.7	1.7	1.6	1.6	1.7	1.7	1.7	1.7	1.8
Pressure shaft	[mUSD]	2.4	2.6	2.8	2.9	3.1	3.3	3.4	3.6	3.7	3.9	4.0
Powerhouse - Civil Works	[mUSD]	8.0	9.0	9.3	9.4	9.6	10.5	11.8	11.9	12.0	12.5	12.6
Powerhouse - HSS	[mUSD]	0.4	0.4	0.4	0.4	0.5	0.5	0.6	0.5	0.6	0.6	0.6
Powerhouse - HM	[mUSD]	15.1	17.2	17.2	17.6	17.9	18.5	21.3	21.1	21.2	21.8	22.3
Powerhouse - EM	[mUSD]	16.0	16.0	16.7	17.3	18.0	18.7	19.7	20.3	20.9	21.6	22.0
Transmission line	[mUSD]	6.4	6.4	6.4	6.8	6.8	6.8	6.8	6.8	6.8	6.8	6.8
Tailrace tunnel	[mUSD]	4.7	5.0	5.3	5.8	6.2	6.6	7.0	7.4	7.6	7.9	8.1
Surge tank downstream	[mUSD]	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6
Sub Total Direct Costs	[mUSD]	168.7	178.2	184.9	193.8	202.5	212.2	224.1	231.4	237.7	244.5	251.1
Indirect Costs												
Mobilization (7%)	[mUSD]	11.8	12.5	12.9	13.6	14.2	14.9	15.7	16.2	16.6	17.1	17.6
Minor items and unforeseen (20%)	[mUSD]	33.7	35.6	37.0	38.8	40.5	42.4	44.8	46.3	47.5	48.9	50.3
Engineering & Administration (8%)	[mUSD]	13.5	14.3	14.8	15.5	16.2	17.0	17.9	18.5	19.0	19.6	20.1
TOTAL PROJECT COST (HPC)	[mUSD]	227.7	240.6	249.7	261.7	273.4	286.5	302.5	312.3	320.9	330.1	339.2
TOTAL PROJECT COST (HPC, linear regression)	[mUSD]	228.6	240.1	251.5	263.0	274.4	285.9	297.3	308.8	320.2	331.7	343.1
COST per MW	USD/kW	2.632	2.530	2.422	2.351	2.272	2.216	2.197	2.136	2.081	2.036	1.994

It can be seen that the project specific investments are in the range of 2,000 to 2,650 USD/kW.

For the powerhouse and waterway configurations considered, the key equipment parameters, such as turbine and generator dimensions (runner speed), setting, etc. are subject to variation. In practice and similarly in the Consultant's HPC software, these variations occur stepwise and not continuously. Starting from a certain threshold value certain equipment parameters remain constant unless the subsequent threshold value is reached and the parameter increases.

This is the case, e.g. for the turbine runner speed and thus the turbine dimensions and costs (and, subsequently, the dimensions and costs of the powerhouse civil works and lifting equipment, for example). In particular, the threshold value for changes in the turbine runner speed may vary from one turbine manufacturer to the other.

For the purpose of optimizing the installed discharge, instead of these stepped cost characteristics, the function of cost was established by best fit regression analysis. This approach avoids the effect of such, to a certain extent, arbitrary definition of threshold values on the resulting optimum installed capacity.

Finally, the function which gives the relation between the installed discharge and the investment costs for Sharmai HPP is established, as given in the figure below:

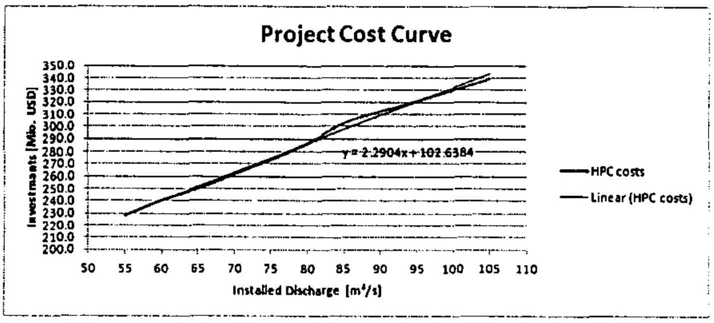


Figure 6-57: Project cost function (U/S site)

The same analysis was performed for the D/S site.

Table 6-18: Optimization analysis - summary of costs (HPC, D/S site)

Sharmai HPP D/S Site - Optimization and Ranking; Preliminary Cost Estimate						
Description	Alternative	1	2	3	4	5
Design discharge	[m³/s]	60	70	80	90	100
Installed power	[MW]	98.0	114.7	133.4	151.5	167.1
Installed power (at busbars)	[MW]	95.1	111.3	129.4	147.0	162.1
Access roads	[mUSD]	1.2	1.2	1.2	1.2	1.2
ESIA measures; Indemnification; Expropriation	[mUSD]	4.1	4.1	4.1	4.1	4.1
Dam and Spillway; Power Intake; River Diversio	[mUSD]	85.5	85.9	86.1	86.5	86.8
Sediment tank	[mUSD]	20.1	23.9	28.2	32.4	36.8
Headrace tunnel	[mUSD]	45.3	53.9	59.9	67.8	74.5
Surge tank upstream	[mUSD]	1.6	1.4	1.6	1.6	1.6
Pressure shaft	[mUSD]	2.6	3.0	3.3	3.6	3.9
Powerhouse - Civil Works	[mUSD]	9.0	9.4	10.5	11.9	12.5
Powerhouse - HSS	[mUSD]	0.4	0.4	0.5	0.5	0.6
Powerhouse - HM	[mUSD]	17.2	17.5	18.5	21.1	21.8
Powerhouse - EM	[mUSD]	16.0	17.3	18.7	20.3	21.5
Transmission line	[mUSD]	6.4	6.8	6.8	6.8	6.8
Tailrace tunnel	[mUSD]	4.8	5.8	6.4	7.3	8.2
Surge tank downstream	[mUSD]	0.5	0.5	0.6	0.6	0.6
Sub Total Direct Costs	[mUSD]	**214.7**	**231.2**	**246.5**	**265.7**	**280.7**
Indirect Costs						
Mobilization (7%)	[mUSD]	15.0	16.2	17.3	18.6	19.6
Minor items and unforeseen (20%)	[mUSD]	42.9	46.2	49.3	53.1	56.1
Engineering & Administration (8%)	[mUSD]	17.2	18.5	19.7	21.3	22.5
TOTAL PROJECT COST (HPC)	[mUSD]	**289.8**	**312.1**	**332.7**	**358.7**	**378.9**
TOTAL PROJECT COST (HPC, linear regression)	[mUSD]	**289.5**	**312.0**	**334.5**	**356.9**	**379.4**
COST per MW	USD/kW	3.048	2.804	2.571	2.440	2.338

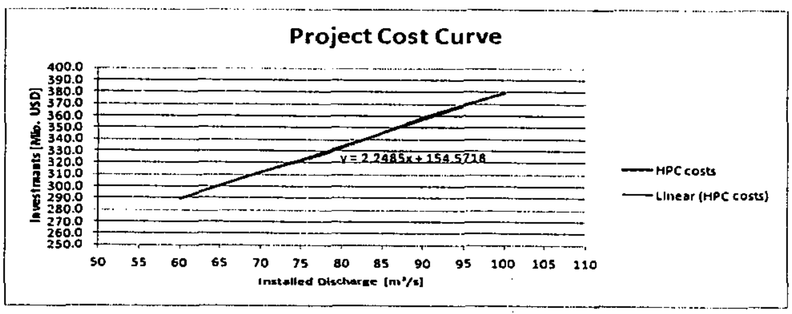


Figure 6-58: Project cost function (PEDO site)

6.6 Preliminary Power Production Analysis

6.6.1 Methodology

In order to quantify the direct benefits attributable to the project, power production at Sharmai HPP was preliminarily analyzed. Similarly to the approach presented in the previous chapter, the power production estimate was performed for the variety of installed flows, from 55 to 105 m³/s with 10 m³/s increment.

Results of the bathymetric survey were used for the generation of the tailwater curve (TWC) at Sharmai plant. The TWC was simulated by means of Hec-RAS and is presented in the figure below.

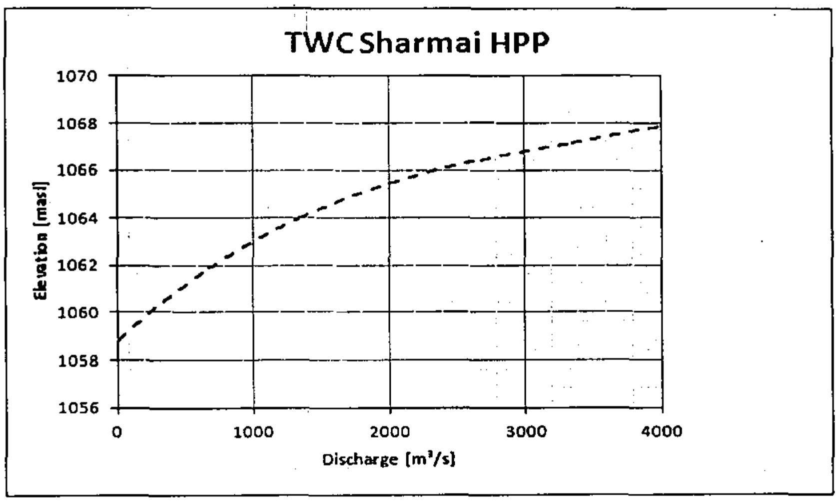


Figure 6-59: Tailwater curve

The reservoir simulation model was based on the series of daily reservoir inflow values at the project site for the period 1961-2017 and did consider the following inputs:

- Environmental minimum release
- reservoir operating water levels (maximum, minimum and flood)
- tailwater curve
- actual gross head (in daily steps)
- hydraulic losses along the power waterways
- actual (daily) flow availability and the considered installed discharges
- efficiency of the generating equipment.

As previously concluded, Sharmai HPP will operate in peaking mode, over 4 hrs/day. Due to the existence of a reservoir with daily storage capacity, during the periods of high inflow, Sharmai HPP will be able to generate power outside the peaking hours. However and from the perspective of the power system, the major value the plant brings to the system is the reliability in provision of additional capacity and energy to it.

Therefore, Sharmai HPP shall be sized according to its envisioned peaking role in the system and with respect to the capacity and energy it can produce with the high guaranteed availability of 90%. For the purpose of the subsequent optimization of the sizing of the plant, the energy generated at Sharmai HPP is divided into groups with different characteristics:

- Peak energy, produced during the peaking time and with the capacity guaranteed with 90% of time
- Off-peak energy, generated outside of the peaking hours and with the lower guaranteed availability.

6.6.2 Results

The results of the power production analysis are visualized in the graphs below. Same analysis was performed for both U/S and D/S sites for the variety of the installed flows, ranging from 55 to 105 m³/s.

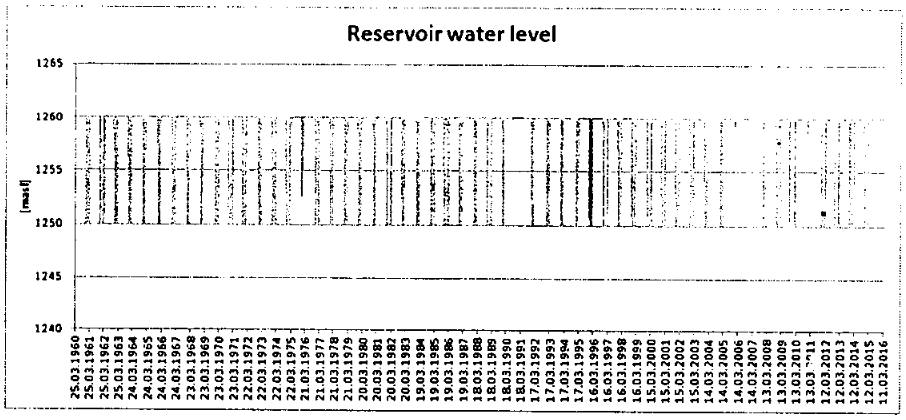


Figure 6-60: Sharmai HPP - reservoir water levels oscillation, U/S site

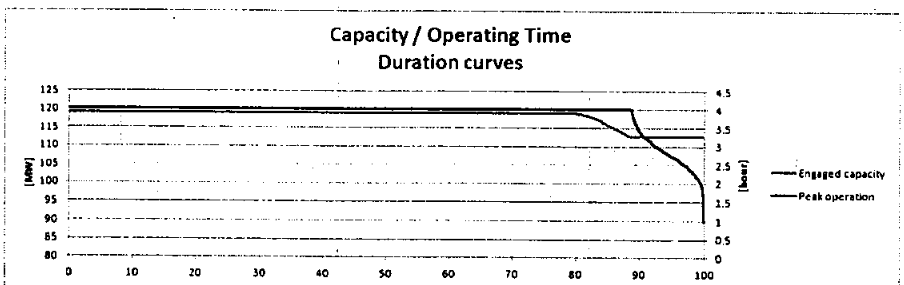


Figure 6-61: Sharmai HPP - capacity/operating time duration curves, U/S site

Strong seasonality in energy generation can be observed in the following graph.

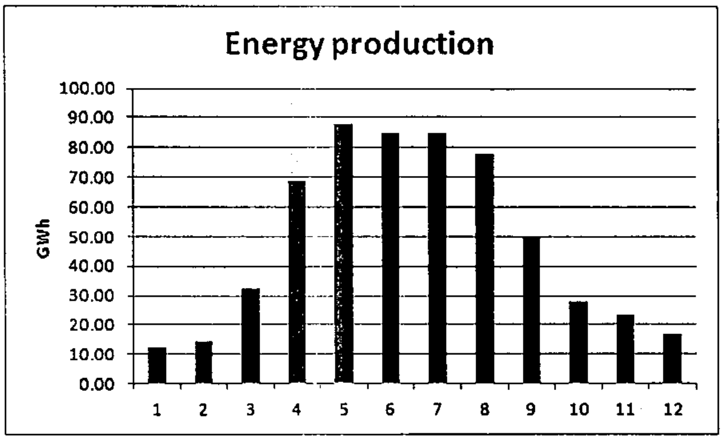


Figure 6-62: Sharmai HPP - total energy production graph, U/S site

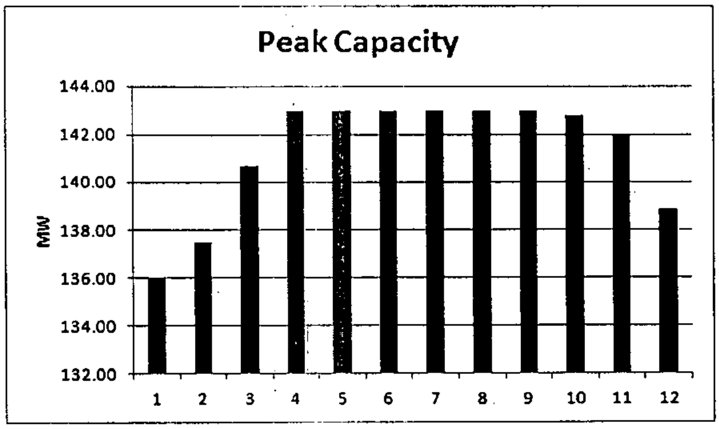


Figure 6-63: Sharmai HPP - peak capacity graph, U/S site

The results for the U/S and D/S site are given in the following tables.

Table 6-19: Sharmai HPP, U/S site - summary of energy production figures

Sharmai HPP U/S Site - Optimization and Ranking: Input data table												
Description	Alternative	1	2	3	4	5	6	7	8	9	10	11
Installed power	[MW]	89.2	98.0	106.3	114.7	124.0	133.3	141.9	150.7	158.9	167.1	175.4
Installed power (at busbars)	[MW]	86.5	95.1	103.1	111.3	120.3	129.3	137.7	146.2	154.2	162.1	170.1
Guaranteed capacity (P90)	[MW]	86.5	94.8	101.2	107.2	108.2	108.2	108.2	108.2	108.2	108.2	108.2
Energy production - Peak	[GWh/y]	126.6	137.2	147.5	157.4	167.0	176.3	185.4	194.2	202.8	211.2	219.3
Energy production - Base	[GWh/y]	356.9	373.1	388.2	402.3	415.5	427.7	439.0	449.4	459.0	467.9	476.0
Energy production - Total	[GWh/y]	483.5	510.3	535.7	559.7	582.5	604.0	624.4	643.6	661.8	679.1	695.3
Plant factor	[-]	0.64	0.61	0.59	0.57	0.55	0.53	0.52	0.50	0.49	0.48	0.47

Table 6-20: Sharmai HPP, D/S site - summary of energy production figures

Sharmai HPP D/S Site - Optimization and Ranking: Input data table						
Description	Alternative	1	2	3	4	5
Installed power	[MW]	98.0	114.7	133.4	151.5	167.1
Installed power (at busbars)	[MW]	95.1	111.3	129.4	147.0	162.1
Guaranteed capacity (P90)	[MW]	94.8	107.2	114.3	114.3	114.3
Energy production - Peak	[GWh/y]	137.8	158.2	177.2	195.3	212.3
Energy production - Base	[GWh/y]	372.6	401.6	426.8	448.3	466.6
Energy production - Total	[GWh/y]	510.4	559.8	604.0	643.6	678.9
Plant factor	[-]	0.61	0.57	0.53	0.50	0.48

The following table offers the overview to the guaranteed capacity Sharmai project is capable to deliver, being operated over 4 hrs per day. It is interesting to see that, regardless to the total installed capacity at the plant, the distribution of inflows and the daily storage character of the reservoir limit the guaranteed capacity P90 to about 108 MW for the U/S and about 115 MW for the D/S site..

Table 6-21: Sharmai HPP U/S dam site, guaranteed capacity P90

Discharge	Installed Power	Production Peak	Production Off-Peak	Guaranteed Capacity P90
[m³/s]	[MW]	[GWh/y]	[GWh/y]	[MW]
55	89.2	126.6	356.9	86.5
60	98.0	137.2	373.1	94.5
65	106.3	147.5	388.2	100.5
70	114.7	157.4	402.3	106.0
75	124.0	167.0	415.5	108.3
80	133.3	176.3	427.7	108.3
85	141.9	185.4	439.0	108.3
90	150.7	194.2	449.4	108.3

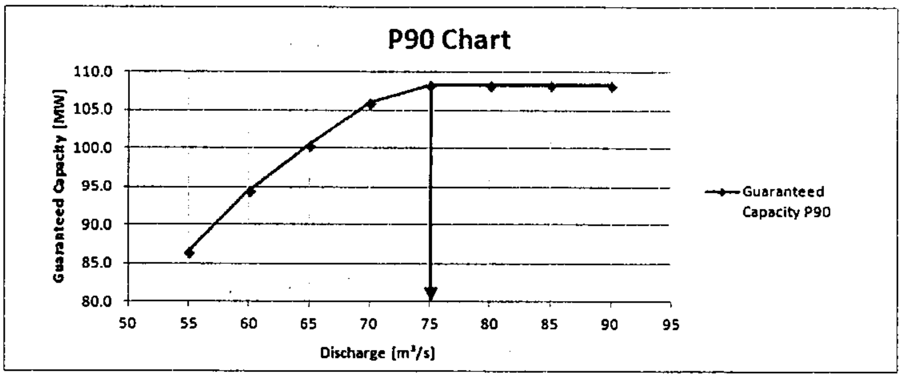
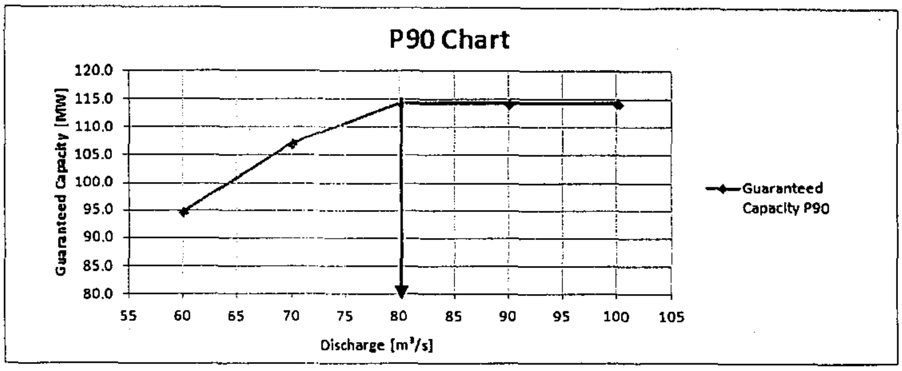


Figure 6-64: Sharmai HPP U/S dam site - guaranteed capacity chart (P90)

Table 6-22: Sharmai HPP D/S dam site, guaranteed capacity P90

Discharge	Installed Power	Production Peak	Production Off-Peak	Guaranteed Capacity P90
[m³/s]	[MW]	[GWh/y]	[GWh/y]	[MW]
60	98.0	137.8	372.6	94.8
70	114.7	158.2	401.6	107.2
80	133.4	177.2	426.8	114.3
90	151.5	195.3	448.3	114.3
100	167.1	212.3	466.6	114.3

Figure 6-65: Sharmai HPP D/S dam site - guaranteed capacity chart (P90)



6.7 Conclusion on Sharmai hydropower project sizing

6.7.1 Introduction

The performed optimization analyses resulted in the definition of the most promising technical layout for Sharmai HPP. Comparative analyses elaborated in the previous chapters (first focusing on technical and later on financial and economic performance of the project) served to:

- identify the optimum conceptual layout of the project
- define the optimum layout of the key project components
- give estimate on the expected construction costs for the variety of installed discharges / capacities
- estimate the revenues under various operating scenarios and
- assess the project's resulting financial and economic performance
- perform sensitivity analyses, i.e. assess the robustness of the project's resulting financial and economic performance against the variation in the main design input parameters.

The financial and economic analyses performed for all considered project configurations in terms of installed capacity confirmed the project's robustness and the viable financial and economic performance.

Due to the limited storage capacity, which allows for a daily regulation, the selection of the installed capacity at the plant is in function of the considered operating time of the plant in the system.

Environmental and social constraints related to the envisioned project configuration confirm the adopted water level in Sharmai reservoir of 1260 masl as the upper boundary in terms of impacts deemed acceptable and manageable.

The different project configurations were compared on the basis of:

- Plant factor

- Economic performance
- Levelized Tariff according to NEPRA guidelines.

The following sections give summary of the results of the optimization analyses, focusing on the dam site selection and sizing of Sharmai HPP.

6.7.2 Dam site selection

Following the conclusions elaborated in the previous chapters, the selection of the project dam site was carried out. As concluded in the chapters 5 and 6, the D/S offers slightly higher guaranteed capacity and the overall energy production compared to the U/S site; however, D/S site offers it under substantially higher investment costs. The plant factors at the two sites are almost identical.

Following the procedure elaborated in chapter 8, the financial performance of the two sites was compared based on the levelized tariff and NPV approach. The results are summarized in the graphs below.

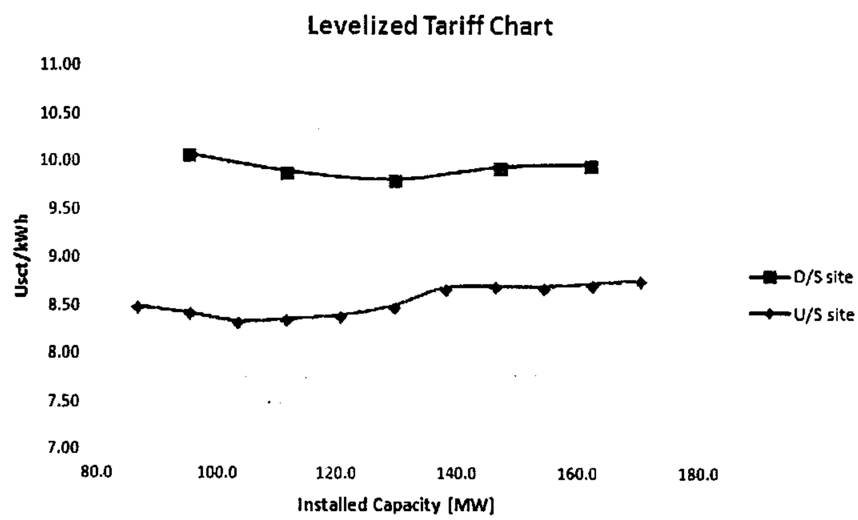


Figure 6-66: Sharmai HPP - dam site comparison (levelized tariff based)

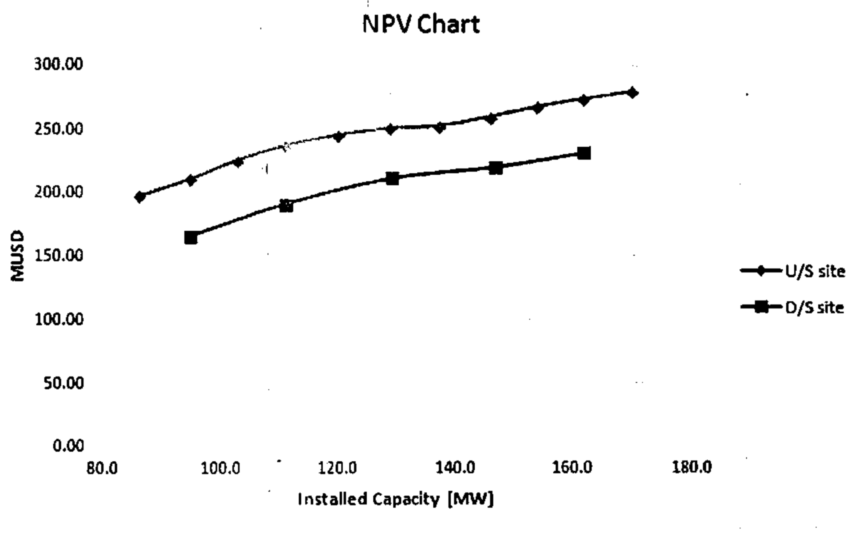


Figure 6-67: Sharmai HPP - dam site comparison (NPV based)

It can be concluded that the U/S site offers comparatively better financial performance for Sharmai HPP. Therefore, U/S site was selected as the dam site for Sharmai HPP. The subsequent plant sizing analyses consider the project configuration with U/S dam site.

6.7.3 Sizing of Sharmai HPP

6.7.3.1 Plant factor approach

The following figure shows development of plant factor with the change of installed capacity.

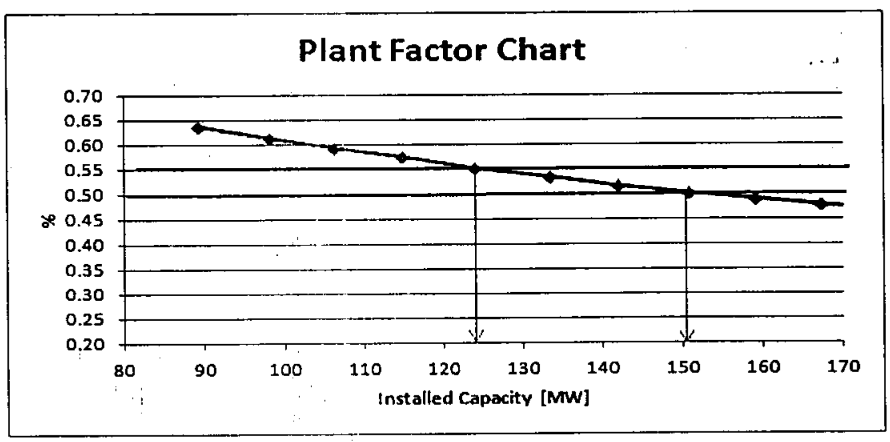


Figure 6-68: Plant factor chart

It can be concluded that the Plant factor analysis indicates the range of 125 to 150 MW as appropriate sizing for Sharmai HPP. Plant factor, compared to other recent IPPs in Northern Pakistan rivers indicates a reasonable plant factor between 0.50 and 0.55. Therefore and based solely on the plant factor approach, it can be concluded that the installed capacity at Sharmai HPP shall not exceed 150 MW.

Similarly, the installed capacity at Sharmai HPP shall not be less than 125 MW – this value gives the maximum guaranteed capacity for Sharmai HPP. Above 125 MW no additional guaranteed capacity can be achieved, but only additional energy, as shown on the graph below.

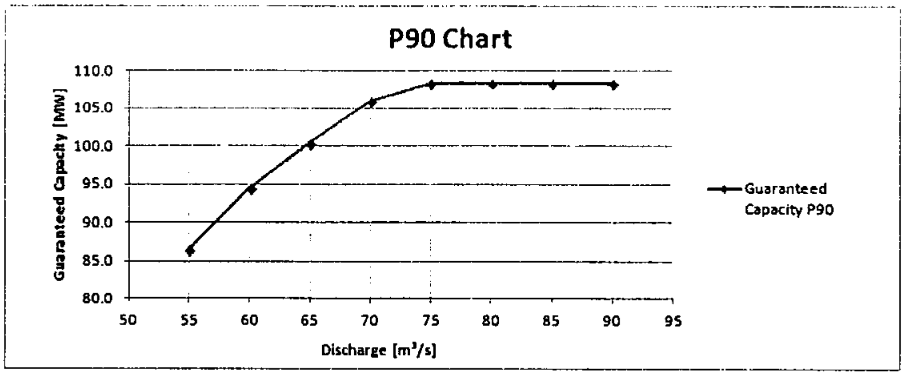


Figure 6-69: Sharmai HPP - guaranteed capacity chart (P90)

6.7.3.2 NPV approach

NPV chart shows an increasing trend within the considered range of installed capacities. NPV chart shows that no distinct optimum is reached on NPV basis due to additional costs and additional energy. However, NPV has a steeper increasing trend up to the installed capacities of about 125 MW; after this point, the increment in NPV slightly reduces.

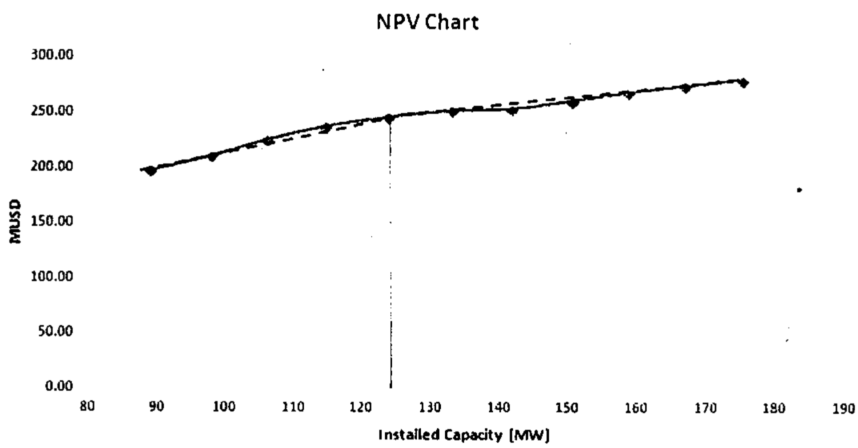


Figure 6-70: NPV chart

The observed increasing trend of NPV can be explained by the continuous increase of both costs and energy production values over the considered range of installed capacities.

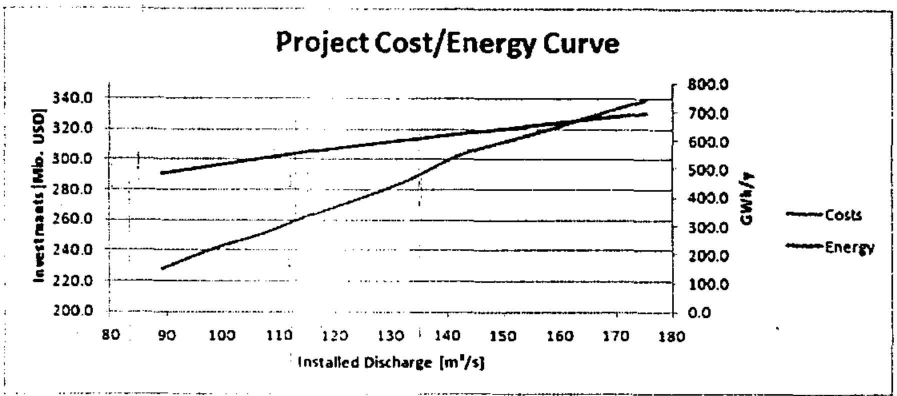


Figure 6-71: Project cost / energy curve

6.7.3.3 Levelized tariff approach

Finally, the results of the financial and economic analyses conclude on the Levelized tariff (according to NEPRA guidelines) for the different considered project configurations.

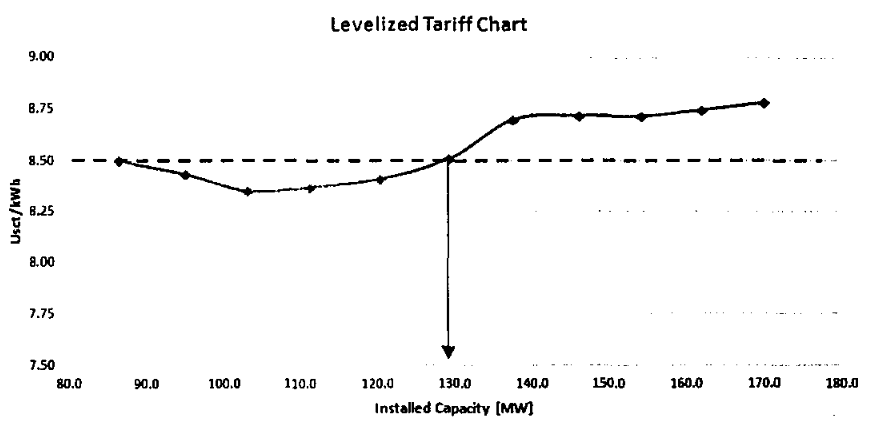


Figure 6-72: Levelized tariff chart

Project Levelized tariff over the range of the installed capacities between 125 MW and 150 MW lies between 8.5 and 8.7 Usc/kWh. The maximum installed capacity at the plant, which maximizes utilization of hydropower potential of the site under financially competitive conditions and with the acceptable plant factor should be considered.

6.8 Conclusion

According to the information made available, Sharmai HPP is to be regarded as a part of a wider plan for utilization of hydropower potential of Panjkora River. There are 2 planned run-off river hydropower projects located upstream of Sharmai and 4 located downstream of it.

There are two upstream located projects, namely:

- Patrak Shringal HPP, 22 MW
- Kalkot Barikot Patrak HPP, 47 MW.

There are four downstream located plants, namely:

- Saidabad HPP, 49 MW
- Wari HPP, 35 MW
- Shalfalam HPP, 60 MW
- Khal Barun HPP, 65 MW
- Koto HPP, 41 MW, 126 m^3/s installed flow (Public Sector Under construction project).

Based on the results of the performed analyses, the alternative with the design discharge of 90 m^3/s was selected, giving an output in the range of 150 MW at Sharmai HPP. Such sizing is recommended from technical and financial aspects, due to following reasons:

- maximization of energy generation, installed and guaranteed plant capacity
- maximize utilization of natural hydropower potential the project site offers
- reasonable levelized tariff
- appropriate plant factor.

The proposed sizing of Sharmai HPP confirms the results of the previous studies, which concluded on 150 MW as of the optimum capacity at the plant.

The adopted conceptual layout and the recommended sizing for Sharmai HPP represent the subjects to development to full feasibility study level design.

7. Feasibility Design

7.1 Introduction

The following chapter describes the adopted feasibility level design of civil works and equipment and summarizes the findings of the energy production analyses, cost estimate and implementation plan for Sharmai hydropower project.

The present document provides results of the additional and more accurate analyses, based on the detailed topographic maps, geological investigations, hydrological and sediment studies and design drawings. The analyses reported under this chapter are accorded to the conclusions and constraints as per the ESIA Report, prepared by Hagler Bailly, the ESIA consultant.

7.2 Design of Civil Works

7.2.1 General

Sharmai HPP will have the following general configuration:

- reservoir, capable to be operated as a daily storage, with the maximum operating level at 1260 masl and minimum operating level at 1255
- a concrete dam with the integrated gated spillway and a stilling basin
- a diversion tunnel at the right abutment
- an open air desander with 3 chambers designed to 50% installed powerhouse flow each
- system of power waterways
  1. power intake
  2. low pressure power tunnel
  3. surge tank
  4. steel lined vertical pressure shaft
- cavern powerhouse, equipped with 3 vertical Francis units
- downstream surge tank
- pressurized tailrace tunnel
- transmission line to Chakdara Substation
- access roads
- construction camps, quarry and depo sites and other temporary facilities.

7.2.2 Dam

Sharmai dam is a concrete gravity structure, which shall accommodate gated spillway block, bottom outlet block and the block with the inlets to sediment tanks. The dam shall have top width of 8m at the elevation 1265 masl and a vertical upstream face. Dam shall be founded on sound rock. Due to weathering, grout curtain performed from a grouting gallery placed throughout the length of the contact with the base rock will be provided.

The bottom outlet block is placed in between the spillway and the sediment tank blocks, allowing flushing of the sediments deposited in front of the power intake and draw down of the reservoir in case needed.

Dam width in the crest is about 150 m.

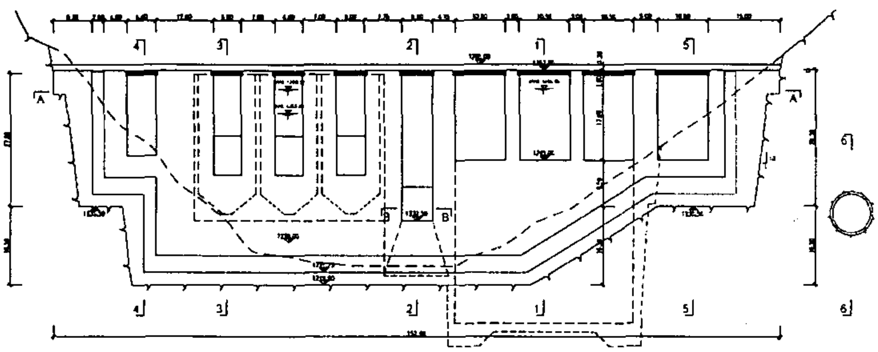


Figure 7-1: Dam section

7.2.3 Spillway and Stilling Basin

Sharmai Spillway shall provide enough discharge capacity to allow release and dissipation of energy of flows up to PMF. The spillway will be equipped with 3 radial gates, each with a flap gate on top, capable of release of floating debris. The spillway sill is placed at the elevation 1245 masl, i.e. 10 m below the minimum OWL in the reservoir. Such configuration allows for flushing of the sediments accumulated in the reservoir and ensures that the reservoir active storage stays available throughout the project's lifetime.

Spillway will be equipped with 3 radial gates (WxH = 10.5 x 15.5 m).

Sizing of the spillway was performed in line with the adopted design criteria provided. The following figure shows the results of the hydraulic analysis for Sharmai Spillway.

The following figure shows the discharge capacity of Sharmai Spillway.

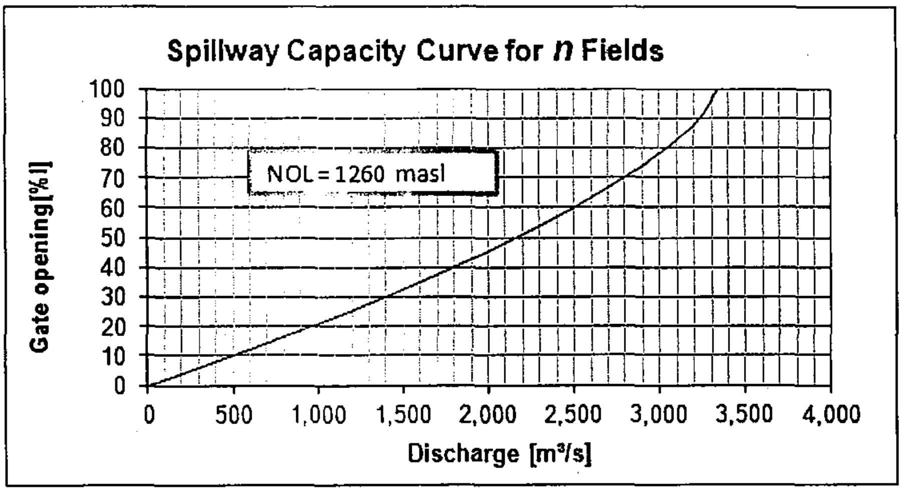


Figure 7-2: Spillway capacity curve

Due to the expected role in reservoir flushing, top layer of the spillway crest and of the chute shall be covered with abrasion resistant concrete.

Spillway design					
n=	3		number of fields		
bp=	3	m	pier width		
cq=	0.48		creager discharge coeff.		
Hus=	1260	masl	max water level		
Hsp=	1245	masl	spillway sill level		
H=	15	m			
B=	8.82	m	required net spillway field width		
Bfin=	9	m	adopted value		
Q=	2180	m³/s	discharge (n-1)		
Hcr=	6.05	m	critical depth on the spillway		
Acr=	163.37	m²	critical flow area		
Fr=	1.00		froude nr		
vcr=	13.34	m/s	critical velocity		
Piers effect					
vres=	4.40	m/s	velocity in the reservoir		
He=	15.99	m	energy level in the reservoir		
kp=	0.01		shape coeff. (USBR, 1987)		
kb=	0.1		contraction coeff		
Btot=	10.28	m	required field width (Creager 1961)		
Bfin=	10.5	m	adopted field width		
HQ1.000=	2223	m³/s	total spillway discharge (n-1) + freeboard	Q1000= 2180	m³/s
HQ10.000=	3335	m³/s	total spillway discharge (n) + freeboard	Q10.000= 3055	m³/s
PMF =	5040	m³/s	check flood n gates, no freeboard	Qpmf= 5040	m³/s

PMF water level = 1264.8 masl

max pressure during pmf:	19.75	m
spillway design pressure:	15	m
head ratio on spillway sill:	1.3	

constr. Width	unit flow
43.5	123.5

Figure 7-3: Spillway design

A conventional stilling basin is selected at the most suitable type for energy dissipation. A configuration with USBR stilling basin type IV with baffles is considered sensitive against abrasion, due to the high bedload content.

Hydraulic calculation for the Stilling Basin is provided in the following figure.

Stilling Basin Design

Q=	3055.0	m³/s	discharge
Etop=	1260	masl	MWL
Bbegin=	27	m	stilling basin active width at the beginning
Bend=	27	m	stilling basin active width at the end
n=	0.014		Manning's roughness coefficient
Ztw=	1237	masl	tailwater level for the design discharge
Zriverbed=	1226	masl	riverbed elevation

USBR	2		type of USBR stilling basin type
unit flow	113.1	m³/s/m'	
Fr=	4.84		Froude nr.
Lc=	6		stilling basin length coefficient

Spillway Crest Section

Zsp=	1245	masl	spillway sill elevation
Fr=	1.00		Froude nr
hcr=	10.93	m	critical depth
Vcr=	10.35	m/s	critical velocity

Stilling Basin - Upstream Section

Zbott=	1211.49	masl	s.b. bottom elev. (iterative)
h1=	3.821	m	first conjugated depth
V1=	29.611	m/s	velocity in the stilling basin
V1,corr=	29.61	m/s	corrected velocity (iterative)
Fr =	4.836		Froude nr in the basin

Stilling Basin - Downstream Section

hcr=	10.93	m	critical depth
h2=	24.29	m	second conjugated depth
V3=	4.66	m/s	velocity in this section
sigma=	1.05		submergence
Zbaz,d=	1211.49	masl	s.b. bottom elevation
Zbaz,d, ad	1211.00	masl	adopted bottom elevation
Bbaz=	27.00	m	stilling basin active width
Lbaz=	145.77	m	stilling basin length
dbaz=	15.00	m	excavation depth
freeboard	1.64	m	freeboard at stilling basin
H wall=	1237.00		

Figure 7-4: Stilling basin design

Stilling basin is envisioned as reinforced concrete structure. Depending on the rock conditions encountered during execution of construction works, the option to leave stilling basin in open cut, i.e. without concrete lining shall be investigated, as it offers certain to considerable cost saving potential.
The following figure shows the adopted design for the spillway and stilling basin.

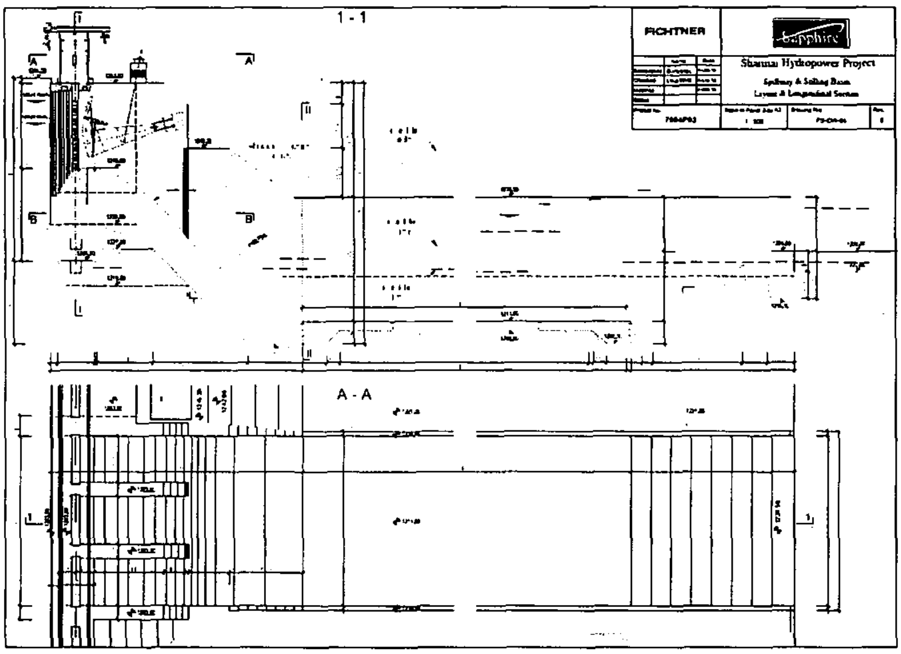


Figure 7-5: Spillway and stilling basin

Due to the magnitude of flows released, it is highly recommended to carry out physical model tests intended to confirm the proposed sizing.

7.2.4 Sediment Tank

Sediment tank block is located close to the left dam abutment. It consists of 3 bays, each designed for 50% flow, in order to allow continuous operation of the plant even during the periods with heavy sediment load. The design head at Sharmai HPP of close to 200m requires the particle size of d=0.2 mm to be removed from the flow entering power waterways.

Besides flow and particle size, design of sediment tanks considered fluctuations in the reservoir water level. Sediment tank chambers will be equipped with gates at both ends and a flushing gate. It is recommended to implement an automatic system for measuring the sediment level in the tanks and operating the gates.

The figure below illustrates the design of sediment tank.

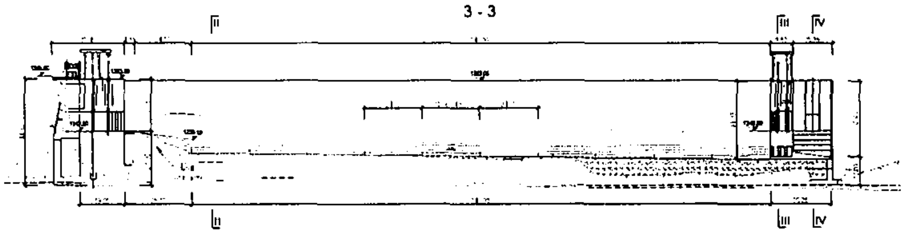
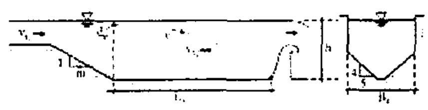


Figure 7-6: Desander section

The following figure shows the results of hydraulic design for sediment tanks.

FICHTNER Hydropower Department	Desander

System



Input data

Discharge	Q	45	[m³/s]
Characteristic grain size	d_{ch}	0.2	[mm]
Number of chambers	n	1	[-]

Calculated data

Discharge per chamber	Q_c	45.00	[m³/s]
Critical flow velocity for begin of movement of deposits (shields)	V_{cr}	0.20	[m/s]
Minimum area of each chamber	A	228.69	[m²]
Recommended width of chamber	b	13.53	[m]
Recommended depth	h	16.91	[m]
According to the settling velocity by Zanke 1982			
Settling velocity still water condition (Zanke)	Vs	0.028	[m/s]
Reduction factor for turbulent flow	α	0.032	[-]
Settling velocity in turbulent flow conditions	vs_turb	0.022	[m/s]
Water depth of basin	h	16.9	[m]
Total width of sandtrap	B_s	13.5	[m]
Required length of basin under still water conditions	$L_{s,still}$	146.3	[m]
Length of basin under consideration of turbulent flow	$L_{s,turb}$	181.3	[m]
According to the settling velocity by DVWK 1986			
Settling velocity still water condition (DVWK)	Vs	0.022	[m/s]
Reduction factor for turbulent flow	α	0.032	[-]
Settling velocity in turbulent flow conditions	vs_turb	0.015	[m/s]
Water depth of basin	h	16.9	[m]
Total width of sandtrap	B_s	13.5	[m]
Required length of basin under still water conditions	$L_{s,still}$	179.8	[m]
Length of basin under consideration of turbulent flow	$L_{s,turb}$	242.1	[m]

$v_{cr} = 0.44 \cdot \sqrt{d_{ch}}$

$\frac{h}{b} = 1.25$

$v_{s,turb} = v'_s - \alpha \cdot v_{cr}$

with $\alpha = 0.132/h^{1/2}$

$v_{s,turb} = v'_s - \alpha \cdot v_{cr}$

with $\alpha = 0.132/h^{1/2}$

Figure 7-7: Desander design

Due to relevance of a sediment tank to safe and reliable operation over the project's lifetime, it is strongly recommended to confirm the efficiency of the sediment tank by means of physical model tests. The tests can further conclude on the necessity to install intermediate gates for flushing of the chambers.

7.2.5 River Diversion

River diversion is envisioned by means of a diversion tunnel, placed in the right abutment. The design of the diversion works was governed by the magnitude of flow to be released during construction. As concluded in the design criteria chapter, the 10-year flood was considered as the flood relevant for dimensioning.

An optimization analysis in respect of diameter of the diversion tunnel vs. investment in cofferdams was performed; the results are shown in the figure below.

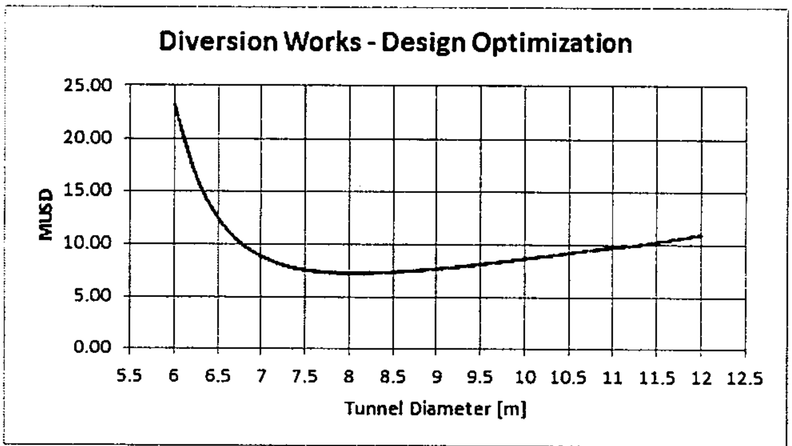


Figure 7-8: Diversion works - optimization chart

Table 7-1: River diversion works design

Summary data		
	Optimum Costs	7.2 Mill. EUR
	Design discharge	730 m³/s
	Number of tunnels	1
	Tunnel diameter	8 m
	Us water level	1244.49 m a.s.l.
	Us coffer dam - crest level	1245.49 m a.s.l.
	Height of US coffer dam	13.49 m
	Length of Us coffer dam	126.98 m
	Volume of Us coffer dam	0.04 Million m
	Ds water level	1230 m a.s.l.
	Ds coffer dam - crest level	1231 m a.s.l.
	Height of Ds coffer dam	9 m
	Length of Ds coffer dam	136 m
	Volume of Ds coffer dam	0.021 Million m

Diversion tunnel will be about 500 m long. After construction, it can be either plugged or converted to a sediment bypass.

Conversion of the diversion tunnel into a sediment bypass would imply somewhat higher costs involved in a gated structure (gate and stoplogs, access and platform/tower), which would allow the utilization of the diversion tunnel as a flushing facility. However and as per the results of the analysis performed in the said respect and presented in the Hydrological Report, the option to utilize the diversion tunnel as a flushing facility is not regarded efficient or sustainable, due to limited efficiency, abrasion and clogging.

The following figure shows the diversion tunnel design.

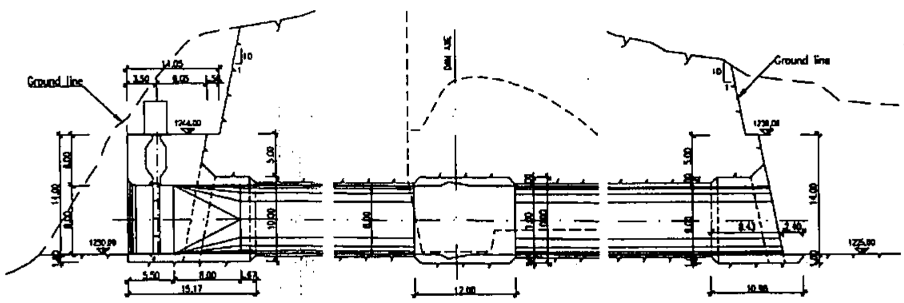


Figure 7-9: Diversion tunnel

It is envisioned as an unlined tunnel with the inner diameter of 8m.

7.2.6 Power tunnel

Power tunnel shall be 8.45 km long. It will be excavated by drill-and-blast technique, from both sides. Possibly and subject to the contractor's work plan, an intermediate adit at the tunnel chainage 7.800 m may need to be considered.

Optimization of the Power tunnel diameter was performed for the rated installed flow and with consideration to the lifetime costs and benefits. The results are presented in the following table and figure.

Table 7-2: Headrace tunnel optimization

		Energy Loss			Costs (30 year horizon)			
D inn,tun [m]	v, tun [m/s]	losses [m]	losses [%]	Energy [GWh/y]	C, civil [mio. USD]	C, losses [mio. USD/y]	C,total [mio. USD]	C,30y [mio. USD]
8	1.79	2.29	1.1%	7.90	91.85	0.55	92.40	163.56
7.75	1.91	2.72	1.4%	9.36	86.20	0.66	86.85	157.58
7.5	2.04	3.24	1.6%	11.15	80.73	0.78	81.51	152.58
7.25	2.18	3.88	1.9%	13.36	75.43	0.94	76.37	148.75
7	2.34	4.68	2.3%	16.11	70.32	1.13	71.45	146.35
6.75	**2.52**	**5.68**	**2.8%**	**19.56**	**65.39**	**1.37**	**66.76**	**145.70**
6.5	2.71	6.94	3.5%	23.92	60.63	1.67	62.31	147.25
6.25	2.93	8.56	4.3%	29.49	56.06	2.06	58.12	151.62
6	3.18	10.64	5.3%	36.66	51.66	2.57	54.23	159.65
5.75	3.47	13.35	6.7%	46.00	47.45	3.22	50.67	172.53
5.5	3.79	16.93	8.5%	58.31	43.41	4.08	47.49	191.91
5.25	4.16	21.69	10.8%	74.73	39.56	5.23	44.79	220.23

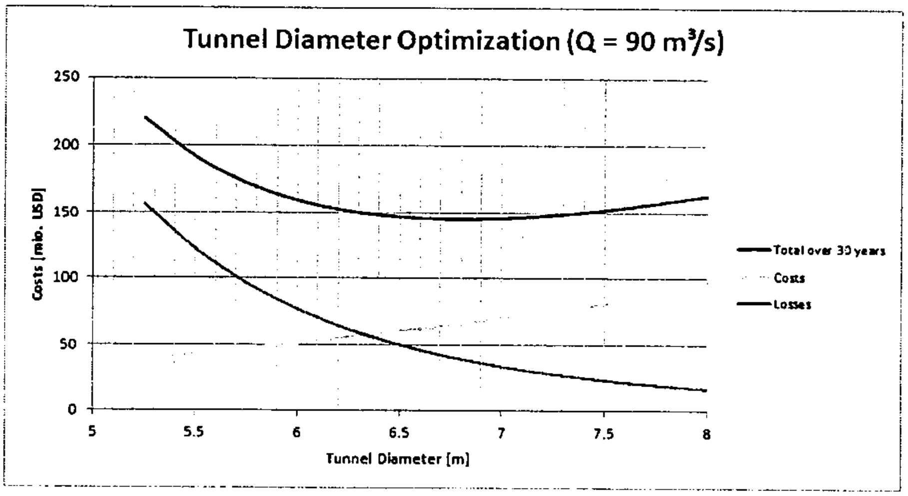


Figure 7-10: Headrace tunnel optimization chart

Power tunnel shall have the following layout, whereas the particular cross-section design details will depend on the rock class encountered, as discussed in the Geological Report.

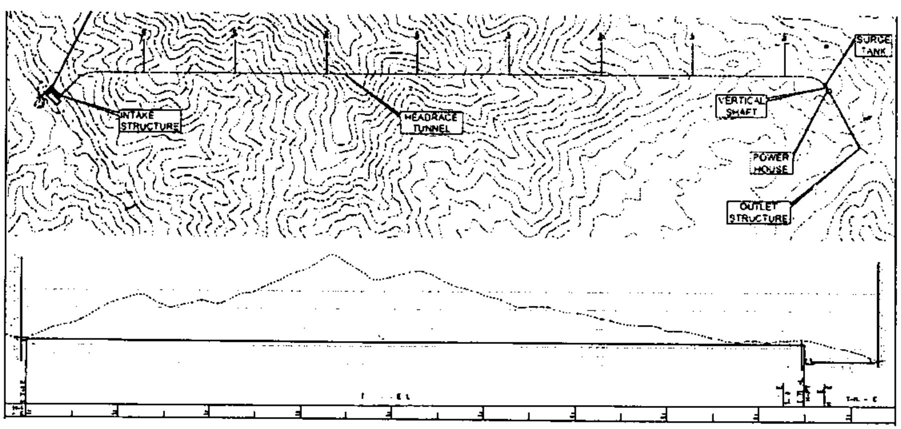


Figure 7-11: Headrace tunnel

The power inlet to the headrace tunnel was designed to assure avoidance of air intrusion in power waterways.

Gordon's formula

D =	6.75	m	tunnel diameter
A =	35.8	m	tunnel area
v =	2.52	m/s	velocity in transition section
Q =	90	m³/s	discharge
c =	0.72		coefficient for symmethric approach
s =	4.70	m	submergence
s_{ad} =	6	m	adopted submergence
MWL =	1260	masl	maximum water level in the res.
mWL =	1255	masl	minimum operating level in the res
h,int =	1242.25	masl	adopted intake inverted sill elevation

Figure 7-12: Air intrusion analysis

7.2.7 Surge tanks

Sharmai layout requires two surge tanks: upstream and downstream. The surge tanks shall accommodate for the pressure fluctuations during hydraulic transients and assure safe and reliable operation of the plant during sudden changes in the operating regime. The proposed design of the surge tanks is based on the performed hydraulic transient analysis, assuming sizing and performance characteristics of the main equipment. The costs are calculated accordingly.

The following figures illustrate the design of the surge tank.

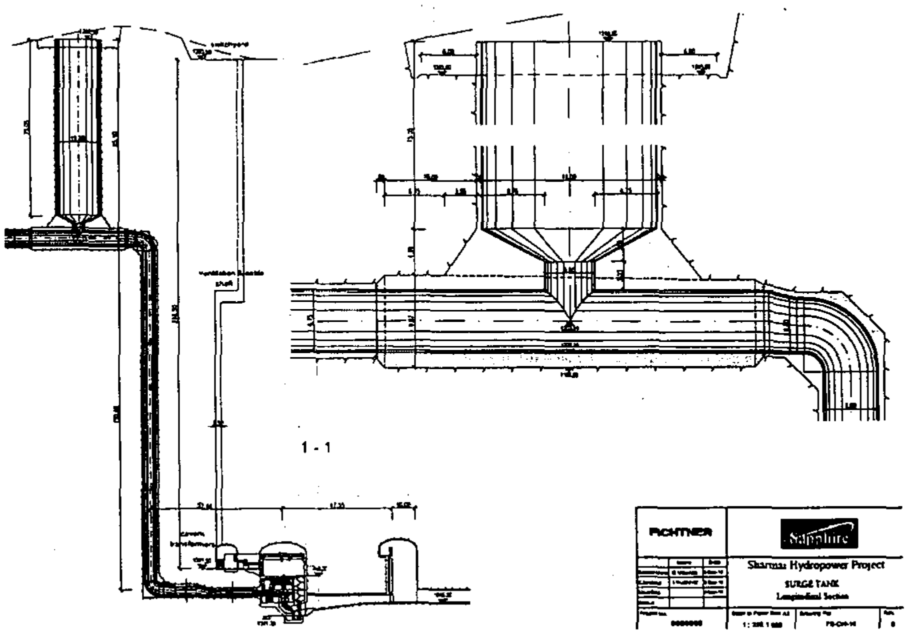


Figure 7-13: Surge tank design

7.2.8 Vertical shaft

Vertical shaft is envisioned as a circular vertical structure of 6.00 m inner diameter, lined with concrete throughout the length and with steel liner.

As hydro-fracturing tests were not performed, steel lining throughout the length of the vertical shaft is considered for the present level of design.

An optimization analysis with the estimated unit rates was performed in order to define the inner diameter of the shaft. The results are presented in the following graph.

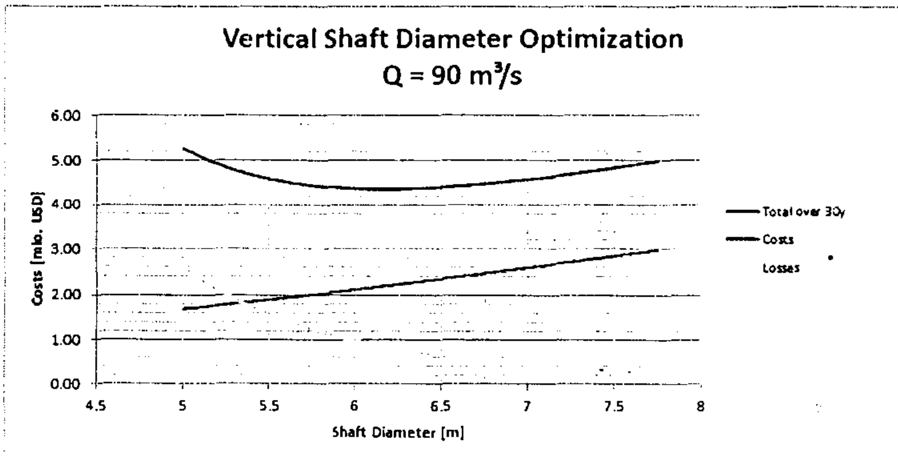


Figure 7-14: Optimization of the diameter of the vertical shaft

Table 7-3: Vertical shaft - diameter optimization

		Energy Loss				Costs (30 year horizon)		
D inn,tun [m]	v, tun [m/s]	losses [m]	losses [%]	Energy [GWh/y]	C, civil [mio. USD]	C, losses [mio. USD/y]	C,total [mio. USD]	C,30y [mio. USD]
7.75	1.91	0.03	0.0%	0.10	2.97	0.01	2.98	5.00
7.5	2.04	0.03	0.0%	0.12	2.84	0.01	2.85	4.84
7.25	2.18	0.04	0.0%	0.14	2.71	0.01	2.73	4.70
7	2.34	0.05	0.0%	0.17	2.59	0.01	2.60	4.57
6.75	2.52	0.06	0.0%	0.20	2.47	0.02	2.49	4.47
6.5	2.71	0.07	0.0%	0.25	2.35	0.02	2.37	4.40
6.25	2.93	0.09	0.0%	0.31	2.23	0.03	2.26	4.36
6	**3.18**	**0.11**	**0.1%**	**0.38**	**2.12**	**0.03**	**2.15**	**4.36**
5.75	3.47	0.13	0.1%	0.48	2.00	0.04	2.04	4.43
5.5	3.79	0.17	0.1%	0.61	1.89	0.05	1.94	4.58
5.25	4.16	0.22	0.1%	0.78	1.78	0.07	1.85	4.85
5	4.58	0.28	0.1%	1.01	1.68	0.09	1.76	5.27

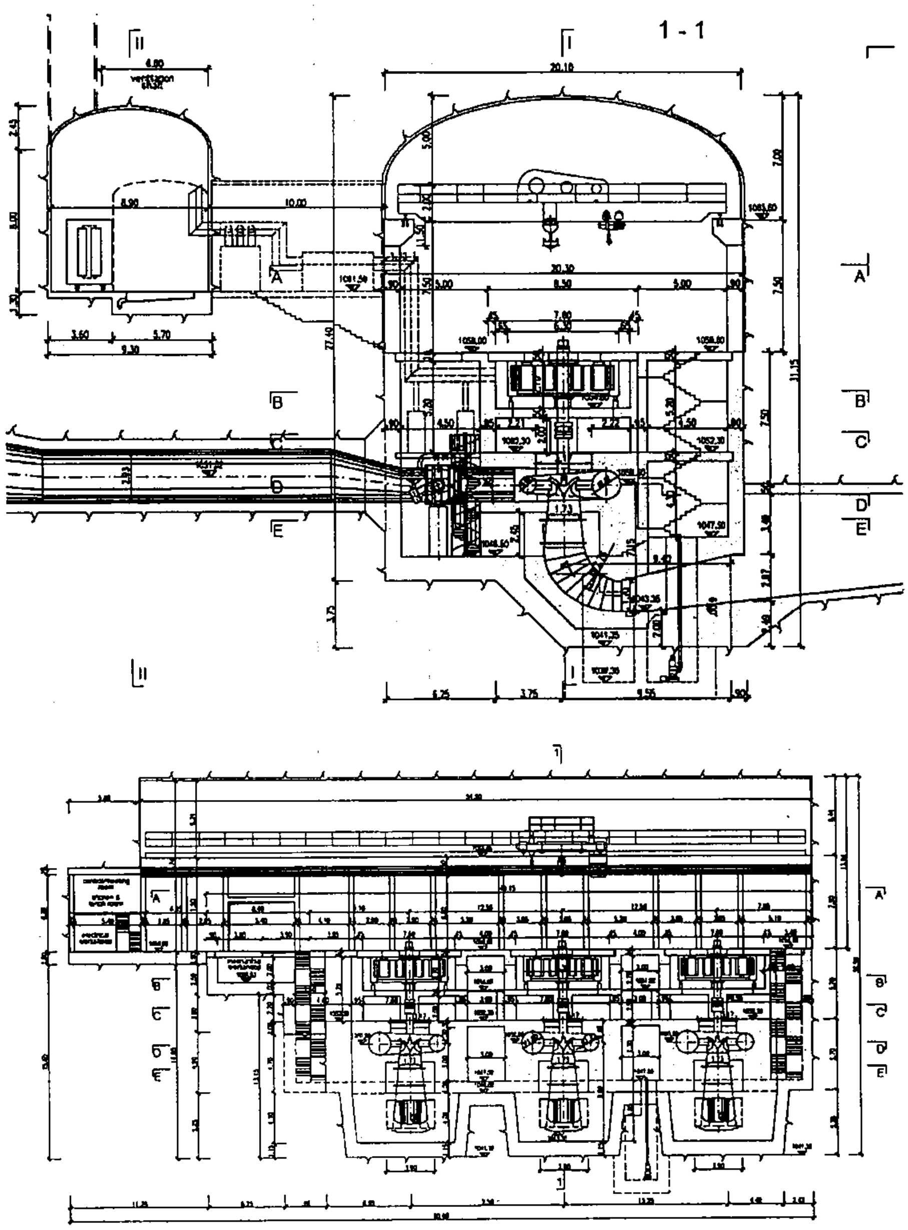
The shaft will end in a manifold to the powerhouse cavern. Size of the manifold and the distance to the other structures predominantly govern the position of the powerhouse cavern.

7.2.9 Powerhouse cavern

Powerhouse cavern shall accommodate 3 Francis vertical units. Main cavern shall have the following dimensions: W x H x L = 20 x 30 x 60 m, all subject to possible small scale adjustments related to the final equipment size. The cavern will have the assembling bay and the control rooms.

There will be a separate cavern for transformers. The caverns will be accessible by an access tunnel from the main N-45 road. Besides access tunnel, there will be separate cable and aeration tunnels, connecting the powerhouse cavern to the switchyard.

Key dimensions and the layout of the caverns and the appurtenant structures is given in the following figures.



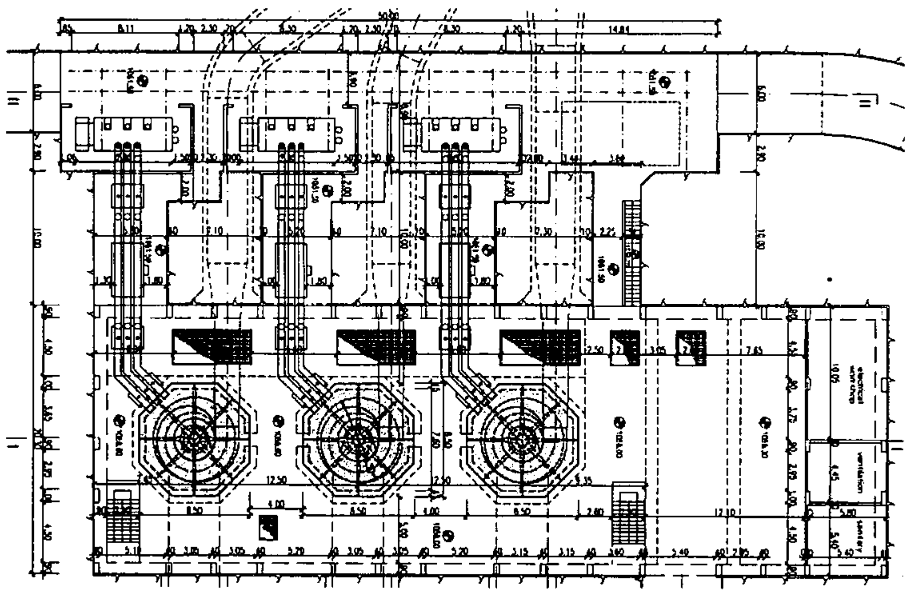


Figure 7-15: Powerhouse and transformer caverns

7.2.10 Outlet tunnel

Power outlet tunnel is designed as a pressure tunnel of the same diameter as the power tunnel. Outlet tunnel is 870 m long and assures pressure flow under all tailwater conditions at the outlet to Panjkora river. The following figure shows the layout and the longitudinal section of the outlet tunnel.

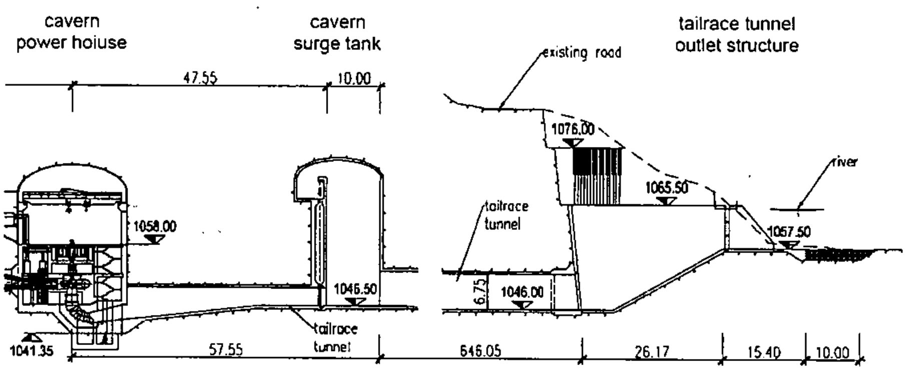


Figure 7-16: Tailrace tunnel

7.2.11 Switchyard

Switchyard is designed as an open air structure, located at a plateau above the powerhouse cavern, close to the surge tank. This option is considered as a cost-effective solution.

Should the size of the switchyard area become critical in terms of ESIA-related impacts (which is not indicated by the results of the ESIA study), the switchyard concept can be changed to SF6. As such, its dimensions could be reduced, since the space requirements of a SF6 switchyard arrangement are lower compared to an air insulated outdoor arrangement.

An SF6 switchgear could then either be located in the extension of the transformer cavern or at the location envisioned for the outdoor module.

There is an access road, connecting the switchyard with the existing road N-45. The following figure illustrates the switchyard plateau.

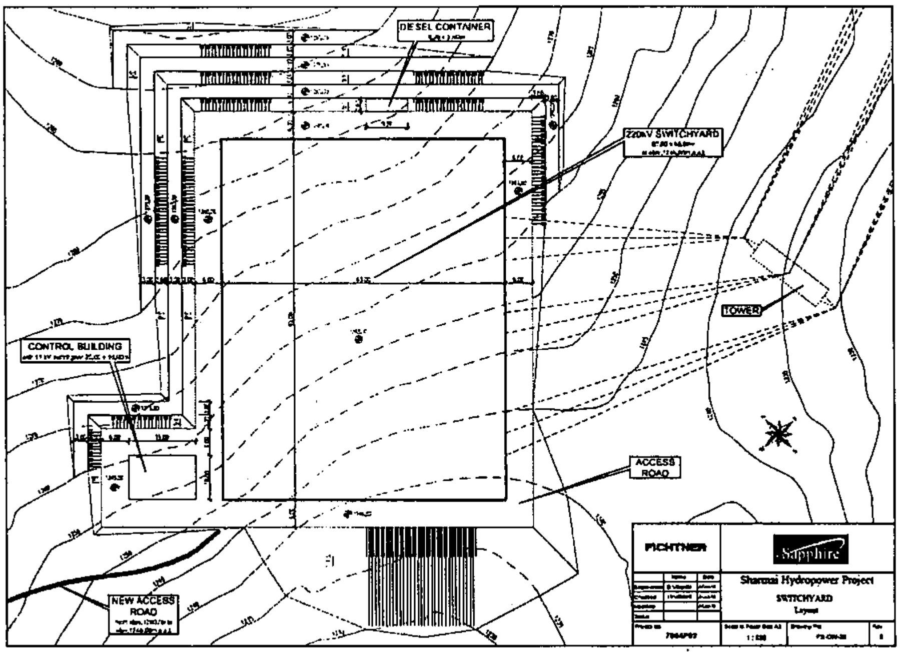


Figure 7-17: Switchyard

7.2.12 Roads

The existing roads may need to be locally improved for the required heavy duty transport during construction and installation of equipment. Where required, new temporary and permanent roads would need to be constructed. The new roads will be connected to the existing road infrastructure. The construction works on the roads can be performed without affecting the traffic on the N-45 road.

The new roads will be paved (permanent roads) or unpaved (temporary and site roads). There will be about 2.5 km of the new roads required and about 6 km of the roads would need to be upgraded. The following figure illustrates the envisioned design of the roads.

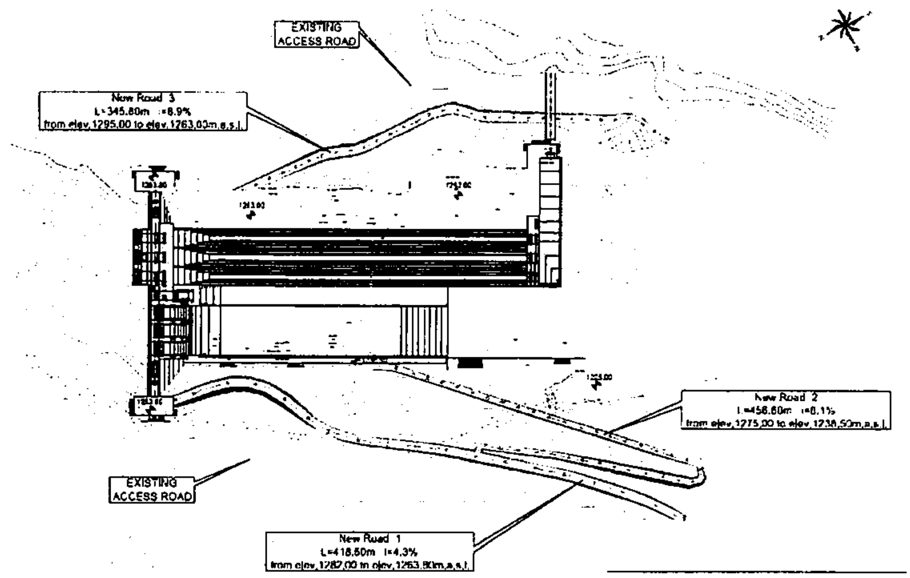


Figure 7-18: Roads - headworks

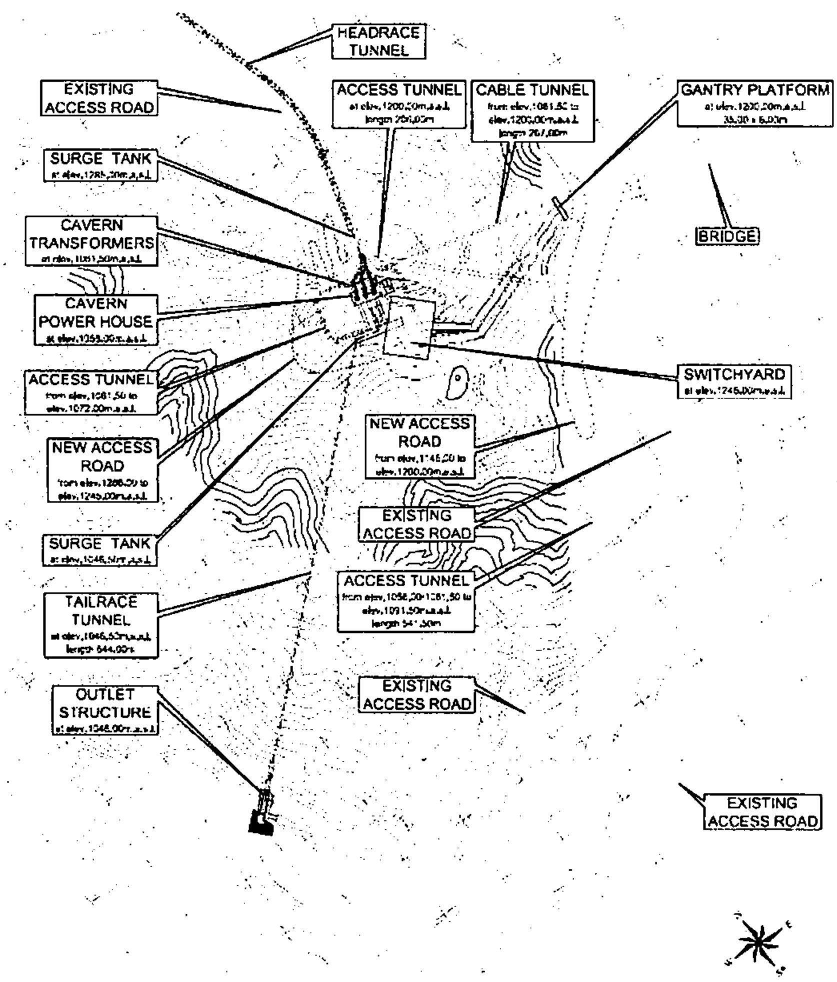


Figure 7-19: Roads - powerhouse area

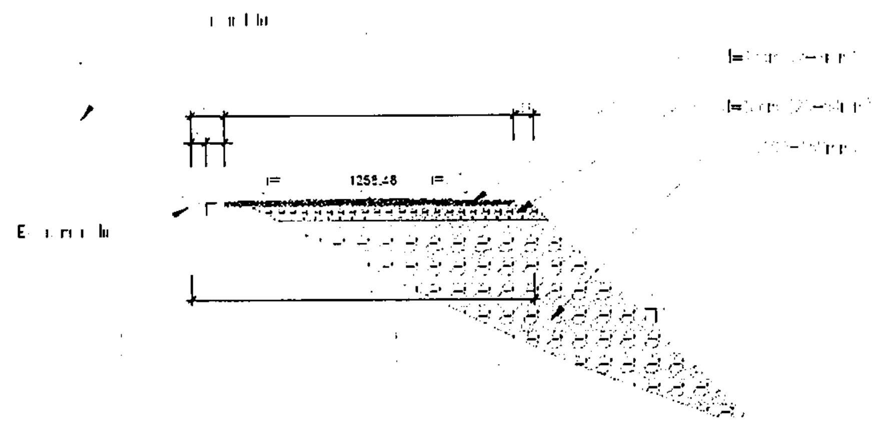


Figure 7-20: Roads - typical design in fill

7.2.13 Temporary structures

Temporary structures, such as construction camps, batching plants, quarry and landfill areas are tentatively positioned; their locations are concluded acceptable in terms of ESIA and are located in vicinity of the existing roads.

Due to the remoteness between the headworks from other sites, the proposed layout envisions two separate batching plants and landfills.

It is finally the contractor's responsibility to adopt or modify the proposed concept. The locations for the temporary structures are given in the following figure.

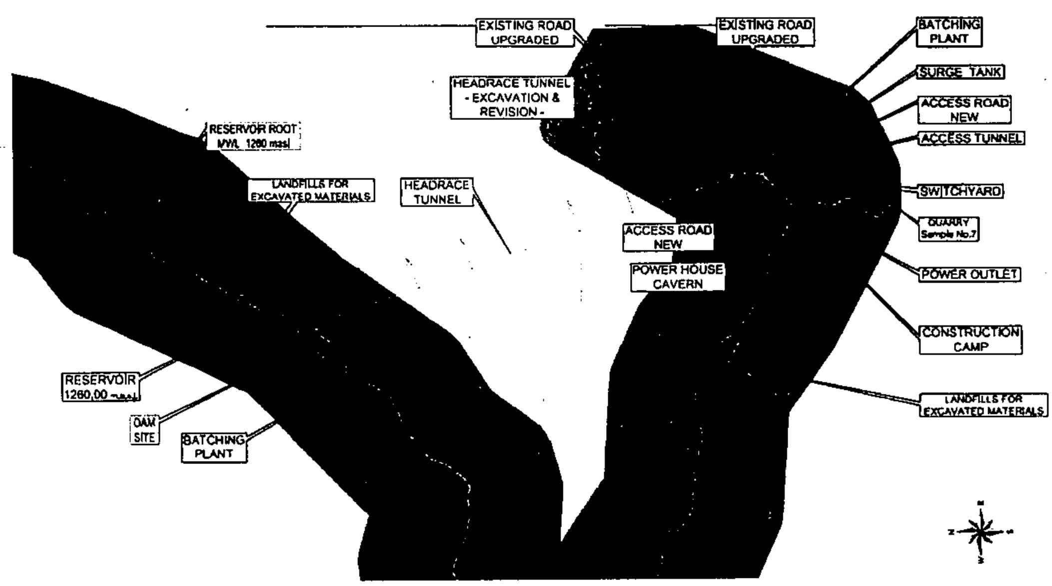


Figure 7-21: Temporary structures

The costs of all temporary site facilities are deemed covered by the Mobilization item in BoQ. The price is in function of the contractor's method statement, equipment and manpower, which may largely vary from one contractor to another.

7.3 Design of Mechanical Equipment

7.3.1 General

The mechanical equipment and main mechanical auxiliaries within the powerplant consist of following major items:

- three vertical shaft Francis type turbines including hydraulic/electronic turbine governors
- main inlet valve in front of each turbine with auxiliaries
- auxiliary mechanical systems such as
  - cooling water system
  - drainage and dewatering system
  - heating, ventilation and air conditioning system
  - oil treatment unit
  - compressed air system
  - mechanical workshop
  - powerhouse overhead travelling crane
  - fire fighting and detection system.

7.3.2 Design criteria

7.3.2.1 Hydrological and hydraulic conditions

7.3.2.1.1 Design discharge

Based on performed optimization analyses a plant design discharge of 90 m^3/s has been determined.

7.3.2.1.2 Number and type of generating units

The choice for the number of generating units is driven by the following considerations:

- relation between the unit capacity and the capacity of the grid
- anticipated load range of regulation between minimum and maximum load / discharge
- capacity loss during maintenance and unplanned unit outages
- maintenance, spare parts, standardization of components
- transportation limits (size and weight of components)
- cost of the electromechanical equipment, the power waterways and the related civil structures.

Considering the above, a three unit arrangement is considered to be the most appropriate and technically and economically optimized solution for a plant discharge of 90 m³/s.

7.3.2.2 Power plant characteristics

7.3.2.2.1 Water levels

The water levels shown below summarize the key hydraulic data of the power scheme used for the layout and dimensioning of the equipment.

Reservoir

• Maximum Flood Level (MFL)	el 1263.8 masl
• Maximum Operating Level (MOL)	el 1260.0 masl
• Minimum Draw Down Level (MDDL)	el 1255.0 masl

Tailwater

• Normal Level (3 units running at rated load)	el 1059.2 masl
• Minimum Level (1 unit running at rated load)	el 1058.9 masl
• Minimum Operating Level (1 unit running at 40% of rated load)	el 1058.8 masl

7.3.2.2.2 Gross and net heads

The following values are used for the design, layout and specification of the turbines:

Gross heads

• Maximum gross head at MOL, one unit in operation at 40% rated load	201.2 m
• Maximum gross head at MOL, one unit in operation	201.1 m
• Maximum gross head at MOL, three units in operation	200.8 m
• Minimum gross head at MDDL, three units in operation	195.8 m

The waterways of Sharmai HPP basically consist of intake, sand trap, headrace tunnel, headrace surge tank, pressure shaft, manifold, draft tube, tailrace surge tank, tailrace tunnel.

The results of the loss calculations led to the following values for head losses at different discharges:

Head losses

• One unit in operation at 40% of rated discharge	2.2 m
• One unit in operation at rated discharge	3.9 m

- Three units in operation at rated discharge 7.9 m

These results have been used for the determination of the net heads as follows.

Net heads

- Normal maximum net head with one unit operating at MOL and 40% of rated discharge 199.0 m
- Normal maximum net head with one unit operating at MOL 197.2 m
- Normal maximum net head with three units operating at MOL 192.9 m
- Minimum net head with three units operating at MDDL 187.9 m

The rated net head for the hydraulic layout of the turbines has been set at 192.9 m. The rated net head is derived from the consideration of a three unit operation at the normal headwater level of 1260 m a.s.l.

All main hydraulic data is summarized in the following table:

Table 7-4: Main hydraulic data of turbine design

Characteristics	Unit	
MFL (Maximum Flood Level)	m a.s.l.	1263.8
MOL (Maximum Operating Level)	m a.s.l.	1260.0
MDDL (Minimum Draw Down Level)	m a.s.l.	1255.0
TWL_{min} (Tailwater Level Minimum at 40% of rated discharge)	m a.s.l.	1058.8
TWL_{1U} (Tailwater Level at single unit operation)	m a.s.l.	1058.9
TWL_{3U} (Tailwater Level at three unit operation)	m a.s.l.	1059.2
H_{gross} (normal range)	m	195.8 - 201.2
$h_{l,min}$ (head losses one unit operating at 40% of rated discharge)	m	2.2
$h_{l,1U}$ (head losses one unit operation)	m	3.9
$h_{l,3U}$ (head losses three units operation)	m	7.9
$H_{net,min1}$ (at MOL and 40% of rated discharge)	m	199.0
$H_{net,max}$ (at MOL and single unit operation)	m	197.2
$H_{net,rated}$ (at MOL and three unit operation)	m	192.9
$H_{net,min2}$ (at MDDL and three unit operation)	m	187.9
Q_{plant}	m³/s	90
Q_{rated} (maximum design discharge per unit)	m³/s	30

7.3.3 Hydraulic design of power waterways - transient analysis

The layout of Sharmai HPP, implying the existence of relatively long pressurized power waterways was analyzed in respect to the envisioned peaking role of the plant in Pakistan's system, which requires safe and prompt response of the units to the instantaneous changes in the operating regime (e.g. standstill to full power, sudden shutdown or load ramping).

7.3.3.1 Waterway acceleration times (Surge Tank criteria)

The effective length of hydraulic waterways being relevant for hydraulic transient operation is the distance from the free surface water level upstream of the turbine to the nearest downstream free water surface, which is usually the river.

For a project layout without surge tanks the effective length is the distance between the power intake and the end of the tailrace tunnel, whereas for power plant schemes with a headrace and tailrace surge tank, the effective length is derived from the distance between the two surge tanks.

The acceleration time of water within the relevant water conduit can be used as a first indication whether a surge tank is required or not for stable operation of the plant. The acceleration time should not exceed a value of 2.5 seconds. If the acceleration time exceeds 2.5 seconds, the requirement for a surge tank is rather high and needs to be investigated in further detail.

The acceleration time is determined as follows:

$$t_s = {\Sigma(L_i \cdot v_i)}\Big/{(H \cdot g)}$$

where

t_s = acceleration time (s)
L_i = length of individual conduit reach (m)
v_i = flow velocity in the individual conduit reach (m/s)
H = head (m)
g = gravity acceleration (m/s^2)

Applying this formula to Sharmai HPP results in the acceleration times presented in the tables below (without and with surge tanks).

Table 7-5: Characteristics of Sharmai waterway without surge tanks

Waterway Section	Length L [m]	Velocity v [m/s]	L x v [m²/s]	Head H [m]	L x v / g x H [s]
Headrace Tunnel	8500	2,5	21250,0	195,8	11,1
Pressure Shaft	200	3,2	640,0	195,8	0,3
Pressure Tunnel	870	2,5	2175,0	195,8	1,1
Water Acceleration without surge tanks					12,5

Table 7-6: Characteristics of Sharmai waterway with headrace surge tank

Waterway Section	Length L [m]	Velocity v [m/s]	L x v [m²/s]	Head H [m]	L x v / g x H [s]
Pressure Shaft	200	3,2	640,0	195,9	0,3
Pressure Tunnel	870	2,5	2175,0	195,8	1,1
Water Acceleration Time with Headrace Surge Tank					1,5

Table 7-7: Characteristics of Sharmai waterway with headrace and tailrace surge tank

Waterway Section	Length L [m]	Velocity v [m/s]	L x v [m²/s]	Head H [m]	L x v / g x H [s]
Pressure Shaft	200	3,2	640,0	195,8	0,3
Water Acceleration Time with Headrace and Tailrace Surge Tank					0,3

The hydraulic starting time without a surge tank would be 12.5 s. When having a headrace surge tank installed close to the downstream end of the headrace tunnel, it reaches a value of 1.5 s being already below the requirement of 2.5 s. Even with the headrace surge tank, the general criteria for operability of the plant is fulfilled, preliminary transient calculation revealed that a tailrace surge tank is required to avoid harmful negative pressures in the long tailrace tunnel during shutdowns. Therefore, the tailrace surge tank is not required due to stability reasons but due to the transient behavior of the system and resulting pressures.

With a tailrace surge tank, the water acceleration time reaches very low values of less than 1 s.

It shall be noted that the applied head of 195. 8 m reflects the minimum possible gross head. This is the worst case operation with regards to calculation of the acceleration times.

7.3.3.2 Type of surge tanks

For head race surge tank, a restricted orifice type surge shaft was selected with a diameter of 19.0 m in order to keep the water level fluctuations in the surge tank as well as the pressure values at the turbine inlet within acceptable limits. The orifice at the surge tank inlet acts as a throttle to damp pressure fluctuations and is arranged in accordance with common design practice.

With regards to the comparably high flow velocities at the throttle, this section shall be steel lined to avoid damages to the concrete structure.

The tailrace surge tank will be designed as an unthrottled surge chamber. the three tailrace tunnels leading from the generating units, will be connected directly to the surge chamber from which the tailrace tunnel starts.

7.3.3.3 Stability criterion

In accordance with commonly accepted international design practice, the cross-sectional area of the surge tank has to be checked by the Thoma-Criterion in order to ensure adequate stability of plant operation. Based on this criterion a cross sectional area is being calculated, with which a stable operation of the system is possible.

The Thoma-Criterion calculates the critical surge tank cross-section as follows:

$$A_{Thoma} = \frac{L * A_t * v_0^2}{2 * g * h_0 * (H_{g,0} - h_0)}$$

Where:

A_{Thoma}	=	Cross section area surge tank according to Thoma (m)
L	=	Length of tunnel (m)
A_t	=	Cross section area tunnel (m²)
v_0^2	=	Stationary velocity in tunnel (m)
g	=	Gravitational acceleration (m²/s)
h_0	=	Losses in tunnel (m)
$H_{g,0}$	=	Minimum gross head at stationary conditions (m)

In order to provide a sufficient safety margin, the actual cross-sectional area of the surge tank shall be minimum 1.5 times larger than the cross-sectional area according to Thoma.

The stability calculations for the headrace surge tank in accordance with Thoma are summarized in the table below.

Table 7-8: Calculation results for Thoma stability criteria

Lenght of tunnel	Lt	m	8500
Velocity in tunnel	vt	m/s	2,52
Diameter of tunnel	Dt	m	6,75
Cross-section area tunnel	At	m2	35,78
Head loss Tunnel	ht	m	3,90
Min head water level	WLHmin	m a.s.l.	1255
Max tailwater level	WLTmax	m a.s.l.	1059
Gross head	hg	m	196
Athoma	Ath	m2	131
Dthoma	Dth	m	12,92
Dselected	DSt	m	19,00
Aproposed	Ast	m2	283,53
Safety factor	[illegible]	[illegible]	2,16

As shown in the table above, with a surge tank diameter of 19.0 m, the safety factor with regards to the Thoma stability criterion is 2.16 and therefore adequate.

The size of the tailrace surge chamber was determined by a preliminary transient analysis. After determination of the required size, a theoretical analysis of potential critical interaction and unstable operating condition was carried out.

7.3.3.4 Transient calculation results

7.3.3.4.1 Main parameters

For all simulated load cases the following main parameters were used:

Table 7-9: Transient analysis - main input parameters

Headrace Surge Tank	
• Diameter	19.0 m
• Crest level	1291.0 masl
• Bottom level	1207.0 masl
Turbine	
• Type	vertical Francis
• Turbine centerline	1050.5 masl
• Number of units per waterway	3
• Nominal speed	428.6 rpm
• Moment of inertia GD² (generator)	675 tm²
• Rated plant discharge	90 m³/s
• Opening time (0-100% load) for one unit	25 s
• Closing time (100-0% load) for one unit	8 s
Tailrace Surge Tank	
• Cross-section	700 m²
• Crest level	1072.0 masl
• Bottom level	1052.0 masl

7.3.3.4.2 Load cases

For the design of the Sharmai headrace and tailrace surge tanks, different load cases were simulated to determine the maximum up- and downsurge in the surge tank.

Table 7-10: Transient analysis - Summary Load Cases

#	Load Case	HWL	TWL	Gross Head
		[masl]	*[masl]*	*[m]*
1	Sequential load acceptance (start-up) of 3 units @ maximum head (maximum reservoir water level and 3 units in operation)	1260.0	1059.1	200.9
2	Simultaneous load rejection (emergency closing) of 3 units @ maximum head (maximum reservoir water level and 3 units in operation)	1260.0	1059.1	200.9
3	Sequential load acceptance (start-up) of 3 units @ minimum head (minimum reservoir water level	1255.0	1060.5	194.5

#	Load Case	HWL [masl]	TWL [masl]	Gross Head [m]
	and 3 units in operation, including tailwater impact)			
4	Simultaneous load rejection (emergency closing) of 3 units @ minimum head (minimum reservoir water level and 3 units in operation, including tailwater impact)	1255.0	1060.5	194.5
5	Sequential load acceptance (start-up) of 3 units @ maximum head (maximum reservoir water level and 3 units in operation), subsequent simultaneous load rejection (emergency closing) of all 3 units at max. tunnel velocity	1260.0	1059.1	200.9
6	Sequential load acceptance (start-up) of 3 units @ minimum head (minimum reservoir water level and 3 units in operation, including tailwater impact), subsequent simultaneous load rejection (emergency closing) of all 3 units at max. tunnel velocity	1255.0	1060.5	194.5

It shall be noted that for load case #3 and #4 the used tailwater level (TWL) is 1060.5 masl. The crest level of the tailrace surge tank is defined by the water level during unit start-ups. Since there is no unit discharge before start-up, for this load cases, the mean annual maximum daily flow was considered in the river (correlates to 1060.5 masl).

7.3.3.4.2.1 Headrace surge tank - maximum upsurge

To determine the maximum upsurge and the related maximum water level in the headrace surge tank, load rejections at the maximum gross head with 3 units in operation are simulated. While load case #2 considers the load rejection of three units, load case #5 simulates the upsurge in the surge tank during a sudden load rejection of all three units after a sequential start up of all three units, at a time when the water velocity in the headrace tunnel is at its maximum.

The results of the transient calculations of the headrace surge tank of load case #2 and #5 are presented in the figures hereunder.

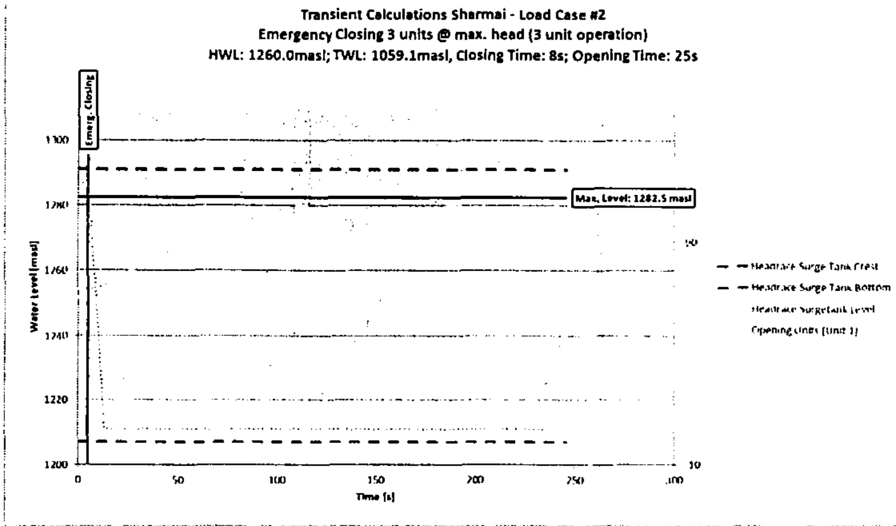


Figure 7-22: Headrace Surge Tank Load Case #2 - Shut down of 3 unit @ max. head

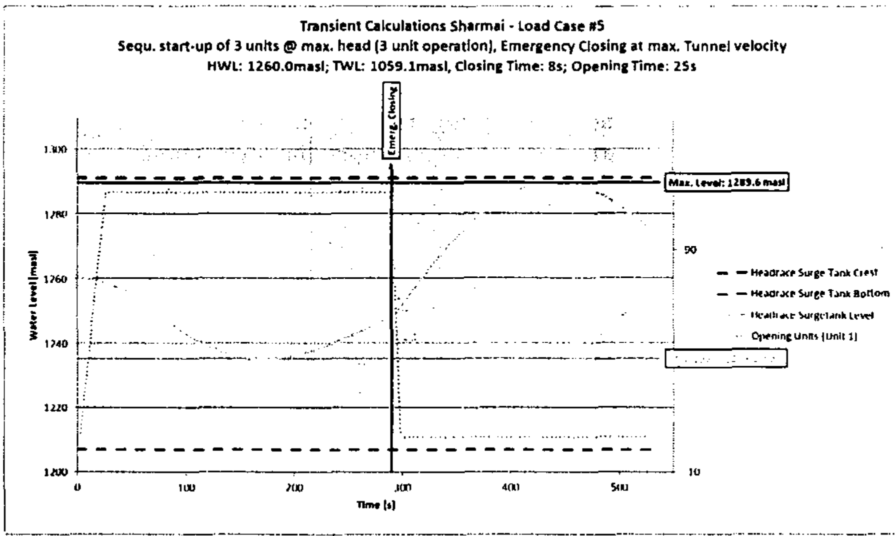


Figure 7-23: Headrace Surge Tank Load Case #5 - Start-up of 3 units @ max. head and subsequent load rejection of 3 units @ max. headrace tunnel velocity

The maximum water level in the surge tank after a simultaneous load rejection of all three units (load case #2) is 1282.5 m a.s.l. what corresponds to an upsurge of 22.5 m. Under consideration of the surge tank crest elevation of 1291.0 m a.s.l. an adequate freeboard of 8.5 m is remaining.

The maximum water level in the surge tank for load case #5 at the maximum reservoir water level is 1289.6 m a.s.l. what corresponds to an upsurge of 29.6 m. Under consideration of the surge tank crest elevation of 1291.0 m a.s.l. an adequate freeboard of 1.4 m is remaining. This is the highest upsurge of the considered load cases.

7.3.3.4.2.2 Headrace surge tank - maximum downsurge

To determine the maximum downsurge and the related minimum water level in the headrace surge tank, subsequent load acceptance (start-up) of the three generating units is simulated at minimum gross head in load case #3. In load case #6, a simultaneous load rejection follows the subsequent load acceptance and maximum tunnel velocity, what has however no impact on the minimum headrace surge tank water level.

The results of the transient calculations of the headrace surge tank of load case #3 are presented in the figures hereunder.

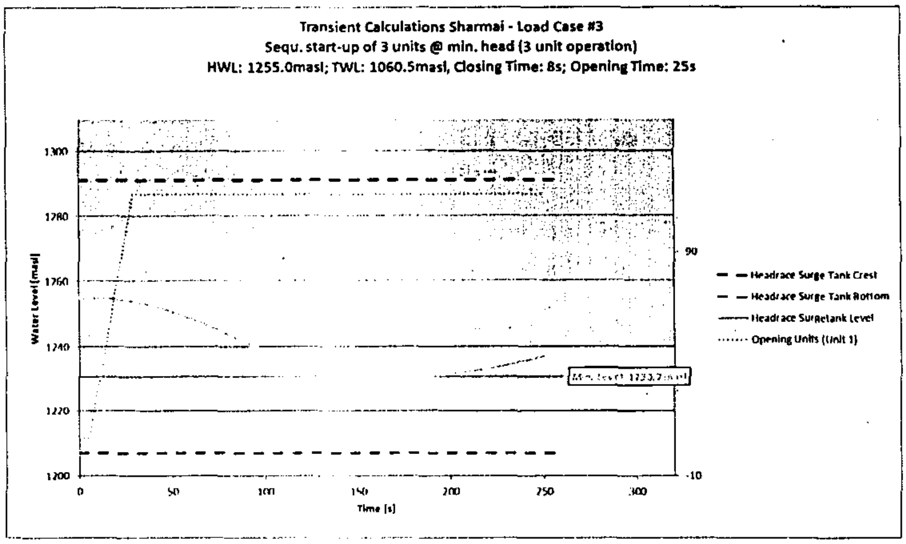


Figure 7-24: Headrace Surge Tank Load Case #3 - Start-up of 3 units @ min. head (3 unit operation with tailwater impact)

The minimum water level in the surge tank after a sequential load acceptance of all three units (load case #3 and #6) is 1230.7 m a.s.l. what corresponds to a downsurge of 24.3m. Under consideration of the surge tank bottom elevation of 1207.0 m a.s.l. an adequate freeboard of 23.7 m is remaining. This is the lowest downsurge at the considered load cases.

7.3.3.4.2.3 Tailrace surge tank - maximum upsurge

To determine the maximum upsurge and the related maximum water level in the tailrace surge tank, subsequent load acceptance (start-up) of the three generating units is simulated at minimum gross head in load case #3.

The results of the transient calculations of the tailrace surge tank of load case #3 are presented in the figures hereunder.

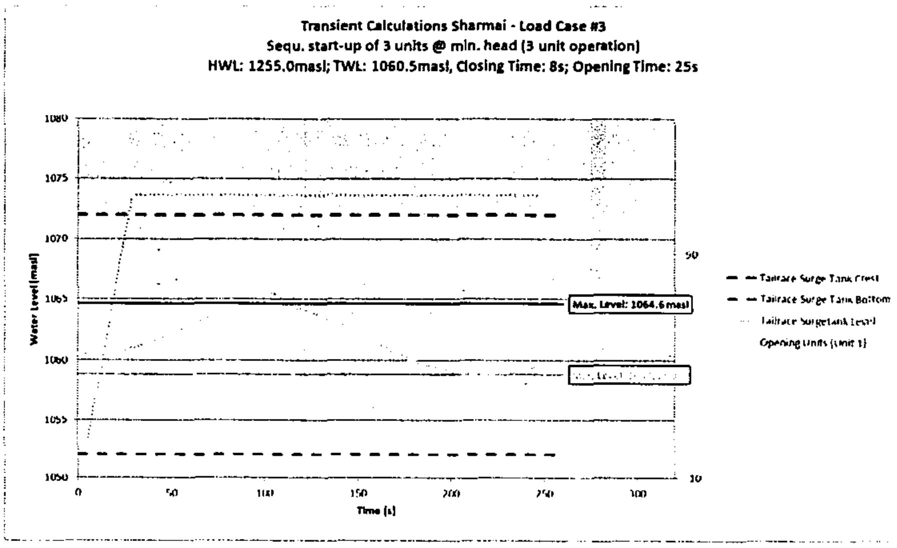


Figure 7-25: Tailrace Surge Tank Load Case #3 - Start-up of 3 units @ min. head (3 unit operation with tailwater impact)

The maximum water level in the tailrace surge tank after a sequential load acceptance of all three units (load case #3 and #6) is 1064.6 m a.s.l. what corresponds to an upsurge of 4.1 m. Under consideration of the tailrace surge tank crest elevation of 1072.0 m a.s.l. an adequate freeboard of 7.4 m is remaining.

This is the highest upsurge at the considered load cases.

7.3.3.4.2.4 Tailrace surge tank - maximum downsurge

To determine the maximum downsurge and the related minimum water level in the tailrace surge tank, load rejections at the maximum gross head with 3 units in operation are simulated.

The results of the transient calculations of tailrace surge tank of load case #2 are presented in the figures hereunder.

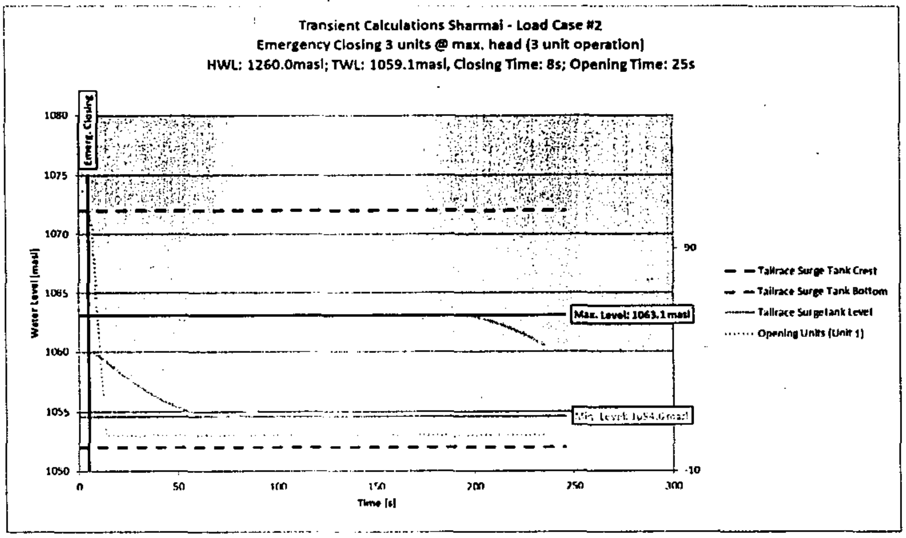


Figure 7-26: Tailrace Surge Tank Load Case #2 - Shut down of 3 unit @ max. head

The minimum water level in the surge tank after a simultaneous load rejection of all three units (load case #2) is 1054.6 m a.s.l. what corresponds to a downsurge of 4.5 m. Under consideration of the tailrace surge tank bottom elevation of 1052.0 m a.s.l. an adequate freeboard of 2.6 m is remaining.

This is the lowest downsurge at the considered load cases.

7.3.3.4.3 Water hammer analysis

For the water hammer analysis and the related maximum pressure rise at the turbine inlet and the maximum rise of the turbine speed, the same load cases as for the maximum up- and downsurge in the surge tanks are considered. The speed rise was calculated for a moment of inertia for the generating equipment GD^2 of 675 tm^2 and a closing time of 8 s was considered. The opening time is 25 s from no-load to full-load.

The relevant load case with regards to maximum pressure rise and turbine speed rise is load case #2 (and #5). The results of the calculations of this load case are presented below.

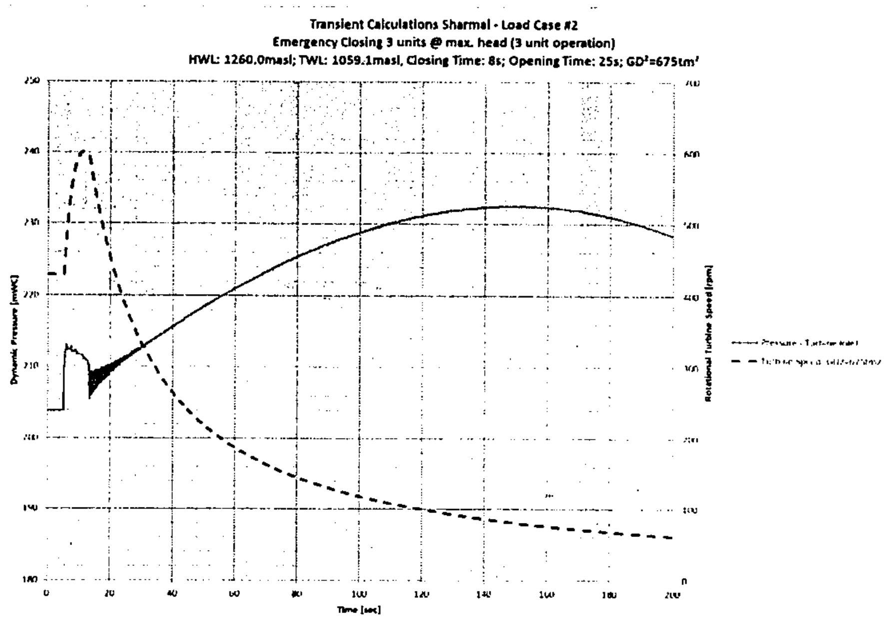


Figure 7-27: Load Case #2: Water hammer analysis

The maximum dynamic pressure at the turbine inlet level is 232.5 mWC which corresponds to 11% pressure rise over the maximum static head at the distributor center line of 209.5 mWC.

The maximum speed after load rejection (closing time Tc = 8 s) with a machine having a total rotating inertia corresponding to $GD^2 = 675$ tm^2 amounts to 600 rpm equivalent to 40 % speed rise over the synchronous speed of 428.6 rpm.

Both the speed rise of 40% and the pressure rise of 11% during the considered load cases are within acceptable and reasonable ranges for Francis turbines (speed rise: 45%, pressure rise: 50%).

7.3.3.5 Summary and conclusions

For this preliminary and general transient analysis, basic load cases were considered. A detailed analysis with regards to stability of the system, max. pressure and speed rise values, maximum and minimum levels in the surge tanks are to be done by the future contractor during a detailed design stage under consideration of the actual operating conditions, generating unit parameters, etc. Such a detailed analysis shall further include additional load cases with various operations of the plane (sequenced shut downs and restarts of different machines, etc.).

The main results of the transient analysis of the load cases considered in this study are summarized in the table hereunder.

Table 7-11: Summary transient calculation results

#	HWL	TWL	Gross Head	Starting Time	Closing Time	GD²	Headrace Surge Tank				Tailrace Surge Tank				Max. Turbine Speed	Max. Speed Rise	Max. Pressure Turbine	Max. Pressure Rise Turbine
							Minimum ST Water Level	Lower Freeboard	Maximum ST Water Level	Upper Freeboard	Minimum ST Water Level	Lower Freeboard	Maximum ST Water Level	Upper Freeboard				
	[masl]	[masl]	[m]	s	s	tm²	masl	m	masl	m	masl	m	masl	m	rpm	%	m	%
1	1260	1059,1	200,9	25	8	675	1235,7	78,2	-	-	1057,5	5,5	1063,4	8,6	-	-	-	-
2	1260	1059,1	200,9	25	8	675	-	-	1282,5	8,5	1054,6	2,6	1063,1	8,9	600	40%	233	11%
3	1255	1060,5	194,5	25	8	675	1230,7	23,7	-	-	1057,5	5,5	1064,6	7,4	-	-	-	-
4	1255	1060,5	194,5	25	8	675	1277,1	70,1	-	-	1056,0	4,0	1064,2	7,8	592	38%	217	3%
5	1260	1059,1	200,9	25	8	675	1235,7	78,2	1289,6	1,4	1056,0	4,0	1063,4	8,6	600	40%	233	11%
6	1255	1060,5	194,5	25	8	675	1230,7	23,7	1284,5	6,5	1057,5	5,5	1064,6	7,4	592	38%	217	3%

Those results were used for the feasibility design of the related structures and a design without special measures and restrictions in the plants operation (fail-safe design) was considered.

The performed transient analyses confirm the technical feasibility of the design adopted for the civil structures involved in the power waterways in terms of the envisioned layout and role of the main generating equipment at Sharmai HPP.

7.3.4 Turbine layout and design

7.3.4.1 Turbine discharge and power

The rated discharge of the turbine at rated net head of 192.9 m is 30.0 m³/s. The corresponding turbine power output is:

$$
\begin{aligned}
P_{T,rated} &= \eta_T \times Q_r \times \rho \times g \times H_{net,rated} \\
&= 0.915 \times 30.0 \frac{m^3}{s} \times 1000 \frac{kg}{m^3} \times 9.8 \frac{m}{s^2} \times 192.9m \\
&= 51.9\ MW
\end{aligned}
$$

Where:

η_T - turbine efficiency at rated head and max. discharge
Q_r - rated discharge
ρ - density of water
g - gravity acceleration

The assumed turbine efficiency η_T of 91.5% to 92% reflects an average value for turbines of similar specific speed at rated net head and maximum discharge. This value is subject to changes in connection to the final turbine design and the related hydraulic layout and therefore may vary depending on the final turbine supplier.

According to the expected operational regime, the variation of head is comparably low and therefore allows for an optimized hydraulic design of the turbine, especially with regards to part-load operation. As a result, the efficiency drop in part load can be minimized by the suppliers. Additionally, the selected number of units (3 turbines) allows for an operation close to the turbine's best point for a large operating range respectively plant discharge.

The maximum turbine power output is reached at the maximum available net head and maximum discharge. The operation at the Maximum Flood Level (MFL) is not considered as operational point. Therefore, the maximum net head is reached with a single unit operating at the Maximum Normal Operating Level of 1260.0 masl. Under consideration of the losses of approx. 3.9 m for single unit operation, the maximum net head is 197.2 m and therefore the maximum single unit output is:

$$\begin{aligned} P_{T,max} &= \eta_T \times Q_r \times \rho \times g \times H_{net,max} \\ &= 0.915 \times 30.0\frac{m^3}{s} \times 1000\frac{kg}{m^3} \times 9.8\frac{m}{s^2} \times 197.2m \\ &= 53.0\, MW \end{aligned}$$

7.3.4.2 Plant installed capacity

The installed capacity of the hydropower plant is understood as the sellable power and is measured at the high voltage site of the transformer terminals, without deduction of the required auxiliary consumptions.

Assuming a generator efficiency of η_G = 98% and a transformer efficiency of η_{Trans}= 99.5% the overall generating efficiency amounts to:

$$\begin{aligned} \eta_{Plant} = \eta_T \times \eta_G \times \eta_{Trans} &= 0.915\ to\ 0.92 \times 0.98 \times 0.995 \\ &= 89.2\ \%\ to\ 89.7\% \end{aligned}$$

The total installed capacity of the Sharmai HPP is:

$$\begin{aligned} P_{Installed} &= 3 \times \eta_{Plant} \times Q_r \times \rho \times g \times H_{net,3U} \\ &= 3 \times 0.894 \times 30.0\frac{m^3}{s} \times 1000\frac{kg}{m^3} \times 9.8\frac{m}{s^2} \times 192.9m \\ &= 152.12\, MW \end{aligned}$$

It shall be noted that the auxiliary consumption of approximately 1% of the generating unit power (η_{ac} = 99 %) is not considered in the above plant installed capacity. Net plant capacity amounts to 150.6 MW.

7.3.4.3 Turbine speed

The determination of the turbine layout and sizing is based on statistical data compiled by the Consultant for similar hydraulic units to the Sharmai HPP project. The main turbine characteristics have been established such that all major suppliers generally can meet those criteria in order to ensure competitive bidding.

Considering the number of generator poles and the AC current frequency of 50 Hz as well as the range of operational heads, the turbine speed has been selected to 428.6 rpm. At this speed, the centerline of the turbine spiral casing would have to be set approx. 3.0 m below the minimum tailwater level to ensure cavitation-free operation (Submergence Hs=-3.0 m). As a result, the maximum allowable turbine center elevation with regards to cavitation is:

$$TWL_{1Unit} + H_S = 1058.9masl - 3.0m = 1055.9masl$$

For the present powerplant layout, the maximum possible turbine setting with regards to the location of powerhouse cavern, the tailrace surge tank and the tailrace tunnel, is 1050.5 m a.s.l.. Since this level is lower than the minimum level required with regards to cavitation the selected turbine setting is 1050.5 m a.s.l.

7.3.4.4 Runaway speed

The runaway speed coefficient has been determined using an empirical formula developed by BUREC which is based on the unit specific speed. For the maximum normal static head of approx. 209.5 m, the following applies:

- Runaway speed coefficient: $f_r = 1.785$
- Max. runaway speed: $nr = n*f_r = 765$ rpm.

At present, the calculated maximum runaway speed has only an indicative character. The final runaway speed will be determined during turbine model tests, which have to be carried out by the selected turbine supplier. The determined final runaway speed will be used for the final design and correspondingly for stress calculations of the turbine and generator parts.

7.3.4.5 Turbine main dimensions

The size of the units is affected by the diameter of the turbine runner. This is influenced in turn by the rotating speed of the runner. The selected speed of 428.6 rpm leads to a turbine runner diameter of approx. 1,730 mm and determines the overall dimensions of the units as summarized hereunder:

Runner

D3=	1,73	m
D1=	2,10	m
H1=	0,16	m
H2=	0,58	m

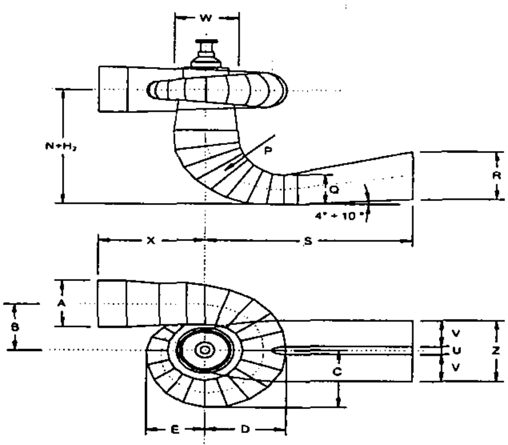
Spiral case/ distributor

Asv=	1,95	m
B=	2,48	m
C=	2,91	m
D=	3,22	m
E=	2,51	m
F[1]=	3,40	m
G[2]=	2,77	m
H[3]=	2,41	m
I[4]=	0,33	m
Asv=	1,83	m

Draft tube

N=	5,08	m
P=	2,77	m
Q=	1,74	m
R=	2,68	m
S=	9,42	m
T=	2,96	m
U[5]=	1,5	m
V=	1,94	m
W=	3,19	m
Z=	5,39	m

1) stay vanes outer circle
2) stay vanes inner circle
3) quide vanes circle diameter
4) quide vanes height
5) middle pier required if ns ≥ 120 (approx.) width of pier→as required by civil works



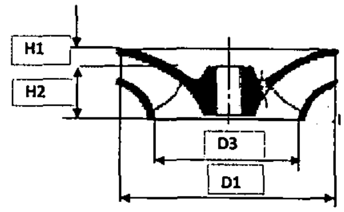


Figure 7-28: Summary main turbine dimensions

The general design of the electro-mechanical equipment and especially of the turbine, considers fast operating times to cope with the requirements related to the plant's planned peaking operation.

Within present feasibility design provisions are made for black-start and isolated grid operation.

In the following table the main parameters are given.

Table 7-12: Main parameters of turbine layout

Characteristic	Unit	Data
Type	-	Francis
Number of Units	-	3
Hn,rated	m	192.9
Q maximum design	m^3/s	3 x 30.0
Runner diameter	mm	approx. 1,730
Setting	m a.s.l.	1050.5
Rated speed	rpm	428.6
$P_{T,\,max}$ (1 unit operating)	MW	52.8
$P_{T,rated}$ (at rated net head and max. discharge)	MW	51.9
$P_{Installed}$	MW	152.12

7.3.4.6 Maintenance of turbine

The design of the turbine will consider operational aspects of inspection, maintenance and repair. Therefore, the turbine runner will be accessible from a manhole in the draft tube. The spiral casing, stay vanes and guide vanes can be inspected by entering the spiral casing.

For repair works on the turbine runner, the turbine runner the turbine will be equipped with a removable draft tube cone to enable removing the turbine runner through the draft tube passages without dismantling the generator rotor. A core drill in the generator and turbine shaft will allow for installation of a rod to lower the turbine runner on a cart. The runner will be transported on this cart through the turbine passage. On the upstream end of the powerhouse cavern, it will be lifted by means of the powerhouse crane.

This allows for fast dismantling and replacement of the turbine runner. It should be considered to have one spare turbine runner, allowing for iterative repair works on the dismantled runner. This approach may also provide advantages over a hard or soft-coating of the runner (to be re-considered during detailed design phase) since repair works of coated runners are very challenging and in general only possible in specialize workshops (or at the manufacturers facilities).

Furthermore, the turbine auxiliaries as the guide vanes and guide vane bearings, the shaft seal, etc. will be design for fast replacement and accessibility (e.g. the guide vanes will be adjustable and exchangeable without the necessity for dismantling of the head cover or bottom ring).

7.3.5 Design of Francis turbine

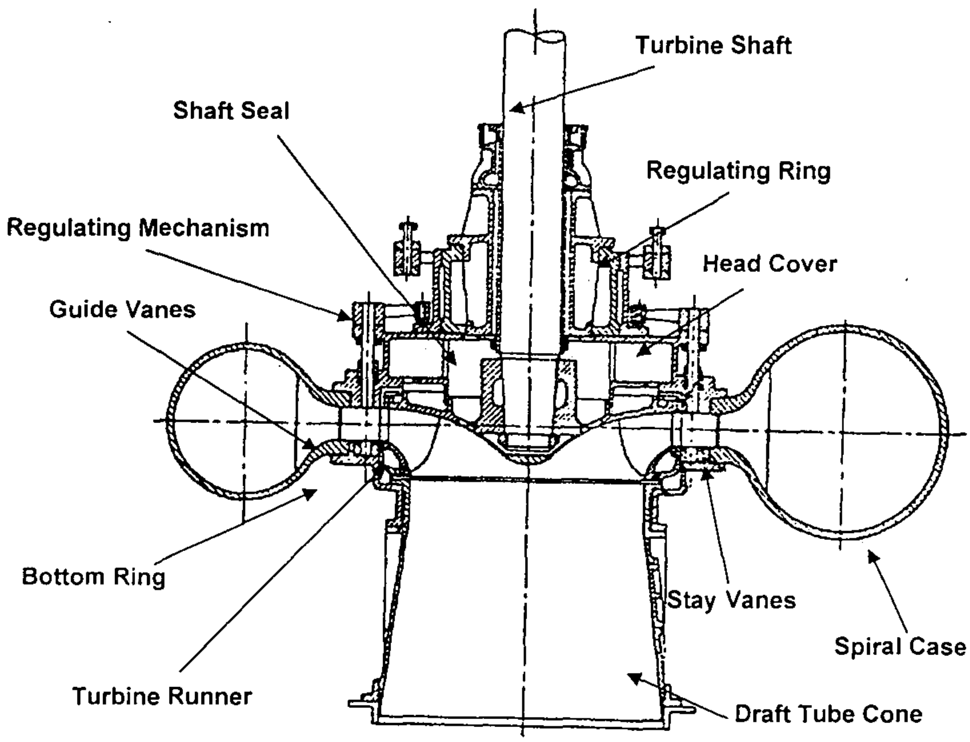


Figure 7-29: Section of Francis Turbine with nomenclature (Source: J.W. Daily "Hydraulic Machinery", Engineering Hydraulics, New York, 1950)

7.3.5.1 Spiral case with stay vanes

The spiral case of welded construction serves as inlet structure to the radial oriented stay and guide vanes, which convey the incoming water from axial to rotational flow. The spiral case for Sharmai will be of the embedded steel type, to bear the occurring loads and avoid any harmful vibrations.

The spiral case inlet is equipped with a manhole to permit access to the guide vanes and turbine runner from the upstream side for maintenance purposes.

The stay vanes provide the required strength of the structure to bear the loads occurring from the regulation mechanism and guide bearing and transfer the loads from the head cover to the foundations / concrete structure below the spiral casing.

7.3.5.2 Set of hydraulically operated guide vanes

The guide vanes are made of stainless steel and are used for the regulation of the incoming flow. The guide vane stems are supported by one lower and two upper self-lubricating bearings, which can be adjusted, exchanged and maintained without dismantling the head cover or bottom ring.

A guide vane lever is clamped to each upper stem, connected to the regulating ring with the aid of a safety link, which deflects if the closing operation is obstructed by an obstacle in between of two neighboring guide vanes. The regulating ring is running on a low resistance bronze or Teflon ring with low friction. It is operated by two double acting hydraulic servomotors.

7.3.5.3 Electronically and hydraulically operated turbine governor

The turbine is controlled by an electronic governor, which transforms each electronic signal into a hydraulic action to be executed by the hydraulic governor. For maintenance and commissioning purposes the governor can be operated from the local control panel of the electronic governor, but under normal operation it is remote controlled from the control room in the powerhouse or from any other place to be designated.

The governor is able to start, run and stop the turbine under each operating condition. It is able to provide at least output, speed or discharge control functions if pre-selected by the plant operator. Furthermore, the governor is capable of control of the reservoir water level, black start and regulation in isolated grid/frequency. The automatic selection of the units in operation and start/stop of the units depending on available discharge will be performed by a joint controller function.

The hydraulic governing system is equipped with a nitrogen system consisting of an accumulator and a nitrogen bottle bank to keep the system pressurized and ensure closing of the wicket gates in case there is no electricity available for operation of the hydraulic pumps.

7.3.5.4 Turbine shaft

The turbine-shaft is made from high-alloy forged steel, conveying the runner torque to the generator. The shaft shall allow for dismantling of the head cover without dismantling the generator.

The turbine shaft (as well as the generator shaft) is hollow in the center, in order to permit easy quality check of the forging and the passage of a lifting rod for dismantling of the turbine runner.

7.3.5.5 Turbine head cover

The turbine head cover is bolted to the stay ring. It encloses the fixed labyrinths, supports the turbine shaft seal, the wicket gate mechanism operating-ring and the upper guide vane bearings. The head cover can be equipped with tapings for a later aeration of the runner if operation at unfavorable operating points requires an air admission for dampening cavitation or partial load effects. The facing plates between the guide vanes and the head cover are made of stainless steel.

7.3.5.6 Axial shaft seal

The axial shaft seal is supported by the turbine head cover, cooled and sealed by clean, filtered water. During turbine operation it limits the water exiting the turbine cover at the rotating shaft. At standstill, a pneumatic standstill seal is inflated in order to avoid passage of water.

7.3.5.7 Guide bearing

The turbine-generator set is supported by one combined thrust and guide-bearing on the top of the generator, one guide bearing at the bottom of the generator and one guide bearing on the turbine head cover. The turbine guide bearing is operated by built-up of a differential pressure, created from the centrifugal forces acting in the oil-reservoir, clamped to the shaft.

7.3.5.8 Runner

The hydraulic profile of the runner and guide vanes has to be adapted to the operational requirements as specified in previous sections. Particularly the operating ranges (head and discharge) have to be duly considered and the runner shall be designed to allow a continuous, fail-safe operation without increased vibration, noise and draft tube pressure pulsations as well a cavitation-free operation over the full operating range.

The runner is a welded construction of high alloy steel made of pre-fabricated cast or forged blades and rings. Due to abrasive materials in the water, a hard or soft-coating of the runner might be considered during the detailed design phase. A cost analysis would need to be made if the extension of the lifetime would justify and compensate for the additional costs for the coating.

The runner is bolted to the turbine flange. Multi-stage labyrinth rings reduce the losses of water and pressure. In the center of the runner a hub cover supports the redirection of the water from radial to axial flow regime. Also the air inlet for the optional air admission system is located inside the cover.

7.3.5.9 Bottom ring

The bottom ring directs the water from the turbine inlet to the runner cone and houses the lower self-lubricating guide vane bearings. The bottom ring is bolted to the runner cone at the outlet of the runner. Detachable multi-stage labyrinth rings are bolted to the bottom ring with an appropriate minimum difference of the hardness to those of the runner. The facing-plates between guide vanes and bottom ring are made of high alloy stainless steel.

7.3.5.10 Draft tube cone and draft tube

The draft tube cone made of welded steel structure with steel-plates is bolted on its upper side to the bottom ring and to the embedded draft tube on the downstream side. The draft tube consists of two parts (vertically splitted) and therefore allows for easy inspection, repair and also dismantling of turbine runner without removal of the generator. After dismantling of the draft tube cone, the turbine runner can be lowered on a transport cart for removal through the draft tube passage and subsequent lifting to the erection bay if extended repair works and inspection are required.

Downstream of the draft tube cone, the embedded draft tube consisting of the elbow and straight section. The draft tube cone is connected to the embedded draft tube by bolted connection.

7.3.6 Main Inlet Valve

In front of each turbine one main inlet valve is installed as emergency shutdown and repair valve of the turbine. The main inlet valves are of the butterfly valve type, since with regard to the available head and discharge, this type is the most cost-efficient solution. Spherical valves result in significantly higher equipment costs and require larger dimensions for access facilities and capacity of lifting equipment.

Opening of the valve is done by means of one or two hydraulic servomotors. Closing is done by means of a counterweight, which closes the valve under all flow conditions, even at blackout or runaway. The axial forces of each butterfly valve are taken by the upstream flange of the inlet pipe. The butterfly valve is equipped with a maintenance seal allowing for repair of the main seal without dewatering the upstream waterways. A dismantling piece, permitting an easy dismantling of the valve, is arranged at the downstream side of the valve. The filling of the space between the pressure pipe and spiral case is effectuated through a hydraulically operated bypass-valve.

The inlet pipe upstream of the main inlet valves of one unit will be equipped with a dewatering pipe with a nominal diameter of DN400 for high-pressure dewatering, leading directly into the tailrace surge tank downstream of the draft tube stop logs. High-pressure dewatering will allow emptying of the power waterways, starting from the full static head.

All three units will be equipped with a low-pressure dewatering pipe with a nominal diameter of DN200 for emptying purposes. These dewatering lines will be used to empty the lower headrace tunnel sections starting from a water level corresponding to the level in the tailrace tunnel respectively the river. The drain lines will be connected to the powerhouse drainage and dewatering sump via the draft tube.

Table 7-13: Main data of main inlet butterfly valves

Characteristic	Unit	
Number of Units	-	3
Nominal diameter DN	mm	1,950
Nominal pressure PN	bar	24

7.3.7 Mechanical auxiliaries

7.3.7.1 Air admission system

The turbines should generally operate without aeration under all operating conditions. However, to improve the operating conditions and to reduce vibration at part-load conditions, aeration-taps are foreseen for a possible later admission of air to the runner and the draft tube. The necessity is finally depending on the supplier's turbine design.

7.3.7.2 Lubrication oil supply system of bearings

Each turbine is equipped with one joint oil supply system for supply of lubricating oil to all bearings. The oil will be pumped be means of one of two redundant electrically driven oil supply pumps, which are backed up by a high-pressure start-up pump to supply the generator axial bearing during start-up. The heated lubrication oil will be cooled by the units' cooling water system.

7.3.7.3 Cooling water system

A cooling water system will be installed to supply sufficient cooling water for the turbine-generator units, the hydraulic governors, lubrication oil system, compressors, air conditioning system and any other equipment installed that requires cooling capacity.

Each unit is equipped with a separate cooling water system and all cooling water systems are identical. A dual cycle cooling water system is used with a separate open raw water circuit (primary circuit) and a closed cooling water circuit (secondary circuit) supplying cooling water to the various equipment components. The primary and secondary cooling water circuits are interconnected by an intermediate heat exchanger of the flat plate type. This type of heat exchanger allows for easy dismantling and cleaning in case of fouling, etc.

The raw water circuits receive their water from the tailrace tunnel / tailrace surge tank by one of two centrifugal pumps (one operating and one standby) through check valves, flow meters, maintenance valves, regulating orifices, automatic self cleaning filters and intermediate heat exchanger and delivered to the tailrace surge tank.

The raw water is passed through two redundant self-cleaning automatic back-flushing filters, allowing maintenance of one system in case the other is operating.

The closed cooling water circuit (secondary circuit) is filled with treated water and is equipped with two redundant circulating pumps supplying the different consumers with cooling water and compensating the system losses. Additional treated water for the closed circuit will only needed in case of losses in the system. Therefore a make-up tank shall be installed which will be fed by the portable water system of the powerhouse cavern.

The cooling water systems of the three units are interconnected by one common pipe to facilitate maintenance of the system and to allow the usage of the primary cooling water circuit of one unit to be used for another unit.

The required cooling water amount of each cooling water system is estimated to be approx. 0.15 m^3/s.

7.3.7.4 Shaft seal water supply filter system

To reduce the leakages at the rotating shaft, each unit is equipped with a shaft seal. This shaft seal is supplied with cooling and sealing water. Both the cooling and sealing water will be taken from the raw water circuit of the cooling water system on the downstream side of the back-flushing filters. To further reduce the suspended particles and reduce wear of the sealing elements, the sealing and cooling water for the shaft seal is passed through an additional fine-filter prior it is lead to the shaft seal. Additional booster pumps shall be installed if required by the used shaft seal system.

7.3.7.5 Drainage and dewatering system

One dewatering and one drainage systems will be installed in the powerhouse cavern of Sharmai.

The dewatering system will consist in general of the following two systems:

- <u>High-Pressure Dewatering System</u>: For dewatering of the waterways upstream of the main inlet valve (down to a level corresponding to the level in the tailrace surge tank respectively river), a high-pressure dewatering line will be installed at the inlet pipe of unit 1. This high-pressure dewatering line (DN400) will directly lead into the tailrace surge tank.
- <u>Low-Pressure Dewatering System:</u> All three units will be equipped with low pressure dewatering lines installed upstream of the main inlet valves for emptying the lower tunnel sections / manifold starting from a water level corresponding to the level in the tailrace tunnel respectively the river. Additionally, each unit shall be equipped with pipes for dewatering of the spiral cases. Both, the low-pressure dewatering lines from upstream of the main inlet valves and from the spiral cases shall lead to

the draft tube of the respective unit. Dewatering pipes from the draft tubes will be connected to the powerhouse cavern dewatering sump.

The drainage system will cover the complete powerhouse cavern and all seepage water / surface leakage at the different powerhouse cavern floors. Additionally, all turbine shaft seal leakages and water collected on the head cover will be collected and lead to the free level drainage sump. All drainage water from the powerhouse as well as from the shaft seals respectively the turbine head covers will pass through oil separators which are expected to be located in the drainage sump. The capacity and as well as the final location of the oil separator(s) will however be determined during a detailed design phase.

The drainage and dewatering system will consist of two free surface sumps sized to collect all leakage from the shaft seals and from the floor drains.

The drainage and dewatering sumps will be located between unit 1 and unit 2. Both sumps will be accessible from the drainage and dewatering floor at el. 1047. 5 m a.s.l. by means of marine ladders. Each sump will be divided into two chambers and both chambers will be interconnected through a pipe and isolating valve in order to allow separation and dewatering of the chambers for cleaning and maintenance.

In each chamber of the two sumps, two vertically aligned raw-water pumps of the submersible type with water level switches for auto starting and stopping will be installed. Both submersible pumps of one sump are connected to one common discharge collector line leading to the tailrace surge tank. The outlet of these collector lines will be located above the maximum possible tailwater of approx. 1065 m a.s.l.

The transformer caver, located at the upstream end of the powerhouse, will be equipped with a separate drainage system. All drainage water, including water / oil from the transformer oil pits located below the three step-up transformers will be lead through an oil separator for the transformer cavern. The oil will be stored in a separate tank to cope with the amount of oil in case of a transformer rupture and operation of the water sprinkler system. The clean water from the transformer cavern oil separator will be lead to the powerhouse cavern main drainage system.

7.3.7.6 Service air system (Low Pressure)

The service air system supplies pressurized air at a working pressure of 8 bar to various consumers in the powerhouse cavern, such as inflatable standstill and maintenance seals on the turbine shaft, generator brakes and service air take-off points located throughout the powerhouse and transformer cavern for air supply to pneumatic tools and other equipment used during maintenance.

The low pressure compressed air system mainly consists of two motor driven screw type compressors, one as stand-by. The compressors will deliver the air to two main air receivers. From these main receivers the air will be distributed to various consumption points in the powerhouse via a network of steel pipes.

Additionally, each unit will be equipped with a separate air accumulator for the generator braking system. These air accumulators will also be supplied by the main compressed air system.

The low pressure compressed air system is designed for automatic operation with the possibility of local manual control of the individual compressors from the local control panel.

The air receivers and the pressure accumulators are designed for normal working pressure of 10 bar in compliance with ASME Boiler and Pressure Vessel Code, Section VIII; Division 1 or equivalent.

7.3.7.7 Heating, Ventilation and Air-Conditioning System (HVAC)

For the powerhouse cavern complex including control room, transformer cavern, access and power cable tunnels, a complete heating, ventilation and air-conditioning system (HVAC) will be installed to provide sufficient flow of air with the required conditions (temperature, humidity) to all areas so that equipment can operate as reliably as possible and at the same time, a safe working environment is provided for people working within those areas. The heating, air conditioning and ventilation system must function as an integral part of the overall fire protection and evacuation plan for the underground complex.

In addition to the general applicable standards, the heating, ventilation and air-conditioning system and related equipment will be designed, constructed, erected and tested under consideration of the following standards:

- ASHRAE (American Society of Heating, Refrigerating and Air Conditioning Engineers)
- AMCA (Air Moving and Conditioning Association)
- ASRE (American Society of Refrigerating Engineers)
- NFPA (National Fire Protection Association).

The heating, ventilation and air-conditioning system will draw fresh air through ducts installed in cable or access tunnels, leading to the surface. The fresh air will be handled by two redundant air-handling units, installed in the powerhouse cavern. After passing the air-handling units, the air will be distributed to the different floors of the powerhouse cavern. The fresh are will be in general supplied to the lower powerhouse levels and transferred to higher floors through transfer grids.

From the machine hall, it will be discharged through the bus passages to the transformer cavern and finally, via access or cable tunnels to the outside.

The tunnel used for discharging used air will be different from the tunnel used for drawing fresh air.

Air circulation will be controlled by fans installed in the ventilation tunnel, the entrance to the bus passage, the entrance to the transformer cavern and at the exit from the transformer cavern.

The battery room in the powerhouse cavern will be equipped with two separate exhaust fans for the exhaust of the ventilation air. The exhaust fans will be located inside the battery room or draw the air from the battery room. Each fan will be equipped with a self-closing non-return damper to avoid ingress of dust and debris and to prevent backflow of air due to the negative pressure in the battery room. The size and capacity of the ventilation system of the battery room will be determined be in accordance with EN 50272 Part 2. The exhaust air will not be mixed with other air distributed in the powerhouse cavern.

The comfort levels are particularly important in those areas (the machine hall floor and erection bay as well as the generator and turbine floor), where men will be regularly working during plant operation or routine maintenance.

The bus passages and transformer cavern are normally not classified as working areas, and here the principal consideration will be to control temperatures to meet equipment operating requirements.

The heat losses, generated by some equipment located in the cavern will be transported by means of air flow rate which is warmed up to approx. 5 K (degree). The outer protective sheet of the 220 kV XLPE cables will be fire-resistant (state of the art).

Toilets, oil store / treatment, washrooms or similar rooms must be ventilated by exhaust air ventilation systems to prevent bad smelling air.

Additionally, to the HVAC system for the powerhouse cavern, also the switchyard building as well as the dam control building will be equipped with a HVAC system to ensure that temperatures and humidity in those building will stay in acceptable limits for the installed equipment and under consideration of their use.

The heating, air conditioning and ventilation system is designed to keep the internal temperatures in the different areas in the ranges stated in the table hereunder:

Table 7-14: Internal temperature ranges for different areas

Area	Internal Air Temperature (°C)	
	Minimum	Maximum
Machine Hall and Erection Bay	18	35
Lower Levels in Power House Cavern	18	35

Area	Internal Air Temperature (°C)	
	Minimum	Maximum
Control Room	22	26
Offices	22	26
Workshop and Stores	18	38
Switchgear Rooms	12	35
Relay Room	12	30
Electronic Equipment Rooms	18	30
Battery Rooms	12	30
Mess Rooms	22	26
Toilets and Washroom	18	30
Switchyard Building	18	35
Dam Control Building	18	35

The minimum air exchange rates for the different areas are defined in the following table:

Table 7-15: Minimum number of air changes for different areas

Area	Number of air changes per hour
Machine Hall and Erection Bay	1
Lower Levels in Powerhouse Cavern	4
Transformer Cavern and Bus Passage	4
Electromechanical Rooms	8
Offices	6 but min. 10 l/s per person
Workshop	6
Battery Rooms	6
Mess Rooms	8
Toilets and Washroom	8
Switchyard Building	4
Dam Control Building	4

The HVAC system is designed in a way that no significant background noise level is added to the area/room. The following table details the maximum permissible noise ratings due to the air conditioning and ventilation equipment under full load operation:

Table 7-16: Maximum permissible noise levels for different areas

Area	Maximum permissible noise level
Machine Hall and Erection Bay	45
Lower Levels in Powerhouse Cavern	60
Transformer Cavern and Bus Passage	60
Electromechanical Room	60
Offices	45
Workshop and Stores	45
Mess Rooms	45

Area	Maximum permissible noise level
Toilets and Washroom	45

The major potential fire hazard and source of smoke and products of combustion are the transformers and the ventilation system will minimize the circulation of combustion products should a fire occur.

Therefore, the design of the powerhouse ventilation must be closely coordinated with the need for the fire detection/annunciation system to react properly in various modes of operation, depending on where the fire has been observed or automatically detected, to ensure the safety of the personnel and the equipment. The ventilation system must accomplish the following:

- ensure a smoke-free route for personnel evacuation
- vent the smoke safely, efficiently and rapidly.

7.3.7.8 Fire fighting system

The power plant will be equipped with a state-of-the-art fire fighting, detection and alarm system. This system will consist of smoke sensors, temperature detectors, deluge valves, water sprinkler system, fire hose cabinets, fire hoses, portable fire extinguishers, control panel.

The fire fighting equipment installed at the power plant will comprise the following components and sub-systems:

- automatic fire fighting system based on deluge valves, water sprinkler system, fire hoses as per NFPA
- microprocessor based fire detection and alarm system
- portable and mobile type chemical fire extinguishers.

The water for the fire fighting system will be drawn from the tailrace surge tank and pumped by two feeder pumps of suitable capacity into a reservoir tank placed in or close to the powerhouse cavern, having capacity for at least 2 hours supply of the single largest deluge in the powerhouse complex. To build up the necessary pressure in the fire fighting water system (approx. 6 bar), two independent pumps, one electrically driven, one driven by a diesel motor will be installed. The reservoir tank will have level switches for automatic starting and stopping of the two feeding pumps.

In case of maintenance work in the tailrace tunnel, the tank/reservoir has to be fed by temporary submergible pumps, installed in the river bed.

7.3.7.9 Oil treatment unit

The facilities will include a mobile oil filtration unit for purifying the transformer oil system. The mobile high vacuum oil treatment unit is equipped with evacuation pumps, vacuum pumps, degasifier, heaters, filters, connecting rubber hoses etc. and any other items required for conditioning of the oil to maintain its dielectric properties. Furthermore, one mobile transformer oil tank of sufficient capacity is included to store oil of one main transformer, with lifting lugs for lifting the tank completely filled with oil.

A separate bearing and governor oil purifying system is foreseen. Both oil treatment units will be capable of the following functions:

- filtration, dehydration and degassing of oil supplied in drums or in a tanker before filling into transformers, respectively into the bearing or governing systems
- processing of oil after long periods of use
- regeneration of oil after long periods of use to restore loss factor, surface tension, saponification, acidity factor etc. to values approaching those of new oil
- addition of inhibitor.

7.3.7.10 Powerhouse overhead travelling crane

The required total maximum lifting capacity of the electrical overhead travelling (EOT) powerhouse crane is determined by the generator rotor which will weigh around 150 tons. The weight of all other equipment to be handled by the main hook will be much less than 100 tons. An auxiliary hook of 10 tons capacity will be provided on the crane and will run along the main bridge beam; this will be used for handling smaller equipment and for normal maintenance. The present proposed capacity of the crane should be reviewed during tender stage as soon as the suppliers confirm the actual weight of the generator rotors.

Travelling crane rail beam elevation	1065.5 masl
Powerhouse machine hall floor elevation	1058.0 masl
Generator floor elevation	1052.3 masl
Turbine runner centerline elevation	1050.5 masl
Lowest powerhouse level	1046.5 masl
Span of travelling crane bridge beams	Approx. 18 m
Length of travelling crane rails	Approx. 54 m
Main crane hoist capacity	150 tons
Auxiliary crane hoist capacity	10 tons
Speed of main hoist	0 - 1.2
Speed of auxiliary hoist	0 - 5
Speed of hoist trolley	0 - 10

Speed of main travelling bridge	0 - 15
Power supply for hoist, trolley and bridge motors	400

A list of all cranes can be found in Chapter 4.

7.3.7.11 Workshop

The power plant is located in a remote area with low infrastructure nearby. Therefore, the maintenance and repair works to be performed will include basic machinery like turning, milling, drilling, grinding, welding, etc.

A suitable mechanical workshop shall be fully equipped with machine tools, equipment and tools necessary for normal maintenance and small repair of the turbines, generators and auxiliary equipment installed in the powerhouse cavern, switchyard and at the power intake. The workshop shall either be placed inside the powerhouse cavern or at a building located close to the access tunnel at the surface.

7.4 Design of Hydraulic Steel Structures

7.4.1 General

The feasibility layout and design of Sharmai HPP comprises the following main components that include hydraulic steel structures (HSS):

- gated spillway
- sand trap
- power intake
- bottom outlet
- tailrace surge tank / draft tube outlet
- tailrace tunnel outlet
- diversion tunnel.

The equipment for the gated spillway includes:

- three radial segment gates with hydraulic drives and hinged flap gates
- one set of stop logs for maintenance
- gantry crane with a capacity 25/5 t (also serving the sand trap inlet and bottom outlet).

The equipment for the sand trap includes:

- three sets of trash rack screens
- one traversing trash rack cleaning machine
- three sand trap inlet gates of the roller type with hydraulic drives
- one set of inlet stop logs for maintenance of the inlet gates as well as for the bottom outlet
- three sand trap outlet gates of the roller type with hydraulic drives

- one set of outlet stop logs for maintenance of the outlet gates
- four flushing gates with hydraulic drives
- sand trap outlet gantry crane with a capacity 15/5 t.

The bottom outlet equipment includes:
- one submerged radial segment gate with hydraulic drive.

The power intake facilities include:
- one emergency roller gate with hydraulic drive.

The tailrace surge tank / draft tube outlet section will include these HSS equipment:
- three sets of stop logs;
- one monorail hoist with a capacity of 10 t.

The tailrace tunnel outlet section will include these HSS equipment:
- two sets of stop logs.

The diversion tunnel will include the following equipment:
- one set of permanent stop logs for closing the diversion tunnel.

Hereunder all cranes which will be installed for the Sharmai HPP project are listed:

Table 7-17: Cranes and related capacities for Sharmai HPP

Crane	Expected Approx. Capacity
Powerhouse overhead travelling crane	150/10 tons
Dam gantry crane	25/5 tons
Sand trap outlet gantry crane	15/5 tons
Draft tube outlet monorail hoist	10 tons

7.4.2 Design criteria

The design of all gates, stop logs and other equipment with the appurtenant drives has to fulfill the requirements according to the standards for hydraulic steel structures DIN 19704 (Criteria for design and calculation) and DIN 19705 (Recommendations for the design, construction and erection of hydraulic steel structural equipment) or equivalent British or American standards.

Particular attention is given to the potential impact of bed load and the expected rate of sediment and abrasive material transport in the river. An appropriate and proven wear resistant corrosion protection system shall be applied to all steel components exposed to water and not made of stainless steel. This system has to comply with ISO 12 944 and ISO 4628.

The hydraulic steel structure equipment described below refers to state-of-the-art design and international recognized standards are applied to the design.

7.4.3 Gated spillway

The spillway is equipped with three radial segment gates including hinged flap gates. The radial gates are used for flood control and discharge of excess water. Operation control of the gates is possible from a local control building located close to the dam and remotely from the main control room located in the power plant.

7.4.3.1 Radial segment gates

The radial segment gates are used for flood release respectively to allow discharge of flood waters when inflows exceed the design of the power station at full reservoir level.

The three radial gates are equipped with flap gates at the top. They are provided to allow for periodic flushing of floating debris and trash arriving in the reservoir and fine regulation of the reservoir water level without the necessity of opening the main radial gate

Basic data and design criteria of the radial gates are:

Table 7-18: Basic data and design criteria of the spillway radial gates

Type	Radial gate with flap
Number of gates	3
Clear width of one spillway bay opening	10.5 m
Gate height (approx.)	15.5 m
Freeboard	0.5 m
Sill elevation	1245.0 masl
Maximum Operational Level	1260.0 masl
Maximum Flood Level	1263.8 masl
Crest elevation of the piers	1263.5 masl
Max. operation load:	All hydraulic loads, dead weights and friction loads
Hoist	Oil-hydraulic
Gate normal operation speed	0.3 m/min

The gates are manufactured with class S355J0 structural steel according to DIN EN 10025 and are designed according to DIN 19704.

Two servomotors connected to the gate arms operate each spillway radial gate. The design and capacity of each servomotor considers the exceptional load case, to hold the complete radial gate, even with one failing servomotor.

Each radial gate is equipped with a separate hydraulic power unit, which is located at the spillway crest. The local control boards will be installed in the dam control building close to the gated spillway.

In the event of main power failure, power supply to the hydraulic units will be provided by a diesel-powered generator located at the switchyard. As an additional backup system, each hydraulic unit is equipped with a hand pump to move the gates. Those hand pumps are installed at the hydraulic power unit.

The trunnion bearings for the gates are of spherical bronze bush type to allow more flexibility of the structures. Both trunnion bearings are self-lubricating.

The frames consist of a sill beam profiled to be tangential to the spillway surface, and side frames consisting of a combined seal seat, and track for the leaf side guide wheels. The seal seats/track surfaces are machined to the tolerance permitted in DIN 19704. Side frames are extended to the top of the spillway piers. All seal seats are of stainless steel, machined and polished.

7.4.3.2 Spillway stop log gate

In order to enable in-situ maintenance works at the radial gates, one set of maintenance stop logs for the spillway, to be installed upstream of the radial gates, is provided. All stop log elements have the same shape and dimensions and are interchangeable.

The stop logs will be installed and lifted with the dam gantry crane and a lifting beam at balanced water level conditions between the stop logs and the radial gate. For lifting of the stop logs, balanced water conditions are achieved by slightly lifting the upper stop log element by means of the gantry crane (crack opening).

The stop logs will be stored in the storage slot at the right side of the spillway.

Table 7-19: Basic data and design criteria of the spillway stop logs

Type	.Sliding / Fixed Roller
Clear width of one spillway bay opening	10.5 m
Number of stop log sets	1
Number of stop log elements	4
Height of one stop log element	approx. 3.9 m
Freeboard	0.50 m
Sill elevation of stop logs	1245.0 masl
Maximum Operational Level	1260.0 masl
Maximum Flood Level	1263.8 masl
Crest elevation of the piers	1263.5 masl
Placing stoplogs	Balanced water conditions
Max. operation load:	All dead weights and friction loads
Sealing:	At downstream side
Lifting device:	Dam gantry crane and lifting beam

7.4.3.3 Dam gantry crane

One gantry crane with an approximate lifting capacity of 25/5 tons will be installed to serve the spillway and dam. The final capacity of the crane is to be coordinated with the design of the equipment to be handled at the dam. The crane shall be able to travel the whole dam crest including the storage slots and the left and right side of the dam.

The gantry crane will serve to install and lift the following equipment:

- sand trap inlet stop logs
- bottom outlet stop logs
- spillway stop logs.

Additionally, it will be used in case of repair works of the main gates at the spillway, sand trap inlet and bottom outlet.

The basic framework and the various mechanisms comprising the crane are designed and fabricated to permit various maneuvers, necessary during the installation of the stop logs, smoothly and accurately. The motor components producing the various movements are fitted with a progressively acting braking system which shall kick-in in case of any accidental or intentional cutting off of the power supply.

The design capacity of the crane considers the dead load, which includes the weight of all parts of the crane, except those parts specified as live load. The maximum live load includes the lifted loads, lifting beams, lifting devices, lower hoist block, hooks, ropes and trolley.

Lateral live loads caused by acceleration, wind, earthquake, collision forces and braking operation have to be considered during design and dimensioning.

Main characteristics of the gantry crane are:

Table 7-20: Basic data and design criteria of the dam gantry crane

Rail track elevation	1263.8 masl
Total rail track length approx.	160 m
Hoisting capacity main hoist	approx. 25 tons
Hoisting speeds main hook:	
normal speed	1.6 m/min
creeping speed	0.3 m/min
Hoisting capacity auxiliary hoist	approx. 5 tons
Hoisting speeds auxiliary hook:	
normal speed	4 m/min
creeping speed	0.5 m/min
Gantry travelling speed	
normal speed	25 m/min
creeping speed	0.6 m/min
Trolley travelling speed	
normal speed	15 m/min
creeping speed	0.6 m/min
Crane control	
from an enclosed cabin fixed to the gantry crane frame	
Design criteria	
crane structure	FEM class A3
crane mechanism	FEM class M2

7.4.4 Sand trap

7.4.4.1 Trash racks

Each of the three sand trap inlets is equipped with trash rack screens covering the complete inlet area with a clear width of approximately 8.0 m and a height of approximately 7.5 m each. The frames and screen segments are fabricated from structural steel plates and sections which comply with EN 10025 or other approved standards dealing with structural steelwork.

The design provides vibration-free performance and minimal head loss. A clear spacing between screen bars of 60 mm is selected and will be re-confirmed in the tender design in co-operation with the turbine manufacturer. The screens have vertical screen bars. The edges of the bars are designed to prevent or suppress flow induced vibration in flow direction. The trash rack shall be designed to withstand a differential head of not less than 18 m.

Suitable differential pressure sensors will be installed to register pressure differentials across the screens. The sensors will trigger an alarm as soon as the differential pressure reaches 0.05 m and will further initiate the operation of the trash rack cleaning machine. If the differential pressure continues to rise and reaches 0.1 m, the sensor shall initiate closing of the sand trap inlet gate.

7.4.4.2 Trash rack cleaning machine

One common trash rack cleaning machine is used for the three sand trap inlets. The cleaning machine is a traversing manual, semi- and automatically operated raking machine to assure removal of debris and trash from the fixed trash rack of the sand trap inlets. The rake is arranged to travel up and down the screens and bring trash to the dam crest and discharge it into a suitable trash skip trolley with the minimum of handling by the operator.

The down movement of the rake head is in the "open" position. Arriving at the screen sill, the rake head automatically tilts to the "closed" position and the collected trash is firmly held by the rake head when travelling upwards. If necessary, special guide plates or bars above the top of the trash rack as well as provisions for guiding the trash over the top of the intake and down to the trash container are installed. The container is designed to enable emptying easily by tilting and it travels on own rails on the inside of the rails for the cleaning machine.

Lateral movement of the rake is ensured by the travel of the unit on the runway rails.

The electrical controls for the raking machine allow the following modes of operation:

- automatic operation
- semi-automatic operation
- manual operation.

A control cabinet shall be installed on the portal from where all operations of raking can be controlled.

The rake head is of the heavy design and it needs to be ensured that the rake can be lowered by gravitational force at least through a 0.5 m thick mat of trash. Additionally, the cleaning machine is equipped with a hydraulic davit for removal of oversized contamination (e.g. timber) arriving at the intake.

The main characteristics of the raking machine are as follows:

Table 7-21: Basic data and design criteria of the trash rack cleaning machine

Mode of operation	local and remote control (manual and automatic operation)
Elevation of operation deck	1263.8 masl
Sill elevation of trash racks	1242.0 m a.s.l.
Height of trash rack (inclined)	12.4 m
Width of trash rack screens	10.0 m
Width of rake	approx. 3.5 m
Rake net capacity	5 tons

7.4.4.3 Sand trap inlet gate

Each of the three sand trap inlets will be equipped with a roller gate, located downstream of the sand trap inlet stop logs. All three gates will be identical.

The sand trap inlet gates, together with the sand trap outlet gates and headrace tunnel intake gate, will serve as emergency closing devices in case of failures or damages in the waterways, extraordinary pressure difference at the trash rack, etc. Additionally, the gates will control the discharge to the three settling tanks and therefore control the operation of the sand trap.

The gates seal upstream and have ballast for gravity closure against full flow with adequate factors of safety against hydraulic forces and friction. The gates shall operate without failure or vibration under the most adverse combination of hydraulic flow and mechanical resistance due to friction, debris or other causes. Operation of the sand trap inlet gates is by hydraulic servomotors served by three independent hydraulic power units to provide complete independent systems and redundancy.

All gate controls and hydraulic units will be housed in the control building at the dam. Control of the gates will be possible by remote control from the powerplant and by an emergency closure control button in the control building at the dam.

Gates will be designed, fabricated and erected in accordance with DIN 19704 as applicable.

Table 7-22: Basic data and design criteria of the sand trap inlet gates

Type	Fixed Roller Gate
Number of gates	3 (one per settling tank)
Clear width of opening	7.0 m
Clear height of opening	8.0 m
Sill elevation	1242.0 masl
Maximum Operational Level	1260.0 masl
Maximum Flood Level	1263.8 masl
Operation	Open against max. differential head Close at max. flow
Max. operation load	All dead weights and friction loads
Seal position:	upstream
Operating mechanism:	Hydraulic Hoist

7.4.4.4 Sand trap outlet gate

Each of the three sand trap outlets will be equipped with a roller gate, located upstream of the sand trap outlet stop logs. All three gates will be identical.

The sand trap outlet gates, together with the sand trap inlet gates and headrace tunnel intake gate, will serve as emergency closing devices in case of failures or damages in the waterways, extraordinary pressure difference at the trash rack, etc. Additionally, the gates will control the discharge from the three settling tanks to the intake chamber and therefore control the operation of the sand.

The gates seal upstream and have ballast for gravity closure against full flow with adequate factors of safety against hydraulic forces and friction. The gates shall operate without failure or vibration under the most adverse combination of hydraulic flow and mechanical resistance due to friction debris or other causes. Operation of the sand trap outlet gates is by hydraulic servomotors served by three independent hydraulic power units to provide complete independent systems and redundancy

at the dam. Control of the gates will be possible by remote control from the powerplant and by an emergency closure control button in the control building at the dam.

Gates will be designed, fabricated and erected in accordance with DIN 19704 as applicable.

Table 7-23: Basic data and design criteria of the sand trap outlet gates

Type	Fixed Roller Gate
Number of gates	3 (one per settling tank)
Clear width of opening	7.0 m
Clear height of opening	8 0 m
Sill elevation	1242.0 masl
Maximum Operational Level	1260.0 masl
Maximum Flood Level	1263.8 masl
Operation	Open against max. differential head Close at max. flow
Max. operation load	All dead weights and friction loads
Seal position	upstream
Operating mechanism:	Hydraulic Hoist

7.4.4.5 Sand trap inlet and bottom outlet stop log gate

In order to dewater the sand trap inlet gates without the necessity of lowering the reservoir for the purpose of inspection and maintenance, one set (one for all three settling tanks) of sand trap inlet stop log gates is provided to be installed upstream of the sand trap inlet gates. The same set of stop logs is used as maintenance stop log gate for the bottom outlet radial gate. Due to the higher water pressure on the bottom outlet gate, the sand trap inlet stop log gates are design for the pressure on the bottom outlet.

All three sand trap inlets are equipped with slots for installation of the inlet stop log gate elements. All stop log elements are of the same type and dimensions.

The stop logs will be installed and lifted with the dam gantry crane and a [illegible] the sand trap inlet gate [illegible] lifting, the balanced water conditions are achieved by slightly lifting the upper stop log element by means of the [illegible] stop logs will be stored in the storage slot at the left side of the dam.

Basic data and design criteria of the sand trap inlet stop logs are:

Table 7-24: Basic data and design criteria for the sand trap inlet stop logs (and bottom outlet)

Type	Sliding / Fixed Roller
Clear width of the intake opening bays	7.0 m
Number of stop log sets	1
Number of stop log elements per set	3
Height of one stop log element	approx. 2.7 m
Sill elevation of stop log intake	1242.0 masl
Sill elevation of stop log bottom outlet	1232.5 masl
Maximum Operational Level	1260.0 masl
Maximum Flood Level	1263.8 masl
Crest of the piers	1265.0 m a.s.l.
Placing stoplogs	Balanced water conditions
Max. operation load:	All dead weights and friction loads
Sealing:	At downstream side
Lifting device:	Gantry crane and lifting beam

7.4.4.6 Sand trap outlet stop log gate

In order to dewater the sand trap outlet gates without the necessity of lowering the intake chamber for the purpose of inspection and maintenance, one set (one for all three settling tanks) of sand trap outlet stop log gates is provided to be installed downstream of the sand trap outlet gates.

All three sand trap outlets are equipped with slots for installation of the stop log gate elements. All stop log elements are of the same type and dimensions.

The stop logs will be installed and lifted with the sand trap outlet gantry crane and a lifting beam at balanced water level conditions between the stop logs and the sand trap outlet gate. For lifting of the stop logs, balanced water conditions are achieved by slightly lifting the upper stop log element by means of the gantry crane (crack opening).

The stop logs will be stored in the storage slot at the left side of sand trap outlet.

Basic data and design criteria of the sand trap outlet stop logs are:

Table 7-25: Basic data and design criteria for the sand trap outlet stop logs

Type	Sliding / Fixed Roller
Clear width of the intake opening bays	7.0 m
Number of stop log sets	1
Number of stop log elements per set	3
Height of one stop log element	approx. 2.7 m
Sill elevation of stop log	1242.0 masl
Maximum Operational Level	1260.0 masl
Maximum Flood Level	1263.8 masl
Crest of the piers	1265.0 m a.s.l.
Placing stoplogs	Balanced water conditions
Max. operation load:	All dead weights and friction loads
Sealing:	At downstream side
Lifting device:	Gantry crane and lifting beam

7.4.4.7 Flushing gates

At the end of each settling tank a flushing gate will be provided. Additionally, a flushing gate will be installed at the bottom of the intake chamber to allow for dewatering.

The flushing gates shall be of the sliding type and shall be operated by means of a hydraulic servomotor. The oil pressure for operation of the servomotors will be from a common hydraulic power unit serving all four flushing gates.

Each channel in which the flushing gates are installed will be steel lined to prevent erosion.

The flushing gates will regularly be opened for flushing of sediments from the sand trap. The flushing gates will be designed to allow for permanent operation at all partial opening positions and will be capable of being opened, closed or stopped in any intermediate position under full one side pressure without vibration.

The gate controls and hydraulic unit will be housed in the control building at the dam. Control of the gates will be possible by remote control from the powerplant and locally in the control building at the dam.

Table 7-26: Basic data and design criteria of the flushing gates

Type	Sliding Gate
Number of gates	4 (one per settling tank and one for dewatering of the intake chamber)
Clear width of opening	2.0 m
Clear height of opening	4.0 m
Sill elevation	Approx. 1231.0 masl
Maximum Operational Level	1260.0 masl
Maximum Flood Level	1263.8 masl
Operation	At all position at maximum flow and head
Max. operation load	All dead weights and friction loads
Seal position:	upstream
Operating mechanism:	Hydraulic Hoist

7.4.4.8 Sand trap outlet gantry crane

One gantry crane with an approximate lifting capacity of 15/5 tons will be installed to serve the sand trap outlet. The final capacity of the crane is to be coordinated with the design of the equipment to be handled at the sand trap outlet. The crane will be able to travel along all three sand trap outlets, the storage slot and the handling platform on the left side of the outlet slots.

The basic framework and the various mechanisms comprising the crane are designed and fabricated to permit various maneuvers, necessary during the installation of the stop logs, smoothly and accurately. The motor components producing the various movements are fitted with a progressively acting braking system which shall kick-in in case of any accidental or intentional cutting off of the power supply.

The design capacity of the crane considers the dead load, which includes the weight of all parts of the crane, except those parts specified as live load. The maximum live load includes the lifted loads, lifting beams, lifting devices, lower hoist block, hooks, ropes and trolley.

Lateral live loads caused by acceleration, wind, earthquake, collision forces and braking operation have to be considered during design and dimensioning.

Main characteristics of the gantry crane are:

Table 7-27: Basic data and design criteria of the sand trap outlet gantry crane

Rail track elevation	1263.8 masl
Total rail track length approx.	110 m
Hoisting capacity main hoist	approx. 15 tons
Hoisting speeds main hook:	
normal speed	1.6 m/min
creeping speed	0.3 m/min
Hoisting capacity auxiliary hoist	approx. 5 tons
Hoisting speeds auxiliary hook:	
normal speed	4 m/min
creeping speed	0.5 m/min
Gantry travelling speed	
normal speed	25 m/min
creeping speed	0.6 m/min
Trolley travelling speed	
normal speed	15 m/min
creeping speed	0.6 m/min
Crane control	
from an enclosed cabin fixed to the gantry crane frame	
Design criteria	
crane structure	FEM class A3
crane mechanism	FEM class M2

7.4.5 Bottom outlet

7.4.5.1 Bottom outlet submerged radial segment gate

At the bottom outlet located between the sand trap inlets and the spillway, a submerged radial segment gate will be installed. The radial segment gate will be used for emptying the reservoir or as additional discharge capacity in case of high floods.

The gate is manufactured with class S355J0 structural steel according to DIN EN 10025 and are designed according to DIN 19704.

Two servomotors connected to the gate arms operate the submerged radial gate. The design and capacity of each servomotor considers the exceptional load case, to hold the complete radial gate even with one failing servomotor. Additionally, the gate and the hydraulic hoist will be designed to allow permanent operation of the gate at all partial openings.

The hydraulic power unit will be located together with the local control boards in the dam control building close to the gated spillway.

In the event of main power failure, power supply to the hydraulic unit will be provided by a diesel-powered generator located at the switchyard. As an additional backup system, the hydraulic unit is equipped with a hand pump to move the gate. This hand pumps are installed at the hydraulic power unit.

The trunnion bearings for the gate are of spherical bronze bush type to allow more flexibility of the structures. Both trunnion bearings are self-lubricating.

The frame consists of a sill and top beam profiled to be tangential to the spillway surface, side frames consisting of a combined seal seat, and track for the leaf side guide wheels. The seal seats/track surfaces are machined to the tolerance permitted in DIN 19704. All seal seats are of stainless steel, machined and polished.

Basic data and design criteria of the radial gate are:

Table 7-28: Basic data and design criteria of the bottom outlet radial gate

Type	Submerged Radial Gate
Number of gates	1
Clear width of opening	6.5 m
Clear height of opening	7.0 m
Sill elevation	1232.5 masl
Maximum Operational Level	1260.0 masl
Maximum Flood Level	1263.8 masl
Crest elevation of the piers	1265.0 masl
Max. operation load:	All hydraulic loads, dead weights and friction loads
Hoist	Oil-hydraulic
Gate normal operation speed	0.3 m/min

7.4.6 Power Intake

7.4.6.1 Emergency intake gate

The power intake will be equipped with a roller gate, located at the headrace tunnel intake.

The intake gate will serve as emergency closing devices in case of failures or damages in the waterways, extraordinary pressure difference at the trash rack, etc.

The gate seals upstream and has ballast for gravity closure against full flow with adequate factors of safety against hydraulic forces and friction. The gate shall operate without failure or vibration under the most adverse combination of hydraulic flow and mechanical resistance due to friction, debris or other causes. Operation of the emergency intake gate is by a hydraulic servomotor served by an independent hydraulic power unit.

All gate controls and the hydraulic unit will be housed in the control building at the dam. Control of the gates will be possible by remote control from the powerplant and by an emergency closure control button in the control building at the dam.

Gates will be designed, fabricated and erected in accordance with DIN 19704 as applicable.

Table 7-29: Basic data and design criteria of the intake gate

Type	Fixed Roller Gate
Number of gates	1
Clear width of opening	5.5 m
Clear height of opening	6.75 m
Sill elevation	1242.0 masl
Maximum Operational Level	1260.0 masl
Maximum Flood Level	1263.8 masl
Operation	Open against max. differential head Close at max. flow
Max. operation load	All dead weights and friction loads
Seal position:	upstream
Operating mechanism:	Hydraulic Hoist

7.4.7 Tailrace Surge Tank / Draft Tube Outlet

7.4.7.1 Draft tube stop logs gates

For maintenance and repair works on the turbines, the tailrace surge tank and tailrace tunnel need to be sealed off by placing stop logs in the corresponding slots located on the upstream side of the tailrace surge tank to allow dewatering of the spiral case and the draft tube itself.

The final dimensions of these stop logs need to be coordinated with the hydraulic design of the draft tube and therefore with the turbine supplier. The design pressure of the stop log gates will correspond to the maximum water level in the tailrace surge tank. In total, three sets of stop logs will be provided for the three generating units, to be able to dewater all units at the same time.

The draft tube stop logs will be installed and removed under equalized pressure conditions using an electrically driven traversing monorail hoist with a capacity of 10t above the slots for the stop log gates at the top of the tailrace surge tank.

Table 7-30: Main data of draft tube stop logs

Type	Sliding / Fixed Roller Type
Clear width of each gate	3.9 m
Clear height	3.5 m
Number of stop log sets	3
Number of elements per set	2
Max water level in the tailrace surge tank	1064.6 masl
Sill elevation (approx.)	1046.5 masl
Operation	Open/Close under balanced no-flow condition
Lifting Device	traversing monorail hoist

7.4.8 Tailrace tunnel outlet

7.4.8.1 Tailrace tunnel stop log gate

For maintenance, inspection and repair works on the tailrace tunnel and tailrace surge tank, the tailrace tunnel needs to be sealed off by placing stop logs in the corresponding slots at the tailrace tunnel outlet structure.

The tailrace tunnel stop logs will be installed and removed under equalized pressure conditions using a mobile crane, running on the operating platform level 1065.5 m a.s.l.

Table 7-31: Main data of tailrace tunnel stop logs

Type	Sliding / Fixed Roller Type
Clear width of each gate	7.0 m
Clear height	8.0 m
Number of stop log sets	1
Number of elements per set	3
Max Water Level	1065.0 masl (PMF)
Sill elevation (approx.)	1057.5 masl
Operation	Open/Close under balanced no-flow condition
Lifting Device	Mobile crane

7.4.9 Diversion tunnel

7.4.9.1 Permanent diversion tunnel stop logs

In order to close the diversion tunnel intake once the construction of the dam is accomplished, one set of permanent stop logs will be provided. These stop logs will be installed after finalization of the construction works at the dam and are not intended to be removed again, therefore, these stop logs do not need to be made of steel structures providing the possibility for removal. As a cost-efficient alternative, simple concrete stop logs will be used for this purpose.

Basic data and design criteria of the permanent diversion tunnel stop logs are:

Table 7-32: Basic data and design criteria of the diversion tunnel stop logs

Clear width of diversion tunnel inlet	approx. 8.0 m
Clear height of diversion tunnel inlet	8.0 m
Number of stop log sets	1
Number of stop log elements per set	4
Height of one stop log element	approx. 2.0 m
Sill elevation of stop logs	1230.0 masl
Maximum Flood Level	1263.8 masl
Placing stop logs	Balanced water head conditions after dam construction
Max. operation load	All dead weights and friction loads
Sealing:	At downstream side
Lifting device:	N/A

7.5 Design of Electrical Equipment

7.5.1 General

This chapter describes the feasibility design of the electrical equipment of the Sharmai power station to be installed at the major project structures such as powerhouse cavern, switchyard and dam area (intake, spillway, etc.). The design concept is based on the assumption to interconnect the Sharmai power station with its corresponding 220 kV switchyard to the 220 kV high voltage grid of PESCO. A double-circuit 220 kV transmission line shall interconnect the Sharmai switchyard with the 220 kV Chakdara substation.

The switchyard at the Sharmai power station will consist of a 220 kV switchyard with three transformer bays for the connection of the three units via their step-up transformers, two line bays for the connection of the transmission line to Chakdara and one bus coupler bay.
The layout of the 220 kV switchyard will be based on a double-busbar scheme, enabling the reliable operation of the power plant.

The required black-start capability of the power plant and the emergency power supply are provided at the power house, switchyard, weir and power intake by installation of stand-by diesel generator sets.

7.5.2 Single Line Diagram

7.5.2.1 Main high-voltage supply scheme for powerhouse and switchyard

The single line diagram of 220/11/kV voltage level is included in the drawing album; the provided functionality is described below and attached hereunder:

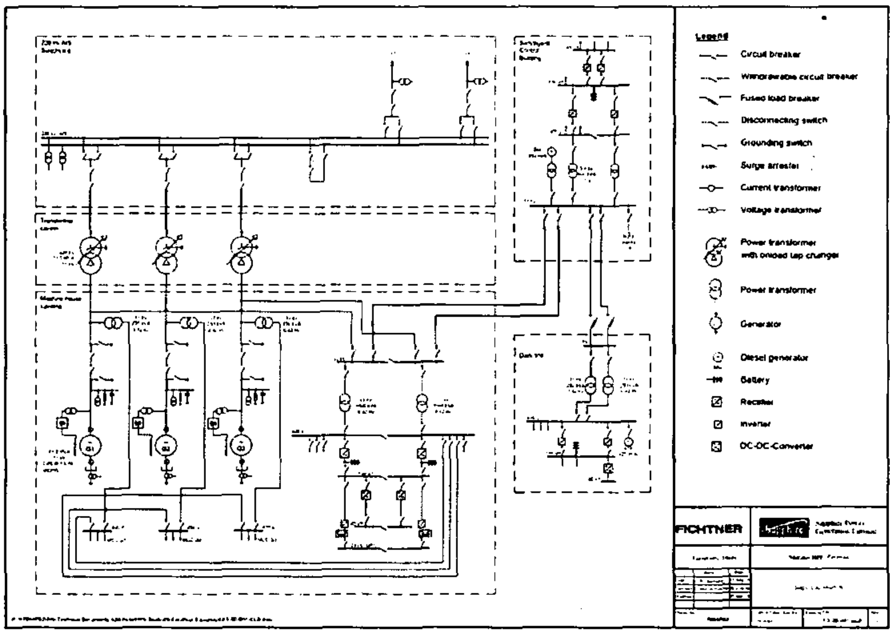


Figure 7-30: Main Single Line Diagram

The applied connection scheme between the generators and their respective main step-up transformers shall be of conventional arrangement, with generator circuit-breaker and tap-offs to the excitation transformer and unit auxiliary transformer. Additional tap-offs to a common 11 kV switchgear will be provided at unit No. 1 and unit No. 3.

As main step-up, oil-immersed, water cooled three-phase, two winding transformers with on-load tap changers will be foreseen. They will be installed in the transformer cavern, adjacent to the powerhouse cavern. The transformer terminals will be suitable for the connection of isolated single-phase bus ducts on the 11 kV side. The 220 kV terminals will be equipped with connection boxes for the connection of 220 kV XLPE-type cables. The 220 kV cables will be run through the cable galley to the surface, where the transition from the 220 kV cables to short overhead interconnection lines will be done. The final connection to the 220 kV air insulated (AIS) switchyard will be realized, as bevor mentioned, with three overhead lines including intermediate towers.

The 220 kV switchyard will have a double busbar scheme ensuring reliability and flexibility during normal and also during exceptional operating conditions. The 220 kV switchyard will be equipped with three transformer bays, two bays for the 220 kV overhead lines and one busbar coupling bay. The lay-out and arrangement will allow an extension for the installation of shunt reactors in case these are required for the grid stability and will allow a future connection of up to two 220 kV transmission line bays.

The 11kV indoor switchgear, located in the powerhouse cavern, will supply power station services within the powerhouse cavern, in the 220 kV AIS switchyard and for dam site consumers. The 11 kV switchgear will be connected to bus duct section of unit 1and 3 leading from the generator circuit breaker to the step-up transformer.

The 11 kV common system of the power station will consist of a 11 kV indoor switchgear in the powerhouse cavern, fed from tie-offs from bus ducts leading from the generator circuit breaker to the step-up transformer as mentioned above. One 11 kV indoor-type switchgear will be located in the control building of the 220 kV switchyard, a further 11 kV switchgear will be located in the control building at the dam site. The 11 kV switchgear at the switchyard will be connected by 11 kV XLPE-type cables to the 11 kV switchgear in the powerhouse, the 11 kV switchgear at the dam site will be fed through the switchgear at the switchyard by a 11 kV overhead transmission line.

7.5.2.2 Station service supply diagram (powerhouse cavern)

The power supply to the 400 VAC main distribution board of the powerhouse cavern shall be provided through the two station service transformers 11/0,42 kV with a rating of approx..1000 kVA, connected to the 11kV indoor switchgear. This scheme will provide sufficient redundancy for the station service and unit auxiliary power supply. Any fault in one generating block will not affect the other one and vice versa. The size of station auxiliary transformer will be chosen so that each one will be able to provide the complete station auxiliary power; the station auxiliary transformers will be connected to the 400 V AC main distribution board in the powerhouse cavern

Starting of the units can be obtained from the 220 kV grid supply through the main step-up transformers of unit No. 1 and unit No. 3 which will supply the 11 kV indoor switchgear and the corresponding station service transformers.

In the case of complete power failure an 800 kVA emergency diesel generator set shall provide the emergency power supply to the power station and shall also provide energy for a black start of the power plant. The emergency diesel generator set will be located at the control building of the 220 kV switchyard and will be connected through a 0,42/11 kV step-up transformer to the 11 kV switchgear in the switchyard.

Emergency power to the powerhouse cavern will be supplied through the 11 kV system and the station service transformers to the 400 V AC main distribution board.

The 110 V DC system shall provide power for the control and protection and shall comprise two twin rectifiers, two lead-acid batteries, a main DC distribution board with two bus sections and bus tie-breaker and other sub-distribution boards as required.

The Safe AC system, consisting of two static inverters connected to the 110 V DC system and a distribution board, shall supply the emergency lighting and supply essential consumers of the control system, such as computers, monitors and printers. Static by-pass switches between the 400 V AC main distribution board and the 230 V Safe AC distribution board shall also be provided.

Under normal condition supply of auxiliary power will be through one of the two 100% station auxiliary transformers feeding the 400 V AC main distribution board. In case of power outage, the essential station auxiliaries (about 50% of total auxiliaries) may be fed from the emergency diesel generator set, in case of complete power failure (also used in case of black-start).

Unit related auxiliaries will be fed through the 400 V unit distribution boards, each fed from the relevant unit auxiliary transformer or 400 V AC main distribution board.

For normal starting of a turbine-generator unit, the feeding of the 400 V AC main distribution board and all station auxiliaries will be from the 220 kV grid supply through the unit step-up transformer, while the generator circuit-breaker is open.

Synchronization of a turbine-generator unit will be performed at the generator circuit-breaker.

7.5.2.3 Auxiliary power supply in the 220 kV switchyard.

The 11 kV indoor switchgear, located in the switchyard control building, will supply auxiliary power for the switchyard.

The power supply to the 400 VAC main distribution board of the switchyard shall be provided through the two station service transformers with rating 400 kVA, 11/0,42 kV connected to the 11kV indoor switchgear.

The 110 V DC system shall provide power for the control and protection and the switchgear control system and shall comprise two twin rectifiers, two lead-acid batteries, a main DC distribution board with two bus sections and bus tie-breaker, one DC sub-distribution board in the 220 kV AIS switchyard building and other sub-distribution boards as required.

The Safe AC system, consisting of two static inverters connected to the 110 V DC system and a distribution board, shall supply the emergency lighting and serve essential consumers of the switchyard control system, such as computers, monitors and printers. Static by-pass switches between the 400 V AC main distribution board and the 230 V Safe AC distribution board shall also be provided.

Emergency power supply for the switchyard will be provided from the emergency diesel generator set as indicated in Chapter 7.5.2.2.

7.5.2.4 Auxiliary power supply at dam site

Power supply to the dam site, including power intake, spillway and bottom outlet, will be via a double-circuit 11 kV transmission system, connected to the 11 kV indoor switchgear in the switchyard control building.

Two 250 kVA service transformers shall be provided to supply the dam site consumers. A 11 kV switchgear with fused load-breakers will interconnect the auxiliary transformers with the 11 kV over head transmission lines. The service transformer shall be connected to the 400 V distribution board located in the dam site building.

Emergency power supply for the dam site will be provided from the emergency diesel generator set with a capacity of approx. 125 kVA, located in a container adjacent to the dam site building.

7.5.2.5 Auxiliary supply scheme

Powerhouse
Under normal condition supply of auxiliary power will be through one of the two 100% station auxiliary transformers feeding the 400 V main distribution board. Alternatively, station auxiliaries (about 50% of total auxiliaries) may be fed from the synchronous emergency diesel generator set, in case of complete power failure (also used in case of black-start). Unit related auxiliaries will be fed through the 400 V unit distribution boards, each fed from the relevant unit auxiliary transformer or 400 V main distribution board.

For normal starting of a turbine-generator unit, the feeding of the 400 V main distribution board and all station auxiliaries will be by using the 220 kV grid supply through the unit step-up transformer and the unit auxiliary transformer, while the generator circuit-breaker is open.

Synchronization of a turbine-generator unit will be performed at the generator circuit-breaker. UPS systems will comprise the 2 x 100% redundant 110 VDC, 48 VDC-, 230 V safe AC-systems and one emergency diesel generator system.

Dam Site

The main components will be:

- one 11 kV switchgear with fused load-breakers
- two auxiliary transformer
- one 400 V distribution board
- two 100% UPS systems, battery backed-up (mainly for local control)
- one synchronous emergency diesel set at 400 V.

7.5.3 Electrical equipment within the powerhouse

7.5.3.1 Main generating equipment

7.5.3.1.1 Characteristics of generator

All windings of stator and rotor will be provided with a class „F" insulation system. As the long-term performance of the insulation system is affected by the maximum operating temperature of the windings, the rated output of the generators will be related to a temperature rise corresponding to class „B" insulation. As per IEC 60034 the permitted temperatures respectively temperature rises are:

Table 7-33: Insulation Classes for Generators

Part of machine	Class B Insulation	Class F Insulation
Stator windings maximum temperature rise (measured by embedded temperature detectors ETD)	85 K	110 K
Rotor windings maximum temperature rise (measured by winding resistance)	90 K	110 K
Maximum air-inlet temperature	40 °C	40 °C

K = degree Kelvin (used for temperature differences)

The closed-cycle air-cooled generators will be equipped with air-water heat exchangers, connected to the plant cooling water system. For all load conditions maximum air temperature will be limited to 40°C.

The power and speed of the generators are dictated by the turbine, with its calculated output at the shaft coupling at design heads and design flow. Considering the respective turbine power output, a typical generator efficiency of approx. 98% and a power factor of 0,80 lagging (which allows the generation of the necessary reactive power for voltage regulation at the 220 kV grid), the respective generator design data result as follows:

Table 7-34: Design Parameter for Generator Design

Turbine power			
			Remarks:
P_{rated}	MW	50,4	3 units running at maximum design discharge (at full capacity nominal). Minimum power delivered to each generator at class B temperature rise!
P_{max}	MW	52,9	Only 1 unit is running. Maximum power delivered to a (each) generator at class F temperature rise!
Nominal speed	rpm	428,6	
Rated frequency	Hz	50	
Generator power			
Generator efficiency	%	97.6	
Nominal power factor	-	0,80	Lagging (0,90 leading)
$Ps_{\_rated}$	**MVA**	**61,5**	Minimum power each generator at class B temperature rise must be able to deliver!
$Ps_{\_max}$	**MVA**	**64,5**	Maximum power a (each) generator at class F temperature rise must be able to deliver!

The rated generator voltage will be 11 kV with a regulating range of +/- 5%, which is a typical standard voltage, appropriate for generators of this size and widely used in Pakistan (the recommended voltage range for above generators is 8 to 15 kV). However, for optimization of the generator and bus bar design, the final selection may be left open to the supplier.

The W41 generator bearing arrangement is suitable for vertical-shaft generators of above mentioned rated output. The combined guide and thrust bearing will be arranged above the rotor, the lower guide bearing will be arranged below the rotor. The thrust bearing will be provided with an automatic high-pressure oil-injection, to reduce wear on the babbit-metal coated segments during start-up and shut-down of the unit.

To reduce the run-out time of the unit an air-pressure operated braking system will be provided. The air-brake system will be combined with an oil-pressure jacking system, to allow the jacking up of the rotor for maintenance purposes. For the dimensioning of the civil layout the following generator dimensions were estimated:

Rotor diameter	approx..	3400 mm
Outer stator diameter	approx..	5350 mm
Shaft length	approx..	7600 mm
Weight of complete rotor	approx..	139 tons

Due to the limitations of transport dimensions and weights, the stator housings will be divided and delivered in sections and the winding at the joints will be completed on site. The rotor will be assembled completely at site, including stacking of the rotor rim and fixing of the poles.

7.5.3.1.2 Neutral grounding

In the case of earth-fault current flowing through the generator stator winding the current will have to be limited to a value which will not damage the winding before the protection operates.

The generator neutral will be grounded via a dry-type single-phase power transformer. The secondary winding of this transformer shall be loaded with a resistor. Detailed design of the neutral grounding will be subject to the Tender Designs stage of the Project.

7.5.3.1.3 Fire protection

A CO_2 or inert gas fire protection system will be provided for the generators. The gas cylinder batteries will be located in the vicinity of the generator barrels and provisions for a gas evacuation system will be made. The advantage of a CO_2 gas system would be that in case of replacement or refilling the gas cylinders could be obtained in the local market whereas inert gas cylinders probably would have to be imported.

The alternative of a water spray fire protection system was initially considered but was discarded as damages to the insulation system of the generators cannot be excluded.

7.5.3.1.4 Excitation system

According to the state-of-art the excitation system will be of the fully static-type including digital type programmable automatic voltage regulator, thyristor rectifiers and field-suppression equipment.

The automatic voltage-regulator will be equipped with automatic and manual channels with follower allowing bump less change-over from automatic to manual control in case of fault in the automatic regulation. It will further be furnished with the necessary protection and limitation devices and with a reactive power regulator. The regulating range of the generator voltage shall be +/- 5%.

The excitation energy will be branched-off the generator main bus ducts through a dry-type excitation transformer.

For the initial excitation during start-up, a field-flashing equipment fed from the 400 V station supply and a back-up field-flashing from the DC system will be provided. Provision for coping with power swing in the system will be made (power system stabilizer PSS).

7.5.3.1.5 Generator bus ducts

For the connections between the generators, excitation transformers, generator circuit-breakers, generator neutral, unit auxiliary transformers and

11 kV switchgear, standardized single-phase air-insulated bus ducts will be provided. This type of bus duct offers the advantage to eliminate short-circuits risks and guarantees for high grade of personnel safety, since all live conductors remain inaccessible and the touchable enclosures are grounded.

The bus duct system will incorporate the required current transformers for protection and metering. The voltage transformers as well as the protective capacitors and surge arresters will be arranged in a separate fully enclosed cubicle, directly connected by tap connections to the main bus ducts.

Excitation transformer and unit auxiliary transformer will also be installed in metal-enclosed cubicles with isolated tap connections to the main bus ducts.

Enclosures and conductors of the bus ducts will be of aluminum. For the connection to the equipment suitable flexible connections will be provided. Considering the distance between generator and step-up transformers expansion joints will be considered.

Considering the generator voltage of 11 kV and the expected voltage regulation range of ± 5%, the characteristics of the proposed bus duct system will be as follows:

Table 7-35: Design Parameter for Generator Bus Ducts

Number of bus bar systems	3
Number of phases of each system	3
Rated voltage (between conductors)	11 kV
Maximum system voltage (design)	12 kV
Frequency	50 Hz + 3 %
One minute power frequency withstand test voltage (minimum)	38 kV (rms)
Impulse withstand test voltage	95 kV (peak)
Rated current for main bus ducts	3400 A
Rated current for bus duct tap connections to excitation transf. and unit auxiliary transf. Rated current for bus duct tap connections to the 11 kV indoor switchgear	100 A 300 A
Rated short-circuit current	48 kA, final value to be decided in tender phase

7.5.3.1.6 Generator circuit-breakers

Main data of generator circuit-breaker:

Table 7-36: Design Parameter for Generator Circuit Breakers

Number of circuit-breakers	3
Number of phases of each system	3
Nominal operating voltage	11 kV

Highest system voltage	12 kV
Frequency	50 Hz + 3 %
One minute power frequency withstand test voltage (minimum)	38 kV (rms)
Impulse withstand test voltage	95 kV (peak)
Rated continuous current	3400 A
Rated symmetrical interrupting capability (minimum)	48 kA (rms)
Rated short circuit making current capability (minimum)	125 kA (rms)
Type of circuit-breaker	SF_6
Type of cooling	air, natural
Ratings of disconnecting and grounding switches	consistent with circuit-breaker ratings

7.5.3.2 Step-up transformers

The rating of each oil-immersed closed three-phase transformer will be 61,5 MVA. This rating takes into consideration the maximum capacity of the generators, as per Section 7.5.3.1.1. To be able to adapt to the 220 kV grid requirements of active and reactive power, an on-load tap-changer with a regulating range from -15% to +15%, in steps of 1,25% will be provided. The detailed requirements of the on-load tap-changer will be investigated in the tender design stage once corresponding load-flow studies for the grid and the power plant are available. Because of the location of the step-up transformer in the transformer cavern, the cooling-type of the transformers will be OFWF (oil forced water forced cooling). Short-circuit current levels for the primary and secondary windings will be decided during tendering stage.

The transformer terminals will be suitable for the connection of single-phase bus ducts on the 11 kV side. The 220kV terminals will be equipped with cable connection boxes and suitable for the connection of 220 kV XLPE-type cables as interconnection to the 220 kV AIS switchyard.

Each step-up transformer will be installed in its own concrete compartment in the transformer cavern.

To facilitate the movement of the transformers during installation, swivel type wheels will be provided.

The step-up transformer oil collecting pits will be connected to a common water-oil collector with oil separator.

7.5.3.2.1 Main transformer characteristics

Table 7-37: Design Parameter of Three Phase Transformers

Number of three-phase transformers	3
Type	three-phase, two windings
Rated output of three-phase transformers	61,5 MVA
Frequency	50 Hz

Type of cooling	OFWF
Rated voltage:	
High voltage winding	220 kV
Low voltage winding	11 kV
Rated power frequency withstand voltage (rms value) HV side	460 kV
Rated lighting impulse withstand voltage (peak) BIL HV side	1050 kV
Type of tap changer	on-load tap-changer
Range of tapping	± 12 x 1.25% = ± 15%

7.5.3.2.2 Explosion protection

To eliminate risks such as damages to the transformer cavern structure, damages to auxiliary equipment located in the transformer cavern and prevent risks to humans working or passing by in the transformer cavern each step-up transformer will be equipped with tank pressure relief system, located in the transformer cell. The tank pressure relief system shall consist of a shock absorber and a decompression chamber, oil-gas separation system and gas evacuation pipes.

7.5.3.2.3 Fire protection

An automatically operated fire fighting water-deluge system will be provided for all three-phase unit step-up transformers.

7.5.3.3 Unit auxiliary transformers: 11/0.4 kV

Three three-phase transformers will be provided for unit auxiliary power supply. The rating of each dry-type (cast-resin) transformer will be 250 kVA. The voltage ratio will be 11 / 0.42 kV. Each transformer will be provided with an off-load tap selector having a regulating range from -5% to +5%, in steps of 2.5%. The cooling type of the transformers will be AN.

The 11 kV terminals will be suitable for the connection to the tap-off from the generator bus ducts.

The three-phase unit auxiliary transformers will be installed in metal-clad protection housings within the powerhouse below the generator bus ducts. To facilitate the movement of the transformer during installation, swivel type wheels will be provided.

7.5.3.4 Station auxiliary transformers: 11/0.4 kV

Two three-phase transformers will be provided for auxiliary power supply of the power station. The rating of each dry-type transformer will be 1000 kVA, where each transformer will be suitable to feed the total auxiliary power demand of the power station. The voltage ratio will be 11 / 0.42 kV. Each transformer will be provided with an off-load tap changer having a regulating range from -5% to +5%, in steps of 2.5%. The cooling type of the transformers will be AN.

The 11 kV terminals will be connected to the 11 kV indoor switchgear with 11 kV insulated XLPE-type cables. The 0.4 kV terminals will be connected to the 0.4 kV Main Distribution Board, using phase-segregated bus ducts.

The two three-phase station auxiliary transformers will be installed in metal-clad protection housings in the busbar galleries of unit No. 1 and unit No. 3 below the generator bus duct sections between generator circuit breaker and step-up transformer. To facilitate the movement of the transformer during installation, swivel type wheels will be provided.

7.5.3.5 11 kV Switchgear in the powerhouse cavern

The 11 kV switchgear will be located within the powerhouse cavern. The 11 kV indoor switchgear will be of the steel enclosed type and will comply with the requirements of latest edition of IEC. The switchgear will be complete with vacuum circuit-breakers, dry-type instrument transformers, earthing arrangements, instruments and protective relays.

The 11 kV switchgear will consist of the following cubicles:

- 2 cubicles for the connection to the generator bus-duct systems of unit No. 1 and unit No. 3
- 2 cubicles for the connection of the station auxiliary transformers
- 2 cubicles for the connection to the 11 kV switchgear in the switchyard control building
- 1 spare cubicle
- 1 bus coupler cubicle.

The technical characteristics of the 11 kV switchgear will be as follows:

Table 7-38: Design Parameter of 11 kV Switchgear at Powerhouse

• Rate operation voltage	11 kV
• Maximum operation voltage	12 kV
• Rated power frequency withstand voltage	28 kV (rms value)
• Rated lighting impulse withstand voltage	95 kV (peak)
• Rated short-circuit breaking current	50 kA for 1 second
• Rated current for bus bars and feeders	800 A

7.5.3.6 Auxiliary electrical equipment

7.5.3.6.1 400 V AC auxiliary power supply in the power cavern

The auxiliary power requirements of each unit will be provided through the three unit distribution boards, each one fed from the relevant unit auxiliary transformer or from the 400 V main distribution board. The 400 V main distribution board itself will be fed from either of the two station auxiliary transformers, rated 1000 kVA each, which are connected to the 11kV indoor switchgear. The 11 kV switchgear itself will be fed from the generator bus system of unit No. 1 and 3. Interconnections with suitable interlocks will prevent parallel operation of the station auxiliary transformers and will permit power transfer from one station auxiliary transformer to the other in case of failure.

To provide power for the general station services during stand-still of the power station, the main distribution board will receive power via the 220 kV network, step-up transformer, 11 kV indoor switchgear and station service transformer. In case of a complete power failure, one 800 kVA emergency diesel generating set, located at the switchyard control building and connected via the 11 kV system to the main distribution board in the power house cavern, will provide the required emergency power.

7.5.3.6.2 Uninterrupted power systems

The UPS systems of the power cavern and the switchyard area will comprise the 2 x 100% redundant 110 VDC systems, the 24 VDC systems, the 48 VDC systems (if required), a 400 V Safe AC-system in the switchyard control building and one emergency diesel generator system:

<u>110 V DC Systems in the switchyard control building</u>
The 110 V DC system, will provide power for the electrical protection systems, the station DCS control equipment located in the switchyard area and for the 220kV switchyard and 11 kV switchgear control and will comprise the following components (the final capacities of which will be agreed during final design):

* Two rectifiers of 110 V DC each 100%
* Two lead-acid type batteries each 100%
* A main distribution board with two bus sections and bus-tie breaker and sub-distribution boards as necessary

<u>24 V DC Systems</u>
The 24 V DC power supply for the station main DCS will be provided using suitable number of redundant DC/DC converters connected to the 110 V DC distribution boards.

<u>48 V DC Systems (if required)</u>
The 48 V DC power supply, mainly used for the communication systems, power-line carrier system (PLC), fiber-optical type communication and telephone system, will be provided using suitable number of redundant DC/DC converters connected to the 110 V DC distribution boards.

<u>230 V Safe AC Supply in the switchyard control building</u>
The 400/230 V safe AC supply will provide safe power to the control room equipment of the switchgear control system (PC's, monitors, printers, etc.)

and for the emergency lighting system. It will comprise the following components:

- two inverters 110 V DC/230 V AC each 100%, connected to the 110 V DC main distribution and static by-pass switches, connected via isolating transformers to the 400 V main distribution board (normal voltage)
- one main distribution board and sub-distribution boards as necessary.

The definitive capacity of the inverters will be determined during final design at tender phase.

Diesel Generator Set
The 400 V emergency diesel-generator set will be required to provide the necessary emergency and black-start power supply for the power house cavern and for the switchyard area. The emergency diesel generator set will be located at the switchyard control building and connected via the 11 kV system to the main distribution board in the power house cavern. Details on the emergency diesel generator set are included in Chapter 7.5.4.5.2.

7.5.3.6.3 Fire detection system

A decentralized fire alarm system with detection and release function will be provided for the entire power station, including power house cavern, switchyard control building and control building at the dam site / intake. The system will consist of one central unit (main fire alarm panel) supervising the sub-units located in the different areas of the powerhouse.

7.5.3.6.4 Telephone system

The entire power station will be equipped with a telephone system consisting of a main exchange. This system will take over the internal and external telephone traffic of the power station area.

The system will enable the telephone communication for at least 40 internal subscribes and 5 external lines via OPGW.

7.5.3.6.5 Electrical workshop and laboratory

A suitable equipped workshop and laboratory for maintenance and repair of the electric and electronic equipment will be located in the powerhouse, i.e. in the administration building; if required, another floor in the administration building can be added.

7.5.3.7 Electrical protection system

General
All electrical protection systems will be of the digital (numerical) type and will comprise the following sub-systems:

Generator Protection System

The following list indicates the required protection functions/devices:

- 21 G Minimum impedance protection
- 46 Negative phase sequence protection
- 49 Thermal winding monitoring
- 49 S Stator thermal overload protection
- 32 Reverse power protection
- 81 N Over- and under-frequency protection
- 27 / 59 Over- and under-voltage protection
- 78 Loss of synchronization protection
- 40 Field failure protection
- 64 S (and 59N) Stator ground-fault protection 95% and 100%
- 64 R Rotor ground fault protection
- 87 G Generator differential protection
- 87 GT Generator-transformer and unit aux. transformer differential protection
- bearing protection device
- vibration monitoring.

Step-Up Transformer Protection

In addition to the generator protection, the unit step-up transformers will each be protected by a Buchholz relay and the following protection devices:

- 51 TN Transformer neutral over-current protection
- 50/51 Inverse-time over current protection
- 87 T Transformer differential protection
- transformer pressure-rise and gas detection device
- transformer oil-level device.

Station Service Transformer Protection

The station service transformer will be protected by the following protection devices:

- 50/51 Inverse-time over current protection
- 49 Thermal overload protection.

Unit Auxiliary Transformer Protection

The unit auxiliary transformer will be protected by the following protection devices:

- 50/51 Inverse-time over current protection
- 49 Thermal overload protection.

11 kV Switchgear

Protection comprises 3-phase over-current- and earth-fault relays in the incoming feeders. The over-current relay will be of the inverse-time type with instantaneous tripping set at a high level. The relays will be installed in the relay compartment of the 11 kV panels.

400 V Switchgear
Protection of the 400 V system will be provided by magnetic thermal trip units mounted on the circuit-breakers. In the case of fuse isolators combined with contactors only thermal overload protection will be required. Under-voltage relays on each busbar will supply the criteria for the automatic change-over device for the different supply sources.

7.5.3.8 Control and monitoring system

For reliable, efficient and safe operation of the power station a monitoring and control system will be provided suitable for supervisory, control and monitoring of each individual unit as well as common equipment in the powerhouse, switchyard, gate chamber, dam and power intake.

Due account shall be taken of the requirement for high system availability by implementing a functionally distributed and hierarchical structure, resulting in the following concept:

- The automation technology shall be distributed with decentralized systems and equipment. In this way, the impact of faults in one unit or in a component will be as small as possible.
- The structure of the I & C system shall match to the power station structure. For example, control and signal loops for one unit shall be allocated to separate cubicles or equipment racks.
- Functionally hierarchical structure: in the event of failure of a supervisory level, it shall be possible for the operator, to intervene in the lower level(s) from the control room and also from the local operator panel. Individual open- and closed-loop functions shall be implemented on the lowest automation level. These consist of software modules, which may be controlled optionally from the higher automation level or manually.

The topological diagram of the SCADA is included in the drawing album.

A modular, screen-prompted control system will be used, with references documented for comparable power stations. The control system will contain all functions needed for operating the entire hydropower plant, such as:

- data acquisition and signal processing
- plant control, unit control, group control, automatic equipment control
- monitoring, annunciation
- control and monitoring via VDUs
- data archiving
- communication bus
- time synchronization with GPS-clock
- diagnosis
- engineering tools
- complete system documentation.

The control system will be connected to the Process Stations using redundant serial automation busses (redundant data highway). For remote monitoring of the dam site and other remote areas, an adequate communication link, using fiber-optics, e. g. ADSS cable (All-Dielectric Self-Supporting Aerial), between remote equipment and the powerhouse will be used. The cable will be hanged along the 11 kV overhead transmission line between the switchyard control building and the dam site building.

Provisions will be made to adapt the DCS for future remote monitoring from a remote grid dispatch center.

Operating and Monitoring Facilities
The station shall be operated from a Central Control Room (CCR) located in the powerhouse, i.e. in the administration building.

Within the CCR, two operator's workstations with two monitors will be foreseen, from which the operators will supervise, operate and monitor the units. Also, a further workstation will be provided with one monitor for engineering works at the system. The engineering workstation also will provide backup, in case of an operator workstation malfunction. For hard copies printers (monochrome and color) will be connected to the system. Printouts are output on request.

Local Control
As well as normal operation and monitoring from the CCR, further operating panels (HMI – human-machine-interface) are foreseen at each process station. These will be equipped with monitors, so permitting full operation and monitoring of the relevant unit or related components.

Local HMI will be provided for each of the following areas:

- 3 for the unit control systems (unit 1 to unit 3)
- 1 common for
  common electrical systems
  common mechanical equipment
  emergency diesel set
- 1 for the dam site installations (including power intake, spillway and bottom outlets).

Process Stations
Individual PLC (Programmable Logic Controller) based control system will be used for each unit, for common systems. Each PLC will contain all functions required for automatic operation of the assigned tasks, e. g.:

- signal processing (I/O level)
- open- and closed-loop control
- supervision
- control and monitoring via local HMI
- diagnosis
- engineering tools

- complete system documentation.

Each PLC will have redundant CPU's with hot standby function.

Complete manual control of all control system will be made possible from the local control cubicles (located next to the respective equipment) in case of failure of the control computer or for test and maintenance purposes. The system shall be able to perform all functions through standardized hardware and software modules. It shall be scalable to match the plant complexity.

The hardware shall be made up of a small number of different hardware modules, with these being replaceable during plant operation. The system shall be provided with interfaces to commonly used field bus and remote I/O systems.

Metering
For the monitoring of the power generation and power transmission a metering system using fully static energy meters and energy recorders will be provided. For the metering and billing of each of the outgoing lines, one main and one back-up energy meter with minimum accuracy of 0.5 will be foreseen.

7.5.3.9 Communication

An OPGW (optical ground wire) as part of the transmission line will be installed as communication link with a grid dispatch center and for the transmission line protection. An ADSS (all-dielectric self-supporting aerial) cable with fiber optic will be installed for DCS control communication with the process stations at dam site. The system will be extendible for data transmission as SCADA system at a later stage, all necessary provisions will be considered. The system will allow telecommunication from the telephone extension via the telephone system and to the dispatch center and the telephone network and vice versa.

7.5.4 Main Electrical Equipment in the 220 kV Switchyard

7.5.4.1 220 kV AIS switchgear

The 220 kV AIS switchgear will consist of a 1 and 1/2 busbar scheme and will comprise three (3) generator-transformer bays, two (2) line bays for the interconnection with the 220 kV grid and one (1) shunt reactor bay, if required.

Each transformer and line bay will be equipped with a SF_6 circuit-breaker, disconnecting switches, earthing switches and required current transformers. Voltage transformers will be installed in the line bays. The technical characteristics of the switchgear will be as follows:

Table 7-39: Design Parameter of 220 kV AIS Switchgear

Insulation medium (disconnecting circuit breakers)	SF_6
Maximum operation voltage	245 kV
Rated power frequency withstand voltage (rms value), phase to earth across the open switching device, at minimum operating gas-pressure	460 kV
Rated lighting impulse withstand voltage (peak), phase to earth and across the open switching device, at minimum operating gas-pressure	1050 kV
Rated short-circuit breaking current	31,5 kA for 3 second
Rated bus bar current (minimum)	1000 A
Rated current for generator (step-up transformer) and line bays (minimum)	200 A / 800 A /

7.5.4.1.1 Shunt reactor (optional)

If the detailed load flow and stability calculations, to be executed during the tender design phase, show that a shunt reactor is required a suitable three-phase shunt reactor shall be connected to the 220 kV switchgear.

Table 7-40: Design Parameter of 220 kV Shunt Reactors

Type	three-phase type, oil immersed shunt reactor, outdoor type
Number of unit	1
Rated power	To be determined
Frequency	50 Hz
Type of cooling	ONAN
Rated voltage:	
High voltage outdoor bushing	220 kV

7.5.4.2 220 kV Cable link between step-up transformers and AIS switchyard

The connection between the HV terminals of the step-up transformers and the transition to the short overhead lines leading to the corresponding feeders in the 220kV switchyard will be executed with 220 kV XLPE-type cables. The 220 kV cables will be installed in a cable gallery, running from the transformer cavern to the surface in the vicinity of the access road to the access tunnel to the surge tank. The main technical characteristics of the 220 kV cables are as follows:

Table 7-41: Design Parameter of 220 kV cables

Maximum operation voltage	245 kV
Rated voltage (Uo/U)	220 kV
Short duration power frequency withstand voltage (rms value)	460 kV
Rated lighting impulse withstand voltage (peak)	1050 kV

Rated power per 3-phase cable system	61,5 MVA

The 220 kV single-phase cables will be connected to the terminals of the step-up transformers by adequate cable connection boxes. For the connection to the short 220 kV overhead lines suitable cable terminals, located on the platform of the overhead line gantries will be foreseen. Further surge-arrestors for the protection of the 220 kV cables will be provided and located close to the cable terminals.

7.5.4.3 220 kV overhead line to the AIS switchyard

The interconnection of the 220 kV cable terminals and the gantries in the 220 kV AIS switchyard will be realized by three short 220 kV overhead transmission lines with ACSR conductors and earth wires. The overhead lines will also comprise the gantries on the transition platform, and one set of intermediate suspension towers.

The main technical characteristics of the 400 kV overhead connections are as follows:

Table 7-42: Design Parameter of 220 kV overhead lines

Maximum operation voltage	245 kV
Rated voltage (Uo/U)	220 kV
Short duration power frequency withstand voltage (rms value)	460 kV
Rated lighting impulse withstand voltage (peak)	1050 kV
Rated current per 3-phase OHL system	200 A

Details for the connection of the 220 kV overhead line conductors to the terminal gantry will be assessed during tendering stage.

7.5.4.4 11 kV Switchgear in the switchyard control building

The 11 kV switchgear will be located switchyard control building. The 11 kV indoor switchgear will be of the steel enclosed type and will comply with the requirements of latest edition of IEC. The switchgear will be complete with vacuum circuit-breakers, dry-type instrument transformers, earthing arrangements, instruments and protective relays.

The 11 kV switchgear will consist of the following cubicles:

- 2 cubicles for the connection to the 11 kV XLPE-type cables, coming from the powerhouse cavern
- 2 cubicles for the connection of the auxiliary transformers
- 2 cubicle to the connection of the 11 kV overhead transmission line to the dam site switchgear in 2 cubicles for the connection the switchyard control building

- 1 cubicle for the connection of the step-up transformer of the emergency diesel generator
- 1 cubicle for rural power supply
- 1 spare cubicle.

The technical characteristics of the 11 kV switchgear will be as follows:

Table 7-43: Design Parameter of 11 kV Switchgear at Switchyard Control Building

• Rate operation voltage	11 kV
• Maximum operation voltage	12 kV
• Rated power frequency withstand voltage	28 kV (rms value)
• Rated lighting impulse withstand voltage	95 kV (peak)
• Rated short-circuit breaking current	50 kA for 1 second
• Rated current for bus bars and feeders	800 A

7.5.4.5 Auxiliary power supply in the 220 kV switchyard

7.5.4.5.1 400 V AC auxiliary power supply

The auxiliary power requirements of the 220 kV AIS switchyard and the switchyard control building will be fed from the 400 V main distribution board, located in the switchyard control building. The 400 V main distribution board itself will be fed from either of the two auxiliary transformers, rated 400 kVA each, which are connected to the 11kV indoor switchgear in the switchyard control building. The 11 kV switchgear itself will be fed from the 11 kV switchgear in the powerhouse cavern via two 11 kV XLPE-type cables. Interconnections with suitable interlocks will prevent parallel operation of the auxiliary transformers and will permit power transfer from one auxiliary transformer to the other in case of failure.

7.5.4.5.2 Uninterrupted power system

The UPS systems of the switchyard area will comprise the 2 x 100% redundant 110 VDC systems, the 24 VDC systems, the 48 VDC systems (if required), a 400 V Safe AC-system in the switchyard control building and one emergency diesel generator system:

<u>110 V DC Systems in the switchyard control building</u>
The 110 V DC system, will provide power for the electrical protection systems, the station DCS control equipment located in the switchyard area and for the 220kV switchyard and 11 kV switchgear control and will comprise the following components (the final capacities of which will be agreed during final design):

* Two rectifiers of 110 V DC each 100%
* Two lead-acid type batteries each 100%

* A main distribution board with two bus sections and bus-tie breaker and sub-distribution boards as necessary

24 V DC Systems
The 24 V DC power supply for the station main DCS will be provided using suitable number of redundant DC/DC converters connected to the 110 V DC distribution boards.

48 V DC Systems (if required)
The 48 V DC power supply, mainly used for the communication systems, power-line carrier system (PLC), fiber-optical type communication and telephone system, will be provided using suitable number of redundant DC/DC converters connected to the 110 V DC distribution boards.

230 V Safe AC Supply in the switchyard control building
The 400/230 V safe AC supply will provide safe power to the control room equipment of the switchgear control system (PC's, monitors, printers, etc.) and for the emergency lighting system. It will comprise the following components:

- two inverters 110 V DC/230 V AC each 100%, connected to the 110 V DC main distribution and static by-pass switches, connected via isolating transformers to the 400 V main distribution board (normal voltage)
- one main distribution board and sub-distribution boards as necessary.

The definitive capacity of the inverters will be determined during final design at tender phase.

Diesel Generator Set
The 400 V emergency diesel-generator set will be required to provide the necessary emergency and black-start power supply. The emergency energy generating system will include principally:

- one 4 stroke diesel engine / three-phase AC synchronous generator set, mounted on common base complete with flywheel, air cleaners, vibration damper, forced lubricating system, fuel supply system including storage tank and daily tank, radiator type cooling water system.
- exhaust manifold, silencer and piping
- electronic governor system
- 24V DC electric starter, including battery
- generator with nominal voltage of 400 V, output of 800 kVA, power factor 0.9
- brushless excitation system with rotating diodes.
- Step-up transformer 0.4/11 kV, 800 kVA for the connection to the 11 kV switchgear in the switchyard control building.

The unit will start and build-up voltage automatically within 15 seconds. Shut-down of the diesel engines will be by means of fuel shut-off solenoid. As a result from preliminary estimations of the essential loads to be supplied by the diesel generator set, a rated output of 800 kVA will be considered.

The definite rating of the diesel generator set will be determined during the tender phase of the project.

7.5.4.6 Electrical protection system

General

The detailed requirements of the protection systems will be investigated in the tender design stage once corresponding load-flow studies for the grid and the power plant are available.

All electrical protection systems will be of the digital (numerical) type and will comprise the following sub-systems:

220 kV AIS Switchgear Protection

The protection of the 220 kV AIS switchgear includes protection devices for the outgoing lines, the incoming feeders and busbars.

Line Bays

The transmission line bays will be equipped with distance protection relays with at least three stages, over-voltage relays, disturbance recorders and distance fault locators. Inter-tripping and blocking will be carried out via tele protection. Additionally, they will be equipped with three-phase auto re-closing devices suitable for single- and 3-phase auto re-closing operation. Back-up protection will be provided by a directional earth-fault relay and an inverse-time over-current relay with definite-time earth-fault protection. The protection scheme will be closely coordinated with the network owner.

Transformer Feeder Bays

The transformer feeder bays will be equipped with back-up over-current and earth-fault protection and with transformer voltage regulation and supervision.

Busbars

A busbar differential protection scheme with phase-segregated measurement will be provided for the 220 kV busbars.

11 kV Switchgear

Protection comprises 3-phase over-current- and earth-fault relays in the incoming feeders. The over-current relay will be of the inverse-time type with instantaneous tripping set at a high level. The relays will be installed in the relay compartment of the 33 kV panels.

400 V Switchgear

Protection of the 400 V system will be provided by magnetic thermal trip units mounted on the circuit-breakers. In the case of fuse isolators combined with contactors only thermal overload protection will be required. Under-voltage relays on each busbar will supply the criteria for the automatic change-over device for the different supply sources.

7.5.4.7 Switchyard control and monitoring system SCMS

For reliable, efficient and safe operation of the 220 kV switchyard a monitoring and control system will be provided suitable for supervisory, control and monitoring of each individual bay as well as common equipment in the switchyard.

The switchyard control and monitoring system (SCMS) will be integrated in the overall control and monitoring system as described in Chapter 7.5.3.8.

Design principles

The SCMS shall be a digital control and monitoring system to supervise and operate the switchgears in the substation complete in every respect for monitoring and control inclusive all facilities.

SCMS shall be suitable for supervision, operation and maintenance of the complete substation extension including future extensions.

Design and arrangement of the system shall be state-of-the-art based on IEC 61850 for operation under electrical conditions (including electrical discharge and disturbance level) prevailing in high voltage and medium voltage substations, follow the latest modern engineering practice, ensure optimum continuity and reliability of supply and ensure the safety of equipment and the operating staff. The highest degree of uniformity and interchangeability shall be provided.

Design of the hardware and software shall be suitable for all voltage levels used by the Employer to enable a standardized technical concept.

SCMS shall be designed such that personnel without any computer background shall be able to operate the system with ease and shall incorporate user-friendly features without causing undue operational delay.

The whole equipment shall be pre-assembled and pre-programmed at the supplier's workshop. It is understood that all auxiliary facilities / devices and services necessary are to be provided, i.e. for generation of data base, of displays, programming and for testing, adjustments, parameter settings etc., even if not specified in detail.

The whole equipment shall be designed for indoor installation, installed in steel sheet cubicles with hinged frames and glass door having a protection degree as stated in the technical data sheets.

All components shall be suitable for the local climate and environmental conditions.

The SCMS shall be designed for easy modification of hardware and software and for easy extension of the substation. Maintenance, modification or extension of components may not force a shut-down of the whole SCMS. Self monitoring of single components, modules and communication links shall be incorporated to increase the availability and reliability of the equipment and minimize maintenance.

Failure of any component of the system may not force a total system failure.

Principal system architecture

For safety and availability reasons the SCMS shall be based on a decentralized architecture and on a concept of bay-oriented, distributed intelligence.

Functions shall be decentralized, object-oriented and located as close as possible to the process. The main process information shall be stored in distributed databases.

The topological diagram of the SCADA is included in the drawing album.

Principally the architecture of the SCMS is structured in the following levels:

- Remote level:
  System control operations shall be possible from the remote control centers
- Station level:
  System control operations shall be possible from the operator workstations
- Bay level:
  System control operations shall be possible from the bay control and / or protection units (IEDs)
- Apparatus level:
  System control operations shall be possible by local control from the individual equipment

The Project related new substation extension shall be controlled and supervised from up to two remote control centers or from the operator workstations while individual bays are supervised and controlled from the bay level devices in the control cubicles.

Interlocking between the levels shall be possible by customization. SCMS shall prohibit carrying out the control at the same time from different control levels.

It shall be possible to control and monitor the individual bays from bay level, in case the communication link fails. The station wide interlocking shall also be available when the station computer fails.
At station level, the entire substation shall be controlled and supervised from the operator workstations (HMI).

The station level contains the station-oriented functions, which cannot be realized at bay levels, e.g. alarm list or event list related to the entire substation. Communication with remote control centers via a gateway shall also be a part of the station level.

To provide highest reliability, the station computer, the operator workstations (HMI) and the gateway shall work completely independent, meaning retrieving the process data directly from the bay level devices.

A dedicated master clock for the synchronization of the entire system shall be provided for the complete substation. The master clock shall be independent of the station computer and of the gateway, and shall synchronize all devices via the station bus. The deviation of the different internal clocks shall not be more than 1 ms.

The master clock shall be synchronized by a satellite receiver (GPS). The master clock in the substation shall be battery buffered.

Data transmission between the devices on station and bay level shall take place via the station bus, realized by using fiber-optic cables in a ring configuration, thereby guaranteeing disturbance free communication.

To increase system performance and availability, the system shall support several physically separated networks for the station bus e.g. separate networks for different voltage levels.

At bay level, the bay and / or protection units (IEDs) shall provide all bay level functions regarding control, monitoring and protection, inputs for status indications and outputs for commands. IEDs shall be directly connected to the switchgear without any need for additional interposition or transducers.

IEDs shall be installed in the local control cubicles independent of each other and the operation shall not be affected by any fault occurring at the station level or in other IEDs of the substation.

The SCMS shall contain the following main functional parts:

- Station Computer System
- Human Machine Interface (HMI) with process data base
- gateway for remote supervisory by control centers
- master clock (e.g. GPS receiver)
- protection fault processing
- service, analysis and engineering system
- data exchange between the different system components via serial bus utilizing fiber-optical links
- collection of the relevant data concerning the substation and distribution of the data where needed
- bay and station level devices for control, monitoring and protection
- process interface parallel wired or connected by a process bus.

Metering
For the monitoring of the power generation and power transmission a metering system using fully static energy meters and energy recorders will be provided. For the metering and billing of each of the outgoing lines, one main and one back-up energy meter with minimum accuracy of 0.5 will be foreseen.

7.5.4.8 Communication

An OPGW (optical ground wire) as part of the transmission line will be installed as communication link with a grid dispatch center and for the transmission line protection. The system will allow telecommunication from the telephone extension via the telephone system and to the dispatch center and the telephone network and vice versa.

7.5.5 Power Supply at Dam Site

The power supply at the dam site, including the dam control building near the power intake, will be connected through the 11 kV overhead transmission line to the 11 kV switchgear in the switchyard control building.

The following equipment will be foreseen at dam site:

- one 11 kV switchgear with fused load-breakers for the connection of the transmission lines and of the auxiliary transformers to the 11 kV supply system
- two auxiliary transformers 11/0,42 kV, 250 kVA with off-load tap selector ± 2 x 2.5%
- one emergency diesel generator set with the rated output of 125 kVA
- one 400 V main distribution board at dame site control building and sub-distribution boards as necessary for the spillway, intake and bottom outlet supply.
- two 100% UPS systems, battery backed-up (mainly for local DCS control).

7.5.6 Diesel generator set

The 400 V emergency diesel generator set will be required to provide the necessary emergency power supply at dam site. The emergency energy generating system will include principally:

- one 4 stroke diesel engine / three-phase AC synchronous generator set, mounted on common base complete with flywheel, air cleaners, vibration damper, forced lubricating system, fuel supply system, radiator type cooling water system
- exhaust manifold, silencer and piping
- electronic governor system
- 24V DC electric starter

- generator with nominal voltage of 400 V, nominal output of 125 kVA and power factor 0.9
- brushless excitation system with rotating diodes.

The unit will start and build-up voltage automatically within 15 seconds. Shut-down of the diesel engines will be by means of fuel shut-off solenoid. As a result, from preliminary estimations of the essential loads to be supplied by the diesel generator set, a rated output of 250 kVA will be considered. The definite rating of the diesel generator set will be determined during the tender phase of the project.

The diesel generator set will be connected to the 400 V dam site distribution board, foreseen in the dam control building.

Normally the diesel generator set will be fully automatically controlled from the powerhouse control room, however a local control (testing, synchronizing) will also be provided in the local dam control room.

8. 220 kV Transmission Line Sharmai HPP - Chakdara Substation

8.1 Design

Basic design parameters, relevant for the design of the 220 kV transmission line between Sharmai switchyard and the grid connection in the Chakdara substation are as listed below:

220 kV System

Nominal System Voltage (rms)	220 kV
Rated Voltage for Equipment (rms)	245 kV
Highest system operating voltage (rms)	245 kV
Frequency	50 Hz
System Configuration	3 Phase, solidly grounded
Rated Short-time Withstand Current (3 sec)	31,5 kA
Lightning Impulse Withstand Voltage (kVpeak)	1050 kV
Power frequency withstand voltage (kVrms)	460 kV

Auxiliary supplies

AC LV System	400/230 V +/- 10%
System Configuration	3-phase (4 wire), solidly grounded
DC system for control and protection equipment	110 VDC
DC system for communication systems equipment	48 VDC

8.2 220 kV Transmission Line

In this section, the conceptual design for the 220 kV double-circuit transmission line is presented. The ultimate capacity of Sharmai HPP, relevant for the design of the transmission line, is 151 MW:

The length of the line is approx. 83 km.

8.2.1 Phase conductors and groundwires

Phase Conductors

Conductor type for the new 220 kV transmission line is the ACSR conductor which is a standard transmission line conductor and is widely used worldwide. Furthermore, more than 90% of the world's transmission lines utilize ACSR conductors as they combine the light weight and good conductivity of aluminum with the high tensile strength and ruggedness of steel. This provides higher tensions, less sag, and longer spans lengths than obtainable with most other types of overhead conductors.

For the new 220 kV double-circuit transmission line, the conductor to be supplied shall be a single conductor of aluminum conductor steel reinforced (ACSR) of type "Hawk" as per ASTM B232. The definitive conductor type shall be chosen during the tender design phase, taking into consideration all electrical, mechanical and environmental parameters.

It has also to be considered that, regarding maintenance and spare inventory, the number of conductors used in the high voltage grid should be reduced to a minimum. Considering that other 220 kV transmission lines are interconnected with the Chakdara substation e.g. 220 kV line between Chakdara and Shahi Bagh and between Chakdara and Mardan, it is recommended that the same conductor configuration is used for the new 220 kV Sharmai - Chakdara line.

The conductor configuration shall offer a satisfactory mechanical performance (considering loads from wind pressure), sufficient capacity for the interconnection of the Sharmai power plant and also a high performance with regard to the radio interference (RI), audible noise (AN) and Corona losses, having the surface voltage gradients within the acceptable limits.

Groundwires

The 220 kV Sharmai - Chakdara double-circuit transmission line should be equipped with 2 groundwires:

- one aluminum clad steel earth-wire (ground-wire), ACS 93
- one OPGW of the same mechanical capacity.

The OPGW shall have 48 fibers (24 fibers as per ITU-T G.652 plus 24 fibers as per ITU-T G.655)
The recommended shielding angle for the towers is 15°.

The groundwire will be installed to protect the line against lightning strikes and shall withstand the specified fault current without suffering damage.

The OPGW shall be based on an ACS (aluminum clad steel) wire construction or on a composite construction of AAC wires with ACS wire reinforcement, in order to fulfill the requirements regarding mechanical strength, conductivity for lightning discharge and short circuit current resistance, corrosion resistance and protection of the optical fibers.

The OPGW shall possess:

- electrical conductivity to cope with single-phase short-circuit currents
- electrical conductivity for effective screening and reduction of induced voltages into nearby telecommunication lines and other conductive objects e.g. pipes, fences, etc.
- mechanical performance allowing proper sag co-relation with the phase conductor

- corrosion resistance
- effective protection of the optical fibers.

8.2.2 Insulators

Composite insulator sets consisting of a solid rod composite insulator unit, assembly fittings and phase conductor fittings shall be provided.

Advantages of composite insulators are:

- small volume and light mass (only 1/10~1/7 of glass insulator), no danger of damage and convenient installation and transportation
- pollution resistance, better contamination performance
- anti-dust, hydrophobic surface of silicon rubber, operation without cleaning
- high mechanical strength inside core is made of epoxy glass fiber
- anti-corrosion
- very good behavior to vandalism.

The insulator units for the new line shall consist of a composite long-rod type insulator featuring a glass-fiber reinforcing epoxy rod core with high temperature vulcanized silicone rubber housing and clevis caps.

The insulator string shall be of sufficient length to provide the required electrical performance with regard to the specific leakage path and minimum required withstand voltages.

The total leakage distance shall be not less than:

L.d. >= 220 kV x 31 mm/kV = 6,820 mm

The suspension towers will be equipped with "I" suspension insulator sets. The double suspension insulator sets shall be used for crossings of buildings, main roads, village roads, overhead transmission lines ≥ 132 kV and railways.

Double tension sets will be installed on all tension and terminal towers. Single tension sets are to be used in slack spans (reduced tensions) at terminal towers and gantries.

Spacing between double strings shall be sufficient to assure good behavior of insulators and good performance of guarding rings (if required).

8.2.3 Tower design

The recommended transmission line towers will be lattice steel structures designed for two 220 kV circuits, with vertical arrangement of phase conductors and two groundwires, one classical and one OPGW, placed in such a way that the resulting shielding angle is 15 degrees.

The vertical phase arrangement provides the advantages of a narrower line corridor compared to a double triangle phase or a horizontal conductor arrangement.

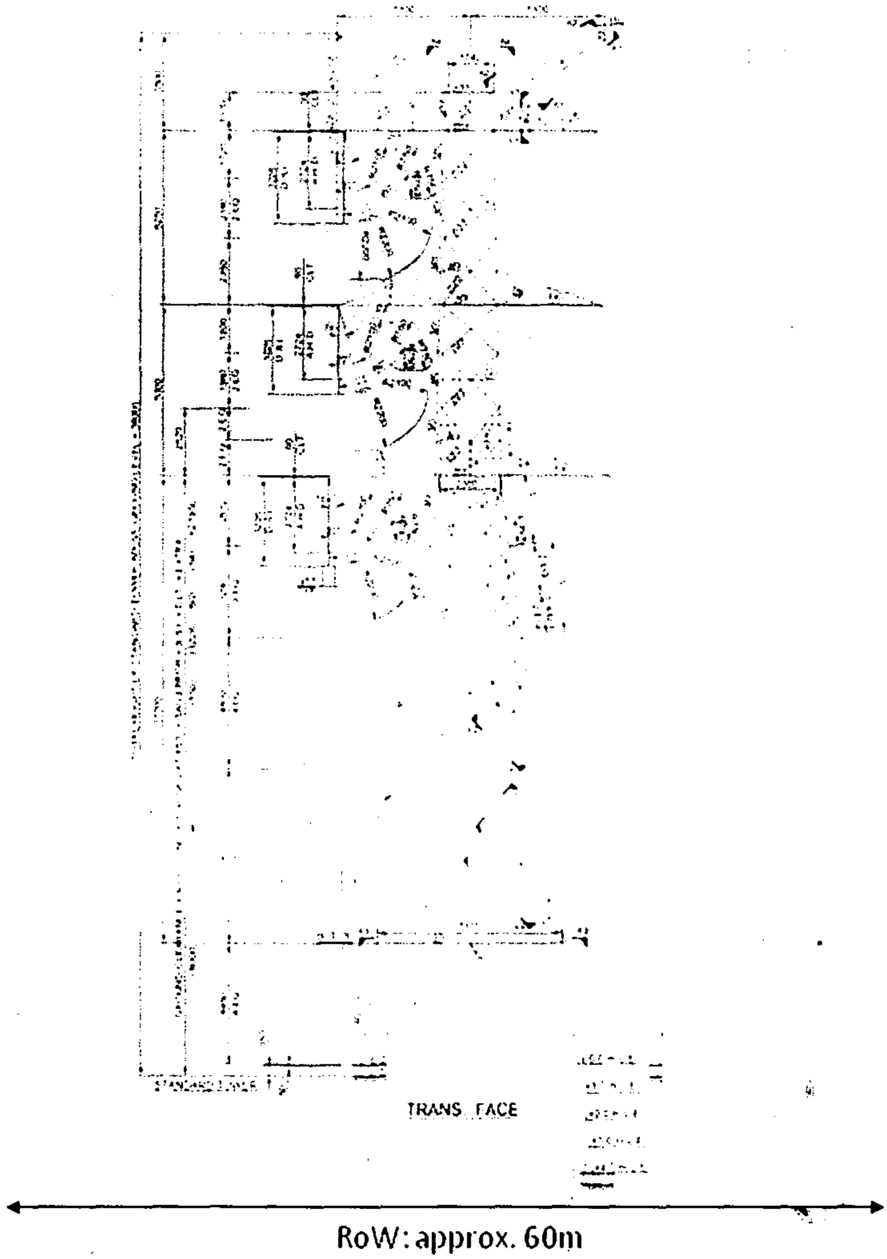


Figure 8-1: Outline geometries for 220 kV double-circuit transmission line

8.3 Chakdara Substation

The interconnect transmission line for Sharmai implies the extension of Chakdara Substations.

8.3.1 Existing Chakdara substation

The new Chakdara Substation is a 220/132 kV substation and is configured in a 1 and 1/2 Breaker arrangement It is was commissioned in July 2018. Space for additional transmission line bays have been allocated inside the Substation premises.

8.3.2 Extension of Chakdara substation

Chakdara Substation will be extended with two 220 kV transmission line bays to accommodate the double-circuit interconnecting transmission line from Sharmai.

Each 220 kV line feeder bay shall consist of the following HV equipment:

- three (3) surge arresters
- three (3) inductive voltage transformers
- one (1) line isolator switch plus earthing switch
- three (3) current transformers
- one (1) three-phase, single-pole operated circuit breaker
- three (3) isolator switches
- two (2) busbar isolator switches.

Furthermore, the existing 220 kV bus system will be extended.

8.4 Transmission Line Routes

A preliminary transmission line route desktop study has been conducted for the interconnection of Sharmai to Chakdara Substation. Detailed figures of the line route can be found in the drawing album and here under:

One basic solution and one alternative solution have been considered for the line route from Sharmai power plant to Chakdara Substation:

- Basic solution:
  Proposed line route following the existing roads to Chakdara Substation
- Alternative solution:
  Proposed line route following the existing road in Southern direction up to coordinates N 3 861 550 / E 769 230, then crossing the mountains until reaching again the road to Chakdara at coordinates N 3 847 385 / E 776 000. The maximum altitude of this transmission line routing will be approx. 1980 m.a.s.l.

The following drawings (also refer to Figures hereunder) show the plan view and the longitudinal profile of each transmission line route:

- Drawing File FS-10-003 Transmission Line - Plan View and Longitudinal Profile (Basic)

- Drawing File FS-10-004 Transmission Line - Plan View and Longitudinal Profile Alternative.

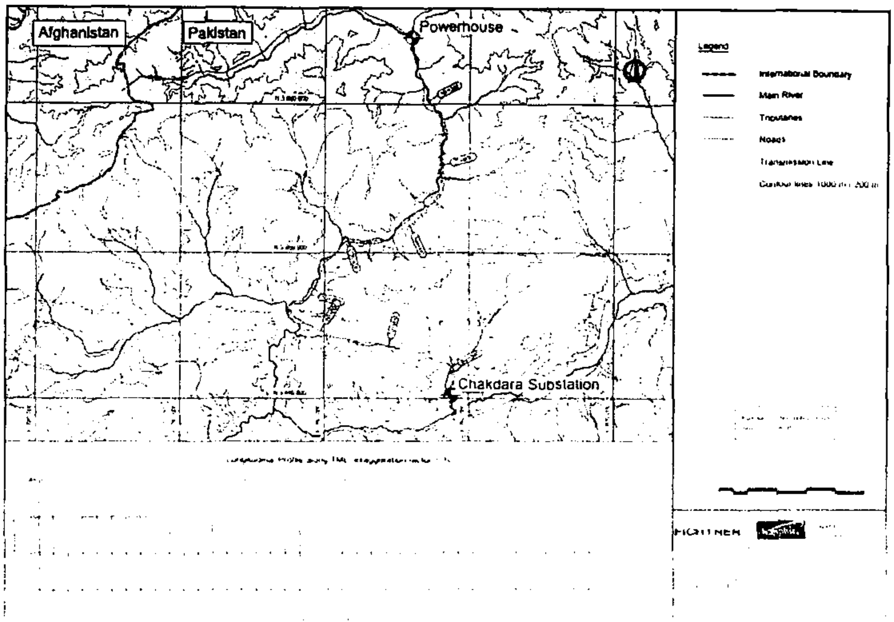


Figure 8-2: Transmission line route (basic solution)

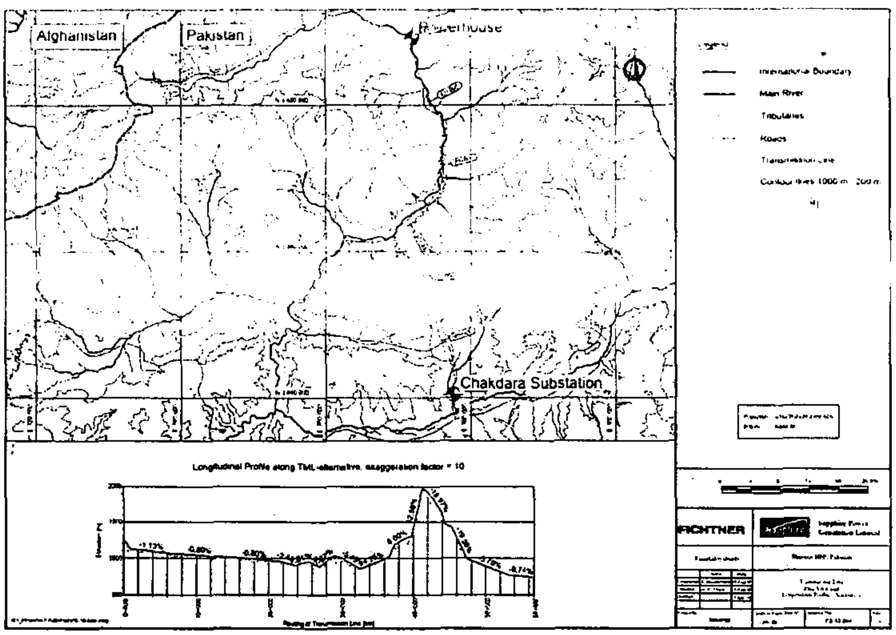


Figure 8-3: Transmission line route (alternative solution)

8.4.1 Transmission line route - basic

The line route is located in Northeastern Pakistan. Situated between 33° and 35° east of Greenwich and between 15° and 16° north of the equator.

Starting at Chardara Substation, the line route will follow the existing road to Sharmai in this section, the line will cross the main road and some village roads.

The extreme terrain points are summarized as follows:

Table 8-1: Summary of extreme terrain points along the routes

Location	Terrain altitude above sea level	Remarks
Sharmai	approx. 1350 m	switchyard location
between	approx. 680 m	Lowest elevation
Chakdara	approx. 740 m	Substation location

8.4.2 Transmission line route - alternative

The line route is located in Northeastern Pakistan. Situated between 71°, 56' and 72°, 03' east of Greenwich and between 34°, 40' and 35°, 07' north of the equator.

Starting at Chardara Substation, the line route will follow the existing road until coordinate coordinates N 3 847 385 / E 776 000. From this point the transmission line will cross the mountains in northern direction. The maximum inclination, until reaching the culmination point of approx. 1980 m.a.s.l. will be in the range of 43% over a distance of approx. 2 km. The transmission line then will descend until coordinates N 3 861 550 / E 769 230, where the existing road to Sharmai is reached again. In this section, the line will cross mountainous terrain. The transmission line will follow the road until Sharmai switchyard, crossing in this section some village roads.

The extreme terrain points are summarized as follows:

Table 8-2: Summary of extreme terrain points along the routes

Location	Terrain altitude above sea level	Remarks
Sharmai	approx. 1350 m	switchyard location
between	approx. 680 m	Lowest elevation
between	approx. 1980 m	Highest elevation
Chakdara	approx. 740 m	Substation location

Comparison of the Line Routes

Basic solution:

The basic solution remains running in parallel to the Dir Chitral Road - Dir Road and Dir Malakand Road to Timergara and further on to the village of Darora which is near the Sharmai power station.

As this routing is passing the 132/33 kV Timergara substation, crossing or parallel running with the existing 132 kV and 33 kV lines must be expected.

The approximate line route length for the basic solution is 83,0 km.

Alternative solution:

The routing of the alternative solution will follow the Dir Chitral Road until coordinate N 3 847 385 / E 776 000 were the transmission line will follow in northern direction the valley leading to the vicinity of the village of. Laram Quilla. From there the transmission line will run in north-northeastern direction, crossing the mountainous ridge at an elevation of approximate 1980 meters and afterwards following the valley leading to the village of Rabaat, located near the Dir Malakand Road. From this point the routing to Sharmai switchyard will be the same as for the basic solution.

As the alternative solution is not passing by Timergara line crossing with existing 132 kV and 33 kV transmission lines could be reduced substantially.

The approximate line route length for alternative solution is 56.7 km. The line route over the mountainous ridge results in additional costs for the line construction in mountainous terrain. However access is given from the rural roads and some field tracks.

Table 8-3: Summary of route lengths

Line Route	Line length
Basic solution	83,0 km
Alternative solution	56.7 km

A significant cost comparison of the two solutions (basic and alternative) only can be done once a more detailed survey of both routings is available The cost for additional line length of 26.3 km of the [illegible]

[illegible]

[illegible] available

9. Power Production Analysis

9.1 Methodology

Power production analysis was performed for Sharmai HPP, assuming its seasonal operation: the plant is envisioned to operate as a peaking plant during the low flow periods, being in operation over 4 hrs/day as a general rule; more than 4 hrs/day the plant shall operate only in order to avoid loss of flow over the spillway. During the high flow season, the plant shall operate as a run-off river plant, preserving the NOL in the reservoir.

The general approach and the model applied for the optimization analyses was updated with the newly obtained data related to the environmental minimum release.

Results of the bathymetric survey were used for the generation of the tailwater curve (TWC) at Sharmai plant. The TWC was simulated by means of Hec-RAS and is presented in the figure below.

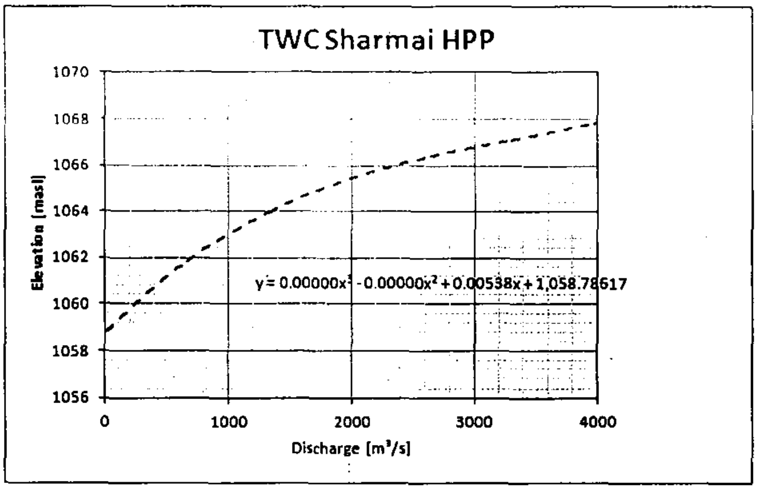


Figure 9-1: Tailwater curve

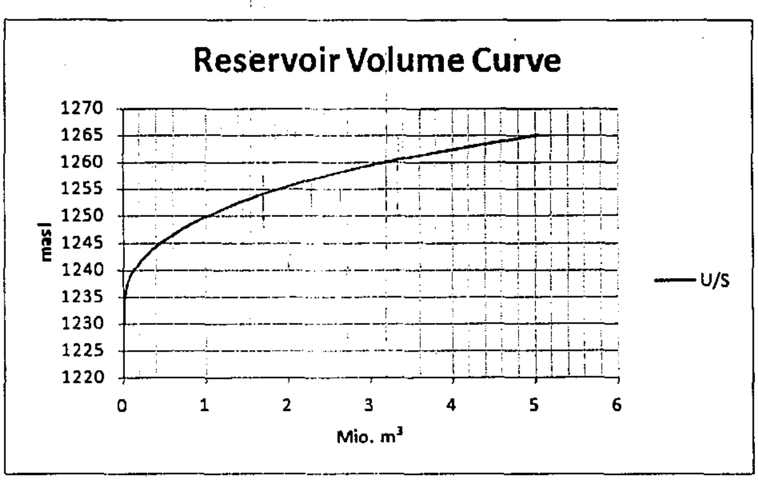


Figure 9-2: Sharmai HPP - reservoir curve

The reservoir volume curve is obtained from the topographic maps and data prepared and processed under the project.

The reservoir simulation model was based on the series of daily reservoir inflow values at the project site for the period 1961-2017 and did consider the following inputs:

- environmental minimum release (as per the ESIA)
- reservoir operating water levels (maximum, minimum and flood)
- tailwater curve
- actual gross head (in daily steps)
- hydraulic losses along the power waterways
  1. calculated friction head losses along the power waterways
  2. estimated local head losses
- actual (daily) flow availability and the considered installed discharges
- efficiency of the generating equipment.

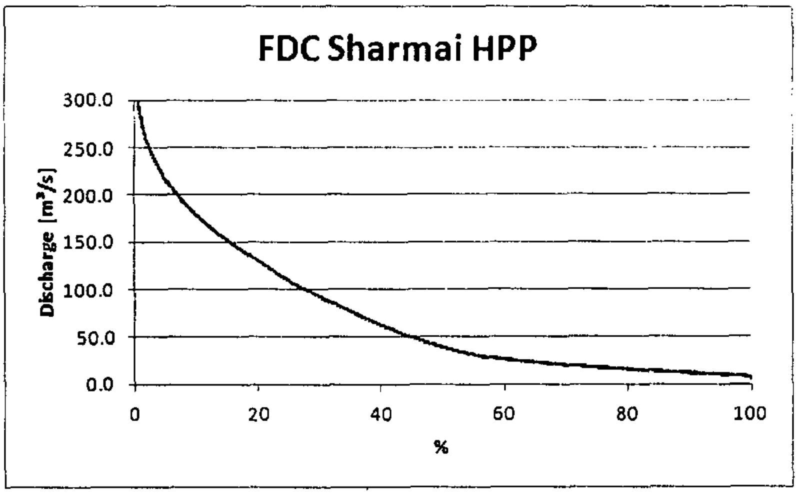


Figure 9-3: Sharmai HPP - flow duration curve

Due to the configuration with 3 units, Sharmai PP will possess flexibility to operate under different flow and demand conditions, covering full range of flows available for power production down to 12 m^3/s while retaining high efficiency at the generating units, as illustrated in the chart below.

Table 9-1: Sharmai turbine efficiencies

Q/Qmax	Q	Efficiency	Q/Qmax	Q	Efficiency	Q/Qmax	Q	Efficiency
0.00	0.001	0.000	1.05	31.500	0.902	1.90	57.000	0.934
0.05	1.500	0.000	1.10	33.000	0.914	1.95	58.500	0.936
0.10	3.000	0.171	1.15	34.500	0.921	2.00	60.000	0.938
0.15	4.500	0.332	1.20	36.000	0.928	2.05	61.500	0.939
0.20	6.000	0.469	1.25	37.500	0.932	2.10	63.000	0.940
0.25	7.500	0.584	1.30	39.000	0.936	2.15	64.500	0.941
0.30	9.000	0.678	1.35	40.500	0.938	2.20	66.000	0.941
0.35	10.500	0.753	1.40	42.000	0.940	2.25	67.500	0.941
0.40	12.000	0.813	1.45	43.500	0.941	2.30	69.000	0.941
0.45	13.500	0.858	1.50	45.000	0.941	2.35	70.500	0.941
0.50	15.000	0.891	1.55	46.500	0.941	2.40	72.000	0.941
0.55	16.500	0.914	1.60	48.000	0.941	2.45	73.500	0.940
0.60	18.000	0.928	1.65	49.500	0.940	2.50	75.000	0.939
0.65	19.500	0.936	1.70	51.000	0.939	2.55	76.500	0.939
0.70	21.000	0.940	1.75	52.500	0.936	2.60	78.000	0.937
0.75	22.500	0.941	1.80	54.000	0.933	2.65	79.500	0.935
0.80	24.000	0.941	1.85	55.500	0.928	2.70	81.000	0.933
0.85	25.500	0.939				2.75	82.500	0.930
0.90	27.000	0.933				2.80	84.000	0.926
0.95	28.500	0.923				2.85	85.500	0.923
1.00	30.000	0.911				2.90	87.000	0.919
						2.95	88.500	0.915
						3.00	90.000	0.911

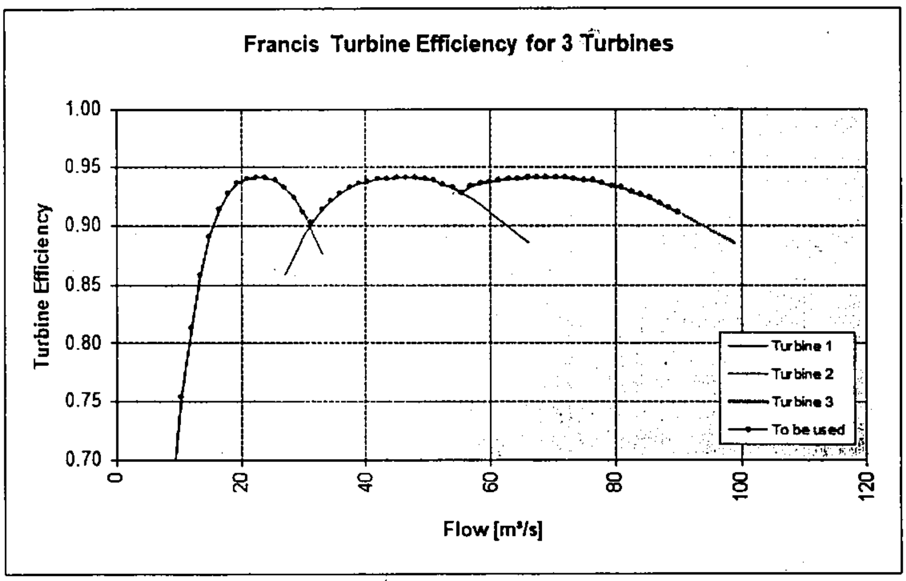


Figure 9-4: Sharmai HPP - turbine efficiency chart

Three units offer the possibility to be operated at a higher overall efficiency. An analysis targeting the weighted average efficiency, which takes in consideration not only turbine efficiencies, but the head losses as well shows an average pondered efficiency of about 91% even at partial loads (possibly relevant for off-peak operation or for the operation as per the system requirements), as presented in the table below.

Table 9-2: **Sharmai HPP weighted average efficiency**

P [%]	Qriver	Q PH	No. of units	eff. Turbine	Local head loss	Gross/Net head ratio	Weighted efficiency
10	178.77	90.00	3	0.91	2.00	96.08%	91.0%
20	130.49	90.00	3	0.91	2.00	96.08%	91.0%
30	92.25	89.45	3	0.91	1.99	96.10%	91.0%
40	62.51	59.71	3	0.91	0.89	98.25%	92.5%
50	39.30	36.50	2	0.91	0.33	99.36%	93.3%
60	26.42	23.62	1	0.93	0.13	99.74%	95.6%
70	20.15	17.35	1	0.905	0.12	99.76%	93.1%
80	15.27	12.47	1	0.855	0.07	99.87%	88.0%
90	11.66	8.86	1	0.785	0.03	99.93%	80.8%
95	9.94	7.14	0	-	-	-	-
100	5.93	3.13	0	-	-	-	-
						Weighted average:	91.0%

As previously concluded, Sharmai HPP will operate in peaking mode, over 4 hrs/day. Due to the existence of a reservoir with daily storage capacity, during the periods of high inflow, Sharmai HPP will be able to generate power outside the peaking hours. However and from the perspective of the power system, the major value the plant brings to the system is the reliability in provision of additional capacity and energy to it, in particular:

- peak energy, produced during the peaking time and with the capacity guaranteed with 90% of time
- off-peak energy, generated outside of the peaking hours and with the lower guaranteed availability
- guaranteed capacity (P90) th plant can provide to the system.

9.2 Results

The results of the power production analysis are visualized in the graphs below.

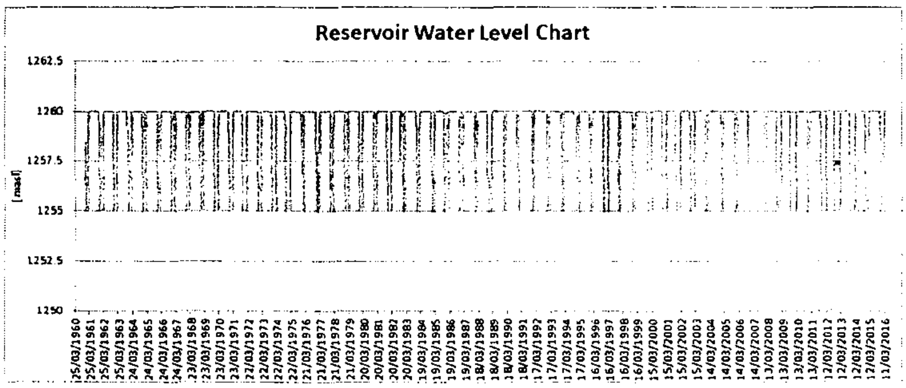


Figure 9-5: **Sharmai HPP - reservoir water levels oscillation**

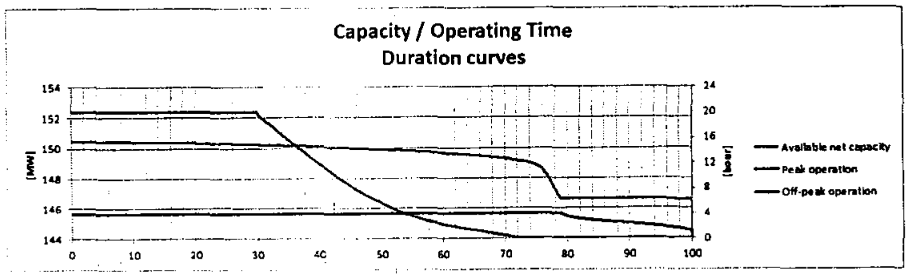


Figure 9-6: Sharmai HPP - capacity/operating time duration curves

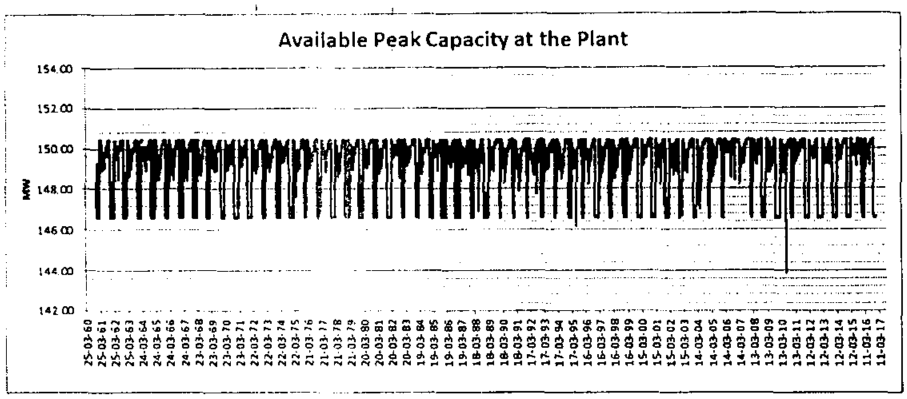


Figure 9-7: Sharmai HPP - available peak capacity

Strong seasonality in energy generation can be observed in the following graph.

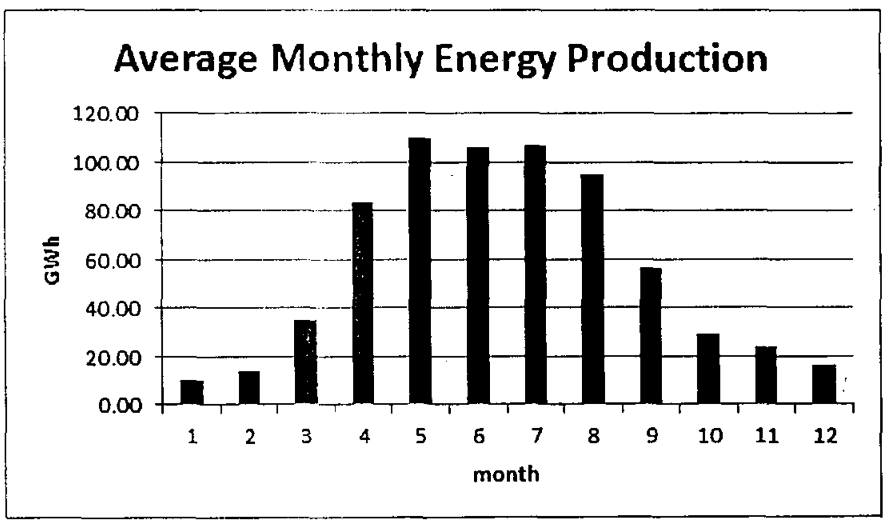


Figure 9-8: Mean monthly energy production

Being designed and operated as a peaking plant, Sharmai HPP can deliver the peak capacity of 146 to 150 MW throughout the year.

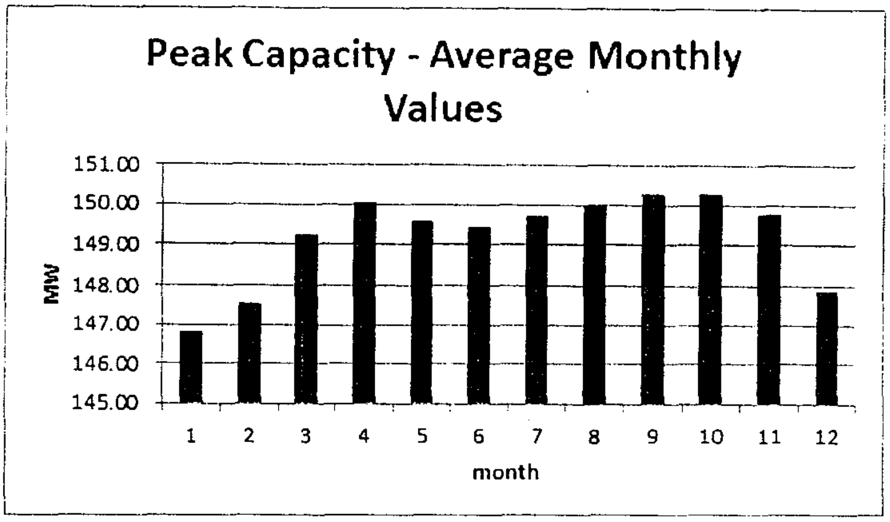


Figure 9-9: Sharmai HPP - peak capacity graph

Table 9-3: Energy production - summary of plant's performance

E peak	E off-peak	E total	P90, peak
[GWh/y]	[GWh/y]	[GWh/y]	[MW]
200,245	489,551	689,796	146.6

Own consumption and the scheduled or forced outages due to exceptional events (floods, heavy sediment inflow, grid failure, etc.) shall be taken into account for the consideration of the project's financial performance.

Own consumption for a project of the size of Sharmai is estimated with about 1%.

Scheduled outages for inspection and regular maintenance of the civil structures, hydraulic steel structures and the equipment are estimated with 15 days per year.

Table 9-4: Energy production - average monthly values (estimates)

Etotal	1	2	3	4	5	6	7	8	9	10	11	12	Total
	[GWh]	[GWh]	[GWh]	[GWh]	[GWh]	[GWh]	[GWh]	[GWh]	[GWh]	[GWh]	[GWh]	[GWh]	[GWh]
1961	10.53	11.00	17.97	72.84	111.34	107.53	111.31	106.27	87.11	35.22	25.25	15.69	712.07
1962	8.78	9.34	14.75	54.58	98.30	105.27	108.14	97.27	54.73	24.44	21.63	15.00	612.22
1963	7.43	6.71	30.40	76.41	111.22	107.50	111.40	98.85	53.74	24.10	26.59	13.45	667.79
1964	10.79	13.66	27.20	80.16	111.44	107.69	109.37	109.15	73.09	28.85	18.37	15.40	705.17
1965	10.13	19.09	25.50	95.98	111.10	107.23	111.15	107.76	58.82	29.59	25.37	14.59	716.31
1966	8.32	13.02	34.78	89.13	111.28	107.35	111.35	99.21	72.90	35.81	23.42	13.47	720.04
1967	7.84	14.82	26.80	67.36	111.17	107.32	111.15	103.64	66.99	32.09	22.14	18.09	689.41
1968	11.32	10.91	26.06	70.65	111.37	107.26	111.11	101.84	47.71	27.18	24.41	22.45	672.28
1969	10.79	15.90	47.53	89.12	109.89	107.42	111.08	103.21	64.70	41.10	30.38	15.85	746.95
1970	9.55	9.14	19.90	69.81	106.94	107.48	103.86	105.48	79.42	34.70	20.88	12.25	679.40
1971	6.47	6.98	14.76	75.49	111.30	107.61	108.97	100.48	47.74	20.70	15.93	9.62	626.05
1972	7.20	12.13	32.17	76.10	111.25	107.31	111.27	101.95	71.36	29.76	27.44	17.49	705.44
1973	12.35	16.67	36.94	91.07	111.17	107.34	111.26	111.40	77.85	31.74	19.00	13.93	740.71
1974	8.57	11.01	24.85	73.98	108.93	107.73	111.01	87.67	40.56	23.84	16.25	12.18	626.57
1975	6.68	9.37	25.66	83.23	111.14	107.44	111.29	110.63	79.82	30.93	24.35	18.93	719.47
1976	13.05	18.07	28.41	95.17	111.23	107.59	111.27	108.75	69.18	31.49	21.76	13.88	729.86
1977	12.15	11.40	17.89	75.72	109.87	107.61	111.33	92.96	52.14	35.79	26.55	16.49	669.88
1978	10.04	9.85	38.73	85.45	111.22	107.47	111.27	107.98	51.80	28.23	28.96	15.58	706.58
1979	8.89	11.45	23.82	86.12	111.15	104.58	111.02	100.03	63.62	25.35	22.65	14.08	682.75
1980	9.77	14.54	44.87	98.30	111.25	107.49	111.43	86.12	53.63	31.27	28.17	17.45	714.29
1981	10.33	13.74	41.05	100.57	111.01	107.71	111.35	96.36	48.96	27.77	21.27	11.06	701.19
1982	8.00	9.00	21.24	67.51	106.09	103.33	95.99	91.24	33.38	26.53	38.14	21.88	622.32
1983	11.80	13.84	38.77	77.08	111.33	107.76	106.60	109.11	70.58	29.81	24.88	18.56	720.11
1984	12.22	12.91	20.83	59.65	109.34	107.37	111.51	110.76	67.93	25.14	26.17	18.22	682.04
1985	11.19	10.81	15.53	54.51	103.72	107.48	105.20	83.14	42.83	27.31	14.09	13.39	589.20
1986	8.44	13.85	34.31	87.10	111.29	103.81	111.31	102.05	41.92	26.05	20.32	21.09	681.54
1987	7.10	9.33	45.14	83.63	111.31	107.65	111.32	108.83	62.69	52.73	34.66	19.40	753.80
1988	9.26	13.01	42.34	91.60	111.18	107.60	111.23	98.69	49.30	24.55	17.40	12.67	688.85
1989	12.05	9.70	17.60	50.19	111.32	107.59	110.90	98.71	51.45	29.96	28.35	26.92	654.74
1990	13.40	22.46	53.08	103.92	110.87	107.59	111.42	102.25	79.19	45.18	36.62	26.02	812.00
1991	22.75	44.30	76.75	107.76	111.07	107.09	111.12	110.14	90.20	38.01	26.63	18.26	864.08
1992	17.37	23.07	47.49	96.11	111.20	107.36	111.04	111.37	85.00	47.71	34.08	19.75	811.55
1993	13.81	12.93	47.79	86.21	111.08	107.56	109.85	92.18	82.48	40.42	36.66	19.21	760.17
1994	12.40	16.39	41.15	76.13	111.14	107.38	111.06	111.40	72.03	35.24	35.61	31.71	761.62
1995	14.46	15.57	42.51	98.62	111.37	107.52	111.01	110.67	54.53	34.14	25.18	17.82	743.41
1996	10.76	17.19	51.84	90.77	111.30	107.22	111.30	101.31	61.02	31.39	19.45	11.77	725.33
1997	6.65	6.07	18.12	86.47	111.22	107.40	111.32	97.80	57.17	32.73	22.41	13.44	670.80
1998	9.37	24.55	47.47	100.02	111.12	107.77	111.25	96.99	55.10	25.43	19.73	10.33	719.13
1999	11.94	18.54	39.99	92.60	111.12	107.60	110.54	90.18	61.95	30.02	29.04	14.12	717.62
2000	9.21	10.10	17.14	67.27	111.07	105.85	104.44	78.50	59.75	33.19	23.23	13.60	633.36
2001	7.38	6.28	9.92	50.28	111.21	105.44	107.44	81.16	50.75	22.04	24.90	12.00	588.79
2002	7.16	11.26	27.25	77.16	110.33	107.66	105.76	94.90	46.49	19.86	17.83	10.78	636.43
2003	6.66	7.48	36.06	96.44	111.24	107.36	111.29	84.49	56.48	26.22	26.95	14.11	684.80
2004	11.51	15.67	60.27	107.49	110.64	107.14	99.38	95.52	46.71	30.37	28.34	20.65	733.69
2005	13.79	16.66	51.67	84.04	111.31	107.61	111.29	79.67	37.95	28.38	20.38	17.52	680.26
2006	15.71	20.35	25.17	65.71	111.50	86.89	73.97	87.17	44.22	23.01	29.31	33.22	616.23
2007	32.97	32.10	67.66	107.86	111.33	107.75	106.96	80.69	58.36	31.07	22.34	25.38	784.48
2008	8.63	13.57	35.32	80.56	111.36	106.55	74.60	59.79	32.63	21.56	12.68	11.72	568.98
2009	7.10	16.95	43.05	81.51	105.83	107.66	111.24	105.40	33.87	15.10	10.24	8.43	646.38
2010	6.24	12.97	61.86	88.22	111.22	101.55	109.46	108.30	60.08	35.65	19.91	16.50	731.96
2011	14.58	20.62	61.39	89.45	111.42	105.70	70.20	51.53	39.57	21.40	20.38	11.26	617.51
2012	8.69	9.48	27.68	105.12	104.27	107.81	109.68	74.93	50.83	19.04	12.33	10.69	640.55
2013	8.11	11.33	59.08	93.09	111.36	107.71	100.52	84.87	37.03	23.32	16.15	9.07	661.66
2014	9.47	12.29	39.18	84.04	111.18	107.75	103.04	63.32	36.60	25.96	20.07	11.94	624.84
2015	7.10	8.93	34.04	92.36	108.57	102.32	108.47	83.42	24.56	26.98	46.34	22.40	665.51
2016	14.62	15.98	37.77	92.51	111.36	106.84	86.57	44.93	35.84	20.93	12.61	10.43	590.39
Average	10.77	14.18	35.65	83.58	110.28	106.58	106.96	95.04	56.90	29.58	24.00	16.27	689.80

Table 9-5: Peak capacity - average monthly values (estimates)

Ppeak	1	2	3	4	5	6	7	8	9	10	11	12	Average
	[MW]	[MW]	[MW]	[MW]	[MW]	[MW]	[MW]	[MW]	[MW]	[MW]	[MW]	[MW]	[MW]
1961	146.82	146.68	148.53	150.20	149.66	149.35	149.61	150.01	150.10	150.40	150.45	147.50	149.11
1962	146.64	146.63	147.17	150.30	150.02	149.78	149.81	150.08	150.30	150.45	150.46	147.18	149.07
1963	146.64	146.64	149.85	150.12	149.48	149.30	149.73	150.06	150.30	150.41	150.40	146.82	149.15
1964	146.89	147.68	149.91	150.15	149.79	149.56	149.56	149.90	150.20	150.43	149.01	147.21	149.19
1965	146.84	149.14	148.73	149.93	149.32	148.93	149.39	149.96	150.28	150.43	150.44	146.97	149.20
1966	146.64	146.98	149.52	150.00	149.57	149.10	149.67	149.94	150.17	150.40	150.45	146.81	149.10
1967	146.64	147.85	148.96	150.15	149.79	149.05	149.40	149.98	150.23	150.42	150.46	147.78	149.23
1968	146.79	147.04	149.07	150.17	149.70	148.97	149.34	149.84	150.33	150.44	150.45	148.97	149.26
1969	146.69	148.05	150.34	150.09	149.62	149.19	149.30	149.74	150.25	150.37	150.42	147.41	149.29
1970	146.63	146.66	148.24	150.22	149.72	149.45	149.92	150.02	150.08	150.40	150.34	146.69	149.03
1971	146.65	146.74	147.35	150.18	149.60	149.46	149.86	150.02	150.33	150.23	147.47	146.63	148.71
1972	146.64	147.41	149.05	150.17	149.54	149.04	149.56	149.95	150.21	150.43	150.43	148.33	149.23
1973	146.70	147.28	150.15	150.06	149.42	149.08	149.54	149.73	150.15	150.41	150.47	146.78	149.15
1974	146.64	147.06	148.02	150.20	149.91	149.63	149.83	150.12	150.37	150.28	147.22	146.64	148.83
1975	146.65	146.63	148.42	150.07	149.38	149.22	149.59	149.69	150.17	150.42	150.45	149.88	149.21
1976	146.67	148.27	149.85	150.00	149.50	149.42	149.56	149.87	150.22	150.42	150.46	146.83	149.26
1977	146.97	146.62	148.29	150.17	149.81	149.45	149.64	150.10	150.31	150.40	150.44	147.36	149.13
1978	146.63	146.62	149.70	150.11	149.49	149.27	149.56	149.92	150.31	150.44	150.43	147.17	149.14
1979	146.63	147.41	148.22	150.07	149.79	149.48	149.62	150.03	150.25	150.45	150.46	146.85	149.10
1980	146.76	147.34	150.02	150.05	149.53	149.29	149.77	150.09	150.30	150.42	150.43	148.15	149.35
1981	146.63	147.97	150.37	149.84	149.21	149.60	149.66	150.07	150.33	150.44	150.32	146.74	149.27
1982	146.64	146.63	147.83	150.23	149.88	149.91	150.08	150.05	150.40	150.25	150.38	149.84	149.34
1983	147.01	147.58	150.38	150.16	149.67	149.66	149.91	149.91	150.18	150.43	150.45	149.61	149.58
1984	146.64	146.77	148.46	150.26	149.73	149.12	149.88	149.98	150.21	150.45	150.44	149.01	149.25
1985	146.63	146.62	147.25	150.30	149.96	149.84	149.86	150.12	150.36	150.24	146.84	147.40	148.78
1986	146.65	148.56	149.15	150.05	149.59	149.71	149.61	149.90	150.36	150.41	149.67	148.01	149.31
1987	146.69	147.19	149.75	150.09	149.74	149.51	149.63	150.03	150.26	150.23	150.40	149.46	149.41
1988	146.63	146.77	149.52	150.02	149.44	149.45	149.50	150.01	150.32	150.38	148.41	146.63	148.92
1989	146.72	146.63	147.82	150.32	149.62	149.43	149.80	150.04	150.31	150.43	150.43	150.44	149.33
1990	147.06	149.58	150.26	149.90	149.02	149.43	149.76	149.98	150.17	150.35	150.39	149.58	149.62
1991	148.16	150.33	150.19	149.66	149.29	148.73	149.36	149.87	150.10	150.39	150.44	149.14	149.64
1992	147.28	150.45	150.34	149.98	149.46	149.11	149.25	149.69	150.01	150.32	150.40	150.14	149.70
1993	147.22	147.31	149.04	149.99	149.46	149.38	149.62	150.11	150.14	150.38	150.39	148.99	149.34
1994	146.78	148.33	150.36	150.11	149.38	149.13	149.28	149.73	150.18	150.40	150.39	150.42	149.54
1995	147.15	148.48	148.98	149.92	149.69	149.34	149.21	149.72	150.30	150.41	150.45	148.98	149.38
1996	146.63	148.54	150.30	150.02	149.59	148.92	149.60	149.85	150.27	150.42	149.59	146.62	149.20
1997	146.65	146.65	148.15	150.03	149.69	149.16	149.62	150.05	150.28	150.41	150.42	146.87	149.00
1998	146.63	148.67	150.33	149.92	149.35	149.68	149.52	150.07	150.30	150.45	150.17	146.65	149.31
1999	146.95	148.66	149.87	150.07	149.35	149.44	149.90	150.10	150.26	150.43	150.43	147.37	149.40
2000	146.63	146.63	147.15	150.23	149.61	149.92	150.02	150.17	150.27	150.41	150.46	146.79	149.03
2001	146.64	146.64	146.94	150.32	149.76	149.88	149.92	150.17	150.32	150.11	149.84	146.63	148.93
2002	146.64	146.75	148.83	150.18	149.66	149.53	149.98	150.04	150.34	150.48	148.99	146.63	149.00
2003	146.65	146.64	150.33	149.92	149.52	149.12	149.58	150.13	150.29	150.44	150.44	146.99	149.17
2004	146.62	146.66	150.27	149.77	148.70	148.80	149.84	150.09	150.34	150.42	150.43	150.45	149.37
2005	146.77	148.50	150.04	150.02	149.78	149.45	149.76	150.17	150.38	150.43	150.43	148.57	149.53
2006	147.27	148.54	150.44	150.24	149.86	150.13	150.21	150.07	150.35	150.46	150.24	150.41	149.85
2007	150.41	150.40	150.19	149.81	149.64	149.66	149.93	150.16	150.28	150.42	150.46	150.45	150.15
2008	146.67	146.75	150.40	150.16	149.68	149.60	150.19	150.27	150.41	149.35	146.61	146.70	148.90
2009	146.64	148.85	150.36	150.04	149.43	149.52	149.51	149.97	150.40	148.05	146.63	146.64	148.84
2010	146.65	147.19	150.25	149.97	149.49	149.82	149.57	149.84	150.27	150.40	150.44	147.93	149.32
2011	146.61	149.68	150.27	150.02	149.76	150.01	150.22	150.32	150.37	150.41	149.86	146.62	149.51
2012	146.64	146.63	148.25	149.98	150.02	149.74	149.84	150.20	150.32	148.98	146.62	146.63	148.65
2013	146.64	146.62	150.08	150.03	149.68	149.60	150.06	150.12	150.39	150.33	147.71	146.63	148.99
2014	146.63	146.61	148.77	150.12	149.77	149.66	150.01	150.26	150.39	150.45	149.44	146.62	149.06
2015	146.64	147.13	150.40	150.02	149.93	150.03	149.95	150.09	150.45	150.44	150.33	149.68	149.59
2016	146.61	146.62	148.99	149.99	149.68	149.85	150.12	150.35	150.39	149.83	146.61	146.63	148.81
Average	146.83	147.56	149.28	150.07	149.60	149.44	149.71	150.01	150.28	150.30	149.77	147.87	149.23

10. Cost Estimate

10.1 Introduction

The cost estimate elaborated within the scope of the Feasibility Study offers an overview to the calculated volumes of materials and works and shows the applied unit and lumpsum prices, which cover all major positions from the prepared design drawings. The prices for the items considered for the detailed priced Bill of Quantities comprise the overall costs of transport, labor, construction equipment and erection.

The unit rates for civil works were derived from tender documents and feasibility studies from similar projects in the region. NHA rates (2014) were considered as well. It should be emphasized that a part of the estimated project costs (and/or subsequent obtained bid prices) may have a wide scatter due to the fluctuation of prices of raw materials, such as steel, copper, galvanizing, oil, etc., major currencies and the actual order volume of the suppliers.

The latter may be of a significant importance in terms of the project implementation time schedule, considering the growing needs and requests for powerhouse hydro-mechanical and electrical equipment purchase in comparison to the current relatively low production capacities of suppliers.

The accuracy of the quantities for earthworks, concrete and reinforcement corresponds to the elaborated feasibility design and the obtained topographical and geotechnical input data, also performed on a feasibility level. The amount of reinforcement per position of works is estimated based on the preliminary stability analysis performed.

Project administrative costs such as governmental and bank fees, financing costs, legal advisers, taxes and duties are considered separately and presented in detail in the report on financial and economic analysis.

The costs for the ESIA mitigation and compensation measures are estimated by the ESIA consultant, in accordance with the currently available level of information. Permanent housing for the project's colony is separately priced for, based on the reference to other similar projects in the region.

Costs for the mobilization are estimated with 5% of the direct costs, based on previous experience of projects of similar nature. These costs comprise the prices for site installations, camps and facilities. The costs for the physical model tests, administration and supervision of the construction works and the EPC design are considered for the estimate of the project total cost.

All indirect cost items are elaborated in the report on financial and economic analyses.
Consequently, the accuracy of the calculated project direct costs is in range of ± 10 to 15 %, - which is in line with the common standards for the

present level of design. Therefore, these results may be considered reliable enough and taken as the input to the financial and economic analyses.

10.2 Bill of Quantities

The following table gives an overview to the calculated quantities for the different positions identified based on the developed feasibility design level drawings.

Table 10-1: Bill of Quantities Cost Estimate

Priced Bill of Quantities, US$

Item	Description	Unit	Quantity	Unit Rate	Amount	Sub totals
A. Civil Works						
1	**Mobilization and Preparatory Works**					12,895,240
1.1	Mobilization and demobilization (camp, installations, etc.)	LS	1.0	5%	11,139,996	
1.2	Clearing and grubbing of Reservoir	m2	430,195.0	0.29	123,681	
1.3	Access roads construction / upgrade, unpaved					
1.3.1	New construction	m	2,315.0	287.50	665,563	
1.3.2	Upgrade of the existing roads / paths	m	6,000.0	161.00	966,000	
2	**River Diversion Works**					8,721,264
2.1	**Cofferdams**					
2.1.1	Upstream cofferdam					
2.1.2	Fill material - Random rockfill	m3	63,285.0	13.80	873,333	
2.1.3	Diaphragm - concrete wall	m3	1,684.5	575.00	968,588	
2.1.4	Downstream cofferdam					
2.1.5	Fill material - Random rockfill	m3	15,023.0	13.80	207,317	
2.1.6	Impermeable carpet	m3	4,030.0	46.00	185,380	
2.2	**Diversion Tunnel**					
2.2.1	Excavation of the inlet portal					
2.2.2	Soil excavation	m3	1,615.0	9.20	14,858	
2.2.3	Rock excavation	m3	2,152.0	17.25	37,122	
2.2.4	Concrete of the inlet portal	m3	1,162.0	172.50	200,445	
2.2.5	Reinforcement	ton	162.7	1,610.00	261,915	
2.2.6	Excavation of the tunnel in rock					
	Class A	m3	1,430.6	88.28	126,293	
	Class B	m3	5,722.2	99.32	568,319	
	Class C	m3	17,166.6	110.35	1,894,395	
	Class D	m3	2,861.1	121.39	347,306	
	Class E	m3	1,430.6	132.42	189,440	
2.2.7	Tunnel concrete lining	m3	3,851.0	207.00	797,157	
2.2.8	Second-stage concrete of the plug in the diversion tunnel	m3	603.0	172.50	104,018	
2.2.9	Reinforcement	ton	890.8	1,610.00	1,434,188	
2.2.10	Excavation of the outlet portal and chute					
2.2.11	Soil excavation	m3	10,196.0	9.20	93,803	
2.2.12	Rock excavation	m3	18,706.0	17.25	322,679	
2.2.13	Concrete of the outlet portal and chute	m3	238.0	172.50	41,055	
2.2.14	Reinforcement	ton	33.3	1,610.00	53,645	
3	**Dam**					5,871,741
3.1	Excavation for Dam foundation					
3.1.1	Soil excavation	m3	14,320.0	9.20	131,744	
3.1.2	Rock excavation	m3	24,543.0	17.25	423,367	
3.2	Concrete					
3.2.1	Left Abutment	m3	9,592.0	201.25	1,930,390	
3.2.2	Right Abutment	m3	6,342.0	201.25	1,276,328	
3.2.3	Reinforcement	ton	796.7	1,610.00	1,282,687	
3.3	Grouting	LS	1.0	5%	252,226	
3.4	Finishing works	LS	1.0	575,000.00	575,000	
4	**Spillway and Stilling Basin**					25,886,899
4.1	Excavation OF the Spillway					
4.1.1	Soil excavation	m3	7,109.0	9.20	65,403	
4.1.2	Rock excavation	m3	24,999.0	17.25	431,233	
4.2	Concrete					
4.2.1	Lean concrete for approach apron	m3	27,757.0	86.25	2,394,041	
4.2.2	Concrete for side walls	m3	12,671.0	143.75	1,821,456	
4.2.3	Reinforcement	ton	3,415.2	1,610.00	5,498,488	
4.2.4	Bridge above the spillway	LS	1.0	230,000.00	230,000	
4.3	Excavation for the Stilling Basin					
4.3.1	Soil excavation	m3	40,952.0	9.20	376,758	
4.3.2	Rock excavation	m3	105,212.0	17.25	1,814,907	
4.4	Concrete					
4.4.1	Concrete for the Stilling Basin foundation and side walls	m3	35,750.0	143.75	5,139,063	
4.4.2	Reinforcement	ton	5,005.0	1,610.00	8,058,050	
4.5	Dewatering	LS	1.0	57,500.00	57,500	
5	**Bottom Outlet**					7,494,719
5.1	Excavation of the Bottom Outlet					
5.1.1	Soil excavation	m3	833.0	9.20	7,664	
5.1.2	Rock excavation	m3	2,238.0	17.25	38,606	
5.2	Concrete					
5.2.1	Concrete of the Inlet structure	m3	12,538.0	172.50	2,162,805	
5.2.2	Reinforcement	ton	626.9	1,610.00	1,009,309	
5.3	Excavation of the Stilling Basin					
5.3.1	Soil excavation	m3	2,912.0	9.20	26,790	
5.3.2	Rock excavation	m3	6,696.0	17.25	115,506	
5.4	Concrete					
5.4.1	Concrete of the Stilling Basin foundation and side walls	m3	12,269.0	143.75	1,763,669	
5.4.2	Reinforcement	ton	1,472.3	1,610.00	2,370,371	

6	**Settling Tank**					**37,730,199**
6 1	Excavation of the Setting Tank					
6 1 1	Soil excavation	m3	7,828 0	9 20	72,018	
6 1 2	Rock excavation	m3	26,104 0	17 25	450,294	
6 1 3	Backfilling	m3	266,507 0	13 80	3 677 797	
6 2	Concrete					
6 2 1	Concrete of the Settling Tank foundation and side walls	m3	70,772.0	143.75	10,173,475	
6.2.2	Lean concrete	m3	39,929 0	86.25	3,443,876	
6.2.3	Reinforcement	ton	7,658.2	1,610.00	12,329,718	
6.3	Excavation of the Setting Tank Chamber					
6.3.1	Soil excavation	m3	362.0	9.20	3,330	
6.3.2	Rock excavation	m3	700.0	17.25	12,075	
6.3.3	Backfilling	m3	13,699.0	13.80	189,046	
6.4	Concrete					
6.4.1	Concrete of the Setting Tank Chamber	m3	19,988.0	143.75	2,873,275	
6.4.2	Reinforcement	ton	2,798.3	1,610.00	4,505,295	
7	**Power Waterways - Headrace Tunnel & Surge Tank**					**113,915,898**
7.1	**Headrace Tunel**					
7.1	Inlet portals					
7.1.1	Soil excavation	m3	313.0	9.20	2,880	
7 1 2	Rock excavation	m3	1,742.0	17.25	30,050	
7 1 3	Concrete					
7 1 4	Concrete of the inlet portals	m3	1,954.0	172.50	337,065	
7 1 5	Reinforcement	ton	273.6	1,610 00	440 432	
7 1 6	Tunnel					
7 1 7	Tunnel excavation					
	Class A	m3	20,772.2	87 84	1,824 609	
	Class B	m3	83 088 6	98 82	8 210,740	
	Class C	m3	228,493 7	109 80	25,088 373	
	Class D	m3	62,316 5	120 78	7,526 512	
	Class E	m3	20,772 2	131 76	2,736,913	
7 1 8	Concrete					
7 1 9	Concrete of the tunnel lining	m3	115,935 0	207 00	23,998,545	
7.1 10	Reinforcement	ton	20,868.3	1,610.00	33,597,963	
7.2	**Surge Tank**					
7 2 1	Excavation of the Surge Tank					
7 2 2	Soil excavation	m3	7,200.0	9 20	66,240	
7 2 3	Rock excavation					
	Class A	m3	1,227 4	119 60	146,797	
	Class B	m3	4,909 6	134 55	660,587	
	Class C	m3	13,501.4	149 50	2,018,459	
	Class D	m3	2,454 8	164 45	403,692	
	Class E	m3	2,454 8	179 40	440,391	
7 2 4	Concrete					
7 2 5	Concrete of the Surge Tank lining	m3	2,500.0	172 50	431,250	
7.2.6	Reinforcement	ton	400.0	1,610.00	644,000	
7.3	**Vertical Shaft**					
7.3 1	Excavation of the Vertical Shaft (Pressure Tunnel)					
7.3 2	Rock excavation					
	Class A	m3	98.1	119.60	11,727	
	Class B	m3	392.2	134.55	52,771	
	Class C	m3	1,078.6	149.50	161,243	
	Class D	m3	196.1	164.45	32,249	
	Class E	m3	196.1	179.40	35,180	
7 3 3	Concrete					
7 3.4	Concrete support	m3	6,419.0	172.50	1,107,278	
7.3 5	Reinforcement	ton	1,155.4	1,610.00	1,860,226	
7.4	**Manifold**					
7 4 1	Excavation of the Manifold					
7 4 2	Rock excavation	m3	2,094 0	172.50	361,215	
7 4 3	Concrete					
7 4 4	Concrete support	m3	621.0	207.00	128,547	
7 4 5	Reinforcement	ton	111 8	1,610 00	179,966	
7 4 6	Steel lining	ton	300.0	4,600 00	1,380,000	
8	**Powerhouse**					**9,273,953**
8 1	Access tunnels construction	m				
8 1 1	Rock excavation	m3	19,930.0	109.25	2,177,353	
8 1 2	Concrete for support	m3	1,631 0	172 50	281,348	
8 1 3	Reinforcement	ton	293 6	1,610 00	472,664	
8.2	Powerhouse cavern					
8.2 1	Rock excavation					
	Class A	m3	1,458.9	87.40	127,503	
	Class B	m3	5,835.4	98.33	573,766	
	Class C	m3	16,047 4	109.25	1,753,173	
	Class D	m3	2,917 7	120 18	350,635	
	Class E	m3	2,917 7	131 10	382,510	
8 2 2	Concrete					
8 2 3	Mass concrete	m3	3,320 0	172 50	572,700	
8 2 4	Structural concrete	m3	3,170 0	207 00	656,190	
8 2 5	Reinforcement	ton	839 2	1,610 00	1,351,112	
8 3	Architectural, landscaping and finishing works	LS	1 0	575,000 00	575,000	

9	**Cavern Transformers & Ventilation/Cable Shaft**					**1,490,096**
9 1	Excavation of the Cavern Transformers					
9 1.1	Rock excavation					
	Class A	m3	322.3	87 40	28,185	
	Class B	m3	1,289.0	98 33	126,741	
	Class C	m3	3,544 8	109 25	387,264	
	Class D	m3	644 5	120 18	77,453	
	Class E	m3	644 5	131 10	84,494	
9.2	Concrete					
9.2.1	Structural concrete	m3	907 0	172 50	156,458	
9 2 2	Reinforcement	ton	145 1	1,610 00	233,643	
9 3	Excavation of the Ventilation/Cable Shaft					
9.3 1	Rock excavation	m3	1,848.0	149 50	276,276	
9 4	Concrete					
9 4 1	Concrete support	m3	298 0	143.75	42,838	
9 4 2	Reinforcement	ton	47 7	1,610 00	76,765	
10	**Downstream Surge Tank**					**4,893,241**
10.1	Excavation of the Surge tank					
10 1 1	Rock excavation					
	Class A	m3	1,286.9	87 40	112,471	
	Class B	m3	5,147.4	98.33	506,118	
	Class C	m3	14,155.4	109.25	1,546,472	
	Class D	m3	2,573.7	120.18	309,294	
	Class E	m3	2,573.7	131.10	337,412	
10 2	Concrete					
10 2 1	Concrete of the Surge Tank	m3	4,801.0	143 75	690,144	
10 2 2	Reinforcement	ton	864.2	1,610 00	1,391,330	
11	**Tailrace Tunnel**					**4,533,930**
11 1	Excavation					
11.1.2	Rock excavation					
	Class A	m3	1,348.4	91.04	122,761	
	Class B	m3	12,135.6	102.42	1,242,951	
	Class C	m3	10,787.2	113.80	1,227,606	
	Class D	m3	1,348.4	125.18	168,796	
	Class E	m3	1,348 4	136.56	184,141	
11.2	Concrete					
11 2.1	Concrete support	m3	3,294.0	172.50	568,215	
11 2 2	Concrete of the Outlet structure	m3	150 0	207 00	31,050	
11 2 3	Reinforcement	ton	613 9	1,610 00	988,411	
12	**Switchyard**					**2,987,993**
12 1	Excavation					
12 1.1	Soil excavation	m3	138,377 6	9.20	1,273,074	
12 1.2	Rock excavation	m3	51,891.6	17 25	895,130	
12.1.3	Filling	m3	12,280 0	13 80	169,464	
12 2	Concrete					
12 2.1	Concrete of the foundations, slabs	m3	250.0	172.50	43,125	
12.2 2	Reinforcement	ton	20.0	1,610.00	32,200	
12.3	Control Building, incl. Gantry portal and Fencing	LS	1 0	575,000 00	575,000	
13	**Subtotal Civil Works**					**235,695,163**
13.1	Direct costs	LS			235,695,163	
B. Equipment						**90,446,200**
	Generating Equipment and Transmission Lines					
B.1	Turbines, Governors, Main Inlet Valves, and related auxiliary systems	LS	3.0	3,809,375.00	11,428,125	
B.2	Generators, Excitation System and related auxiliary systems	LS	3.0	3,858,250.00	11,574,750	
B.3	Mechanical Auxiliary Systems	LS	1.0	11,011,250.00	11,011,250	
B.4	Electrical Auxiliary Systems Power Plant	LS	1.0	16,166,125.00	16,166,125	
B 5	Electrical Auxiliary Systems Dam Site	LS	1.0	681,950.00	681,950	
B 6	Electrical Auxiliary Systems 22kV AIS Switchyard Sharmai	LS	1.0	3,312,000.00	3,312,000	
B 7	Chakdara Substation extension	LS	1.0	1,288,000.00	1,288,000	
B 8	Transmission line 220 kV to Chakdara Substation	LS	1.0	23,862,500.00	23,862,500	
B 9	Hydraulic Steel Structures	LS	1.0	11,120,500.00	11,120,500	
C. Social and Environmental						**5,740,580**
C 1	ESIA costs, incl. expropriation, resettlement and indemnification	LS	1.00	3,240,580	3,240,580	
C.2	Housing colony	LS	1.00	2,500,000	2,500,000	
D. Engineering and Administration						**30,921,362**
D.1	Physical modeling, construction supervision and administration	LS	1.00	4%	12,861,970	
D.2	EPC design works	LS	1.00	6%	18,059,391.77	
	Total Cost (A,B,C,D)					**362,802,305**

The direct project costs (without taxes, duties, price contingencies and financing costs) are estimated at around 363 Mio. USD (or about 2,385 USD/kW of the installed power) excluding price escalation during the duration of construction works/project implementation, i.e. on the reference date end 2018.

Cost breakdown is given in the figure below.

Table 10-2: Cost breakdown

Mobilization and Demobilization, incl. Site Facilities and Camps	11,263,677
Access Roads	1,631,563
Design Works	18,059,392
Civil Works	222,799,923
HSS Works incl. Installation	11,120,500
Mechanical Works incl. Installation	22,439,375
Electrical Equipment incl. Installation	31,734,825
Transmission Line and Chakdara Substation Upgrade	25,150,500
Housing Colony	2,500,000
Resattlement and Environment Mitigation Cost	3,240,580
Physical Modeling, Tender Documents, Supervision, Administration	12,861,970
TOTAL COST	**362,802,305**

11. Project Implementation

11.1 General

Hydropower projects are usually capital intensive, require investment over a longer period of time and require full completion and commissioning before any revenues are earned. The elaborated feasibility design of the project has therefore given particular emphasis to the optimization of the construction process.

A master project implementation schedule including preparatory activities, tendering phase, main construction works and the implementation of social and environmental procedures and measures is prepared. The schedule assumes an EPC implementation concept.

The project's starting date (01 February 2019) was proposed taking into account the ongoing and planned activities on feasibility and tender design.

Main activities in implementation of Sharmai project are:

- Preliminary:
  1. Licensing / Permitting
  2. ESIA and RAP - Implementation
  3. Financial Close

- EPC tendering:
  1. EoI
  2. Bidding Period
  3. Evaluation and Negotiation
  4. Commencement Date

- Mobilization, followed by:
  1. Physical Model Tests for Spillway and Stilling Basin
  2. Preparation of Detailed Design Documents
  3. Equipment Design, Manufacture and Supply

- Site Supervision

- Preparatory Works:
  1. Construction Camp (Downstream) and Site Infrastructure
  2. Site installations (Headworks)
  3. Preparation of Depot and Quarry Areas
  4. Access Roads to Headworks
  5. Access Roads to Powerhouse Access Tunnel, Switchyard and Surge Tank

- Main Works: HEAD WORKS
  1. First Phase - River Diversion
     1.1 Excavation of Diversion Tunnel

2..1 Concrete Works on Inlet/Outlet Portals
3..1 Construction of Cofferdams
4..1 Reservoir Cleaning and Grubbing

2. Second Phase - Concrete Works
   1..2 Excavation Dam Foundation & Abutments
   2..2 Excavation Stilling Basin
   3..2 Concrete Works Spillway Section
   4..2 Concrete Works on Sediment Tank and Bottom Outlet Sections
   5..2 Grouting Works
   6..2 Concrete Works on Dam Crest
   7..2 Installation of Spillway Gantry Crane
   8..2 Installation of Spillway Gates
   9..2 Installation of Bottom Outlet Gate
   10..2 Installation of Gates on Sediment Tank
   11..2 Instrumentation - Piezometers, Survey Control Points, Drainage
3. Third Phase
   1..3 Removal of Cofferdams
   2..3 Closure of Diversion Tunnel

- Main Works: INTAKE and INLET TUNNELS
  1. Open air excavation; inlet portal structure
  2. Concrete works; intake
  3. Wheel gates and trash racks

- Main Works: HEADRACE TUNNEL
  1. Excavation full face from chainage 0+000
  2. Shotcrete and support works from chainage 0+000 km
  3. Excavation full face from chainage 8+600 km
  4. Shotcrete and support works from chainage 8+600 km

- Main Works: SURGE TANK AND PRESSURE SHAFT
  1. Excavation surge tank
  2. Concrete works surge tank
  3. Excavation Powerhouse Access Tunnel
  4. Excavation Headrace Tunnel from 8+500 km to 7+800 km
  5. Excavation horizontal penstock
  6. Excavation vertical shaft
  7. Concrete works Vertical shaft and Manifold
  8. Steel lining vertical shaft and manifold

- Main Works: OUTLET TUNNEL
  1. Excavation outlet tunnel portals (open air)
  2. Tunnel excavation full face
  3. Concrete works

- Main Works: POWER HOUSE CAVERN
  1. Excavation and support works - cavern
  2. Concrete works - first stage (upto el.1055 masl)
  3. Concrete works - second stage

- Main Works: HM- / EM Equipment
  1. Installation of powerhouse crane
  2. Installation of draft tube
  3. Installation of spiral case and manifold
  4. Installation of hydromechanical equipment (gates. etc.)
  5. Installation of turbine-generator equipment
  6. Installation of electrical equipment

- Main Works: TRANSFORMER CAVERN
  1. Excavation and support works - cavern
  2. Excavation and support works - ventilation and cable tunnel
  3. EM Equipment
  4. Installation of transformers
  5. Installation of electrical equipment (to switchyard)

- Main Works: SWITCHYARD and CONTROL BUILDING
  1. Civil works
  2. EM works

- Main Works: TRANSMISSION
  1. Transmission 2200kV
  2. Extension of substation Chakdara

- TESTS on COMPLETION
  1. Pre-Commissioning (dry) tests
  2. Commissioning tests
  3. Trial operation

- COMPLETION of PROJECT / commercial operation.

11.2 Tentative Project Schedule

Tendering and contracting of the project is envisioned to take about 5 months. Culmination of this phase is the contract closure and the mobilization of the Contractor. The necessary administrative and legislative activities including licensing are expected to be completed prior to the commencement of the works at site.

Sharmai HPP is situated at the elevation of approx. 1000 - 1300 masl. The climatic conditions may reduce the rate in execution of construction works during winter months.

A detailed construction and implementation schedule is enclosed in the Drawing Album. The schedule shows that the construction period of about 54 months is realistic and plausible objective for the completion of the construction works and installation of the equipment for a project of this size, assuming a 6-day working week.

This period includes the preparatory works and the main construction works, up to the commissioning of the plant. The following figure shows the elaborated project schedule. The schedule is enclosed in Drawing Album.

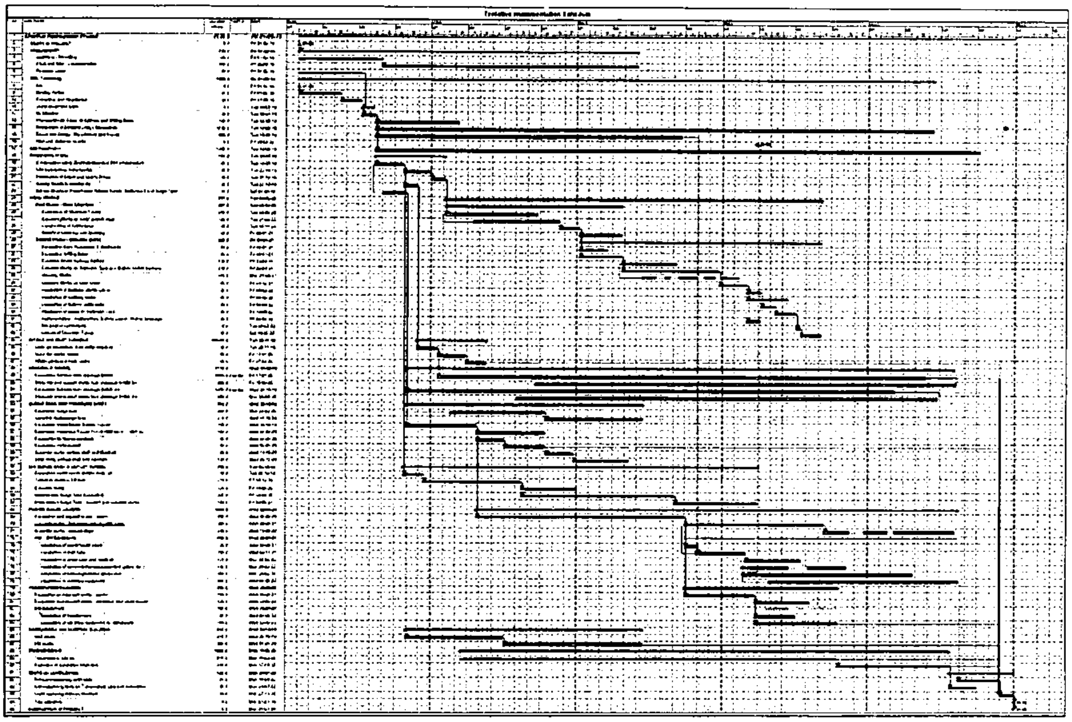


Figure 11-1: Tentative implementation schedule

11.3 Critical Path and Commercial Operation

Based on the elaborated project schedule, it can be concluded that the activities which definitely have the highest influence on the overall project schedule are preparatory activities (financing closure, tendering and contracting of the civil works and equipment supply), as they represent the pre-requisites for the commencement of the main construction works and the works on the underground structures.

The construction activities which lie on the Critical Path will, to a high extent depend on the Contractor's Work Plan, whereas it is expected that the supply and the installation of the equipment and the construction works on the powerhouse cavern will retain their high priority in the final Construction Schedule. On the other side, the construction works on the headrace tunnel may easily intrude on the Critical Path, due to the uncertainties the underground works assume. The schedule envisions almost a simultaneous completion of the construction works on the headrace tunnel and the installation of the equipment in the powerhouse.

The proposed project layout shows the necessity for a parallel execution of different works, which all may affect the overall project timeline.

The possibility to involve a TBM was considered as an alternative to the drill-and-blast method for the headrace tunnel construction. As there is the possibility to construct the tunnel from different access points (headrace, surge tank and interim adit), traditional drill-and-blast method is deemed to offer higher flexibility in case of unexpected underground conditions along the tunnel alignment at lower cost, compared to the TBM. The possibility for TBM application however, remains an option, subject to the EPC contractor's desired approach.

The Tentative Construction Schedule foresees that the different activities are executed in parallel in order to narrow down the time frame required for the completion of the project. It is finally the Contractor's obligation to organize and manage the construction works in the way that the final deadline is met.

A reasonable project's commercial operation can be expected by beginning of 2024.

Hagler Bailly Pakistan

Sharmai Hydropower Project

Environmental and Social Impact Assessment

Volume 1 - Non-technical Summary

HBP Ref.: D8NS3SHR

December 7, 2018

Sapphire Hydro Limited (SHL)

Lahore

Contents

About the Project

Sapphire Hydro Limited (SHL) intends to construct the 150 megawatt (MW) Sharmai Hydropower Project (the Sharmai HPP or the "Project") on the Panjkora River. The dam will be located near the village of Sharmai in Upper Dir District, KP. The powerhouse will be located 18.9 km downstream of the dam near the village of Chumra Payen. The Project area is accessible via road, at a distance of 326.6 km from Islamabad and 239.9 km from Peshawar, the capital of KP province.

This document introduces the Project and its environmental and social impacts in a non-technical language.

What is the Project?

The proposed Sharmai HPP is a run-of-river hydroelectric power project to produce electricity from the flow of water in Panjkora River. The Project, once complete, will have two major components:

1. A concrete dam on Panjkora River with a maximum height of 45 m.
2. The cavern-type powerhouse to generate 150 MW of electric power.

How much water will be diverted?

The diversion of the water will depend on flow in the Panjkora River. Running the Project as a true run of river hydropower project will minimize negative impacts on aquatic ecology downstream. This is the mode of operation recommended in the ESIA.

Cumec

Cumec is the short form of cubic meter per second. It is a unit of flow which equals 35.5 cubic foot per second (cusecs).

Where will the electricity go?

Electricity produced at the powerhouse will be transferred to NTDC, the national company for transmission of electric power, for onward transmission and distribution to consumers.

Who is developing the Project?

Under a contract with the Government of Pakistan, the Project is being developed by the Sapphire Hydro Limited (SHL). SHL is responsible to implement the Project.

Exactly, where is the Project site?

Exhibit 1 shows the location of the Project. The dam will be located near the village of Sharmai while the powerhouse will be located near the village of Chumra Payen.

What is this document?

Any development project—a power plant, a factory, a road, or a canal—requires land and modifies whatever is there on the land. Although the development project itself may be beneficial to the overall economy or the people, the modification of the land and what is on it, can have negative impact for some people, particularly those living on or near the land, and the ecology.

The Environmental and Social Impact Assessment (ESIA) is a predictive study undertaken prior to the development of the project. It has essentially two aims:

1. Identify the potential environmental and social impact of the proposed projects;
2. Design measures to minimize any anticipated adverse impact of the proposed Project and enhance the benefits for the environment and the people.

This document is a non-technical summary of the ESIA report of the Project. The ESIA report was prepared by Hagler Bailly Pakistan (Pvt.) Limited, a leading consultancy firm of the country.

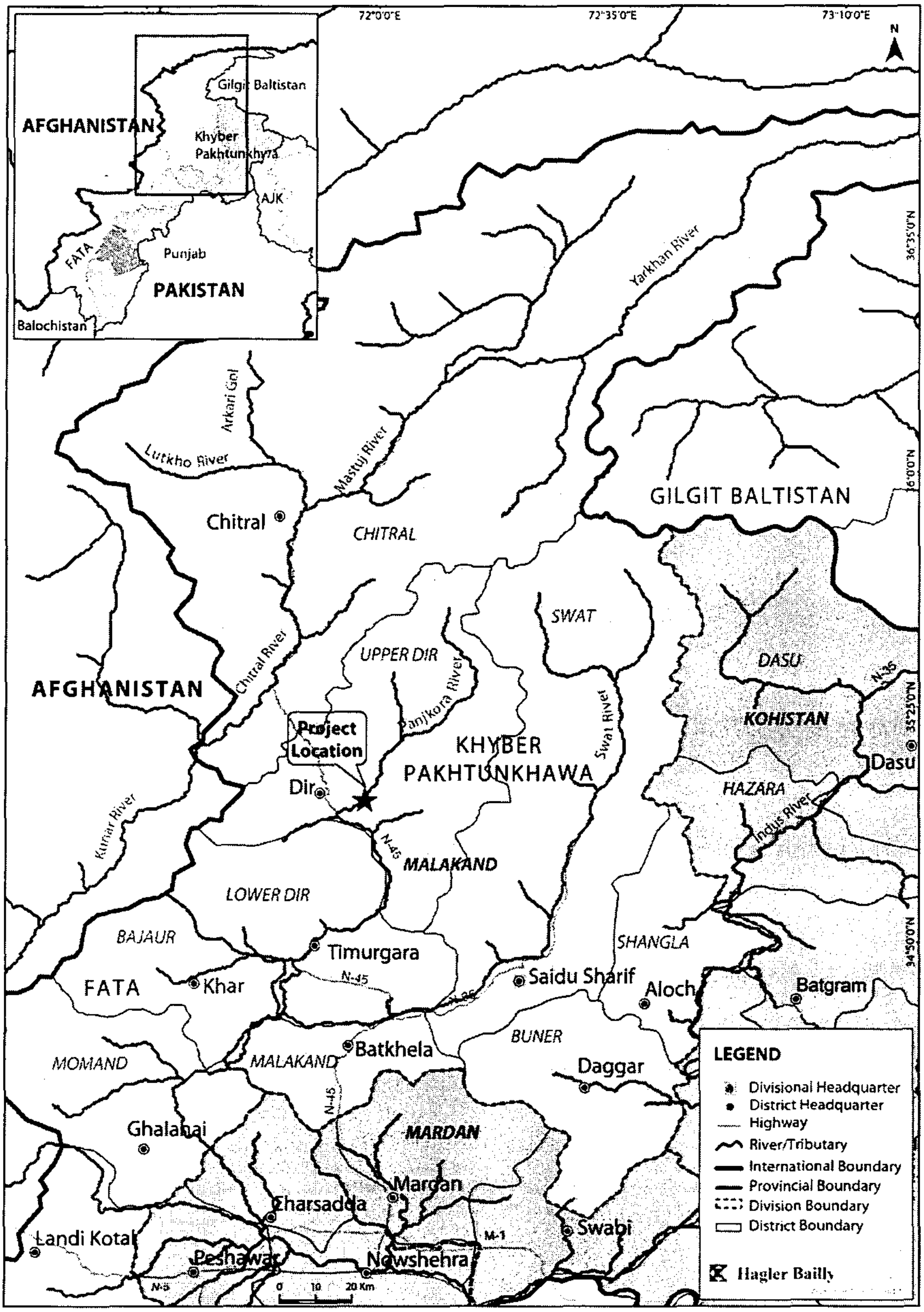
Who will approve the ESIA report?

Preparation of the ESIA report is a legal requirement in Pakistan and in KP. The law requires that all proponents of development projects must assess the anticipated environmental impact of their projects and submit and environmental assessment report to the Environmental Protection Agency (EPA) of KP.

The EPA will evaluate the ESIA against the environmental law and good environmental practices. They will determine whether the ESIA presents enough information to assure that the proposed project design complies with the environmental laws. They will review the potential air pollution, water pollution and noise from the proposed project and judge whether the health and well-being of the people will be protected. The EPA will also determine whether the ecology—the vegetation, fish in the water, the wild animals on the land and the birds—are not going to be destroyed.

A key requirement of ESIA is stakeholder consultation. In it the ESIA team talks to people who are likely to be affected by the Project, provides them with information regarding the Project and seeks their opinion on it. The EPA will also assess whether or not sufficient consultation has been undertaken.

Exhibit 1: Sharmai Hydropower Project Location



As part of the evaluation, the EPA will also conduct public hearings. This event will be open to all in which any person can go and express his or her opinion on the proposed Project. In the end, if the EPA is satisfied they will approve the Project. The construction of the Project will start only after the approval of the ESIA by the EPA.

International lending agencies are involved in the financing of hydropower projects around the world including in Pakistan. Examples of such agencies include The International Finance Corporation (IFC) as well as the Asian Development Bank (ADB). IFC, a member of the World Bank Group, is the largest global development institution focused exclusively on the private sector in developing countries. The IFC is financing a number of HPPs in Pakistan. Similarly the ADB is also financing HPPs in Pakistan. SHL is following IFC and ADB guidelines as best practices.

The ESIA has been developed keeping in consideration the legal requirements as well as guidelines of the IFC.

What will it take to construct this Project?

Construction of the Project will require about 277 acres of land. The land will be utilized for construction of permanent facilities such as the dam, powerhouse, and for temporary facilities required only during construction.

The Project will require aggregate and other construction material which will be obtained locally.

If consultation is mandatory, how many people were consulted?

Different types of consultation were undertaken for this project.

- 45 households from the land acquired for the Project (those who will be affected by the Project) were consulted;
- 31 communities along the Panjkora River were consulted;
- 7 government and non-government organization were consulted.

Why this Project?

KP and Pakistan are going through an acute power shortage. The gap between supply and demand has crossed 7,000 MW. The proposed Project will supply the much needed power to reduce the current gap without relying on import of fuels at the cost of foreign exchange to the country.

The alternatives to the proposed hydropower project include power generation from LNG/imported natural gas-based plants, coal fired steam plants, and fuel oil-based diesel engines. Cost of power generation for the proposed hydropower project is comparable to that for LNG and coal-based options, and lower than that for wind energy and solar PV projects where power generation is intermittent and weather dependent.

The Sharmai Hydropower Project can be completed in four years and is an attractive option amongst currently available alternatives for power generation.

About the Environment

A key component of any ESIA is the environmental and social baseline. This is a description of the environment of the area and includes the land, the people living on it—their social, environmental and cultural conditions—the vegetation and wildlife, and the water resources, the air quality, noise, and the traffic. A comprehensive description of the environmental conditions is presented in the ESIA report grouped in three chapters, the physical, social and the ecological baseline. A brief description of the environment follows.

The area over which the description is provided is called the Study Area. The Study for the Project is shown in **Exhibit 2.**

How is the air quality and water resources in the area?

Air quality in the Project area complies with international guideline limits at all locations as well as National Environmental Quality Standards (NEQS) with the exception of fine particulate matter ($PM_{2.5}$) concentrations that exceed NEQS at two of the three locations tested.

There are low levels of human sources of noise in the area, and very low levels of traffic.

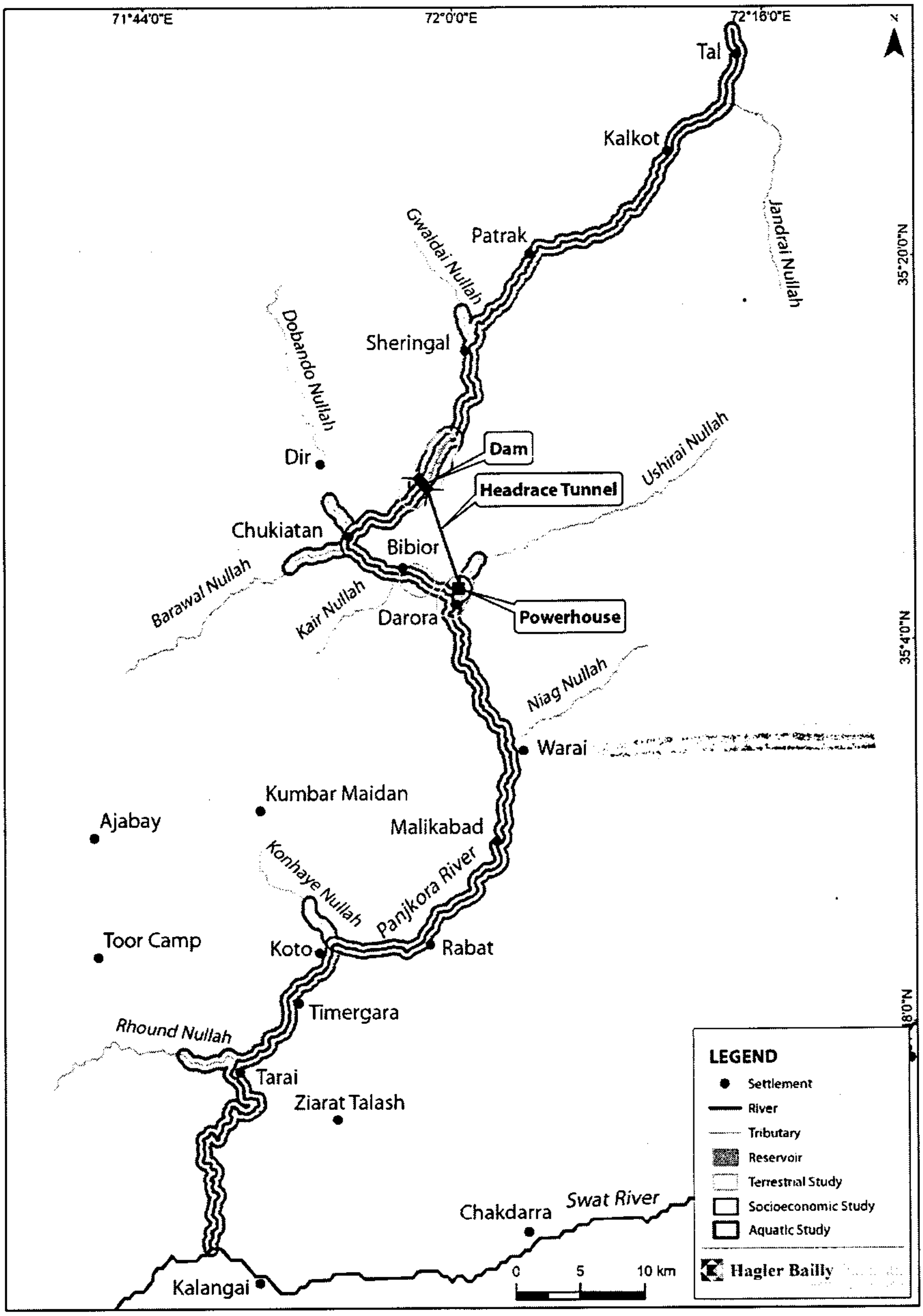
Water resources in the area consist of surface water including rivers and nullahs and groundwater including springs. A census was carried out to map the community water resources for villages near Project facilities. Water quality samples from the Sharmai and community springs were collected and analyzed. All parameters are within permissible levels of NEQS for drinking water and the spring water tested excellent for drinking as there was no bacterial contamination.

What wildlife is found in the area?

The main aspects of the aquatic biodiversity in the Aquatic Study Area include the fish fauna, macro-invertebrates, periphyton biomass, and riparian vegetation. A total of 18 fish species are reported in the Aquatic Study Area (area of impact of the Project). Amongst them there are four of conservation importance including the long distance migratory Mahaseer which is also listed as Endangered on the IUCN Red List, the Alwan Snow Trout also listed as Vulnerable on the IUCN Red List, Suckerhead, as well as one endemic/restricted range species the Stone Loach *Schistura naseeri.*

The most abundant species captured during sampling in 2018 was the Mahaseer. Of the species observed four are of high commercial importance including the Mahaseer, the Common Carp, Spiny Eel and Alwan Snow Trout.

Exhibit 2: Study Areas

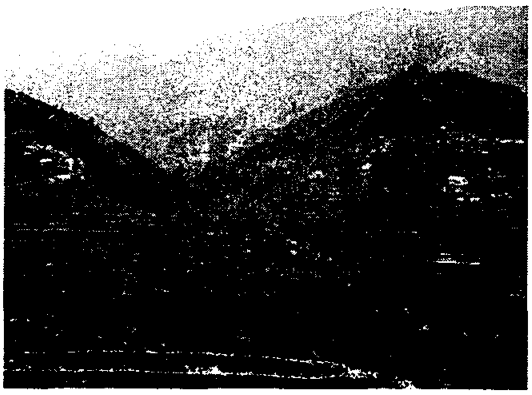


The land area is not too disturbed because of low human population, low grazing of livestock and low levels of extraction of forest resources. The diversity of terrestrial wildlife species is generally low. A total of four mammal species were observed during the surveys carried out as part of the ESIA, namely Asiatic Jackal, Red Fox, Small Asian Mongoose, and Indian Crested Porcupine. None of these species are of conservation importance based on the IUCN Red List. Based on the survey a total of 30 species of birds and seven species of herpetofauna were observed.

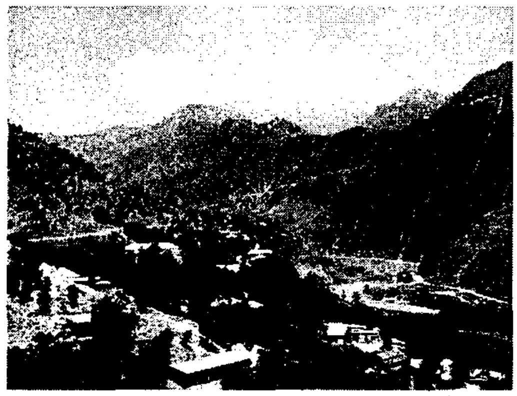
What about the people?

Rural settlement surveys were undertaken in 31 settlements out of the 48 settlements with river dependence or within 1 km of Project facilities. Detailed interviews were conducted with key informants to gather information on each settlement's social and economic setup, with focus on infrastructure and livelihoods. Key physical and socioeconomic features of the Study Area are illustrated in the photographs shown in **Exhibit 3.**

Exhibit 3: Photographs of the Study Area

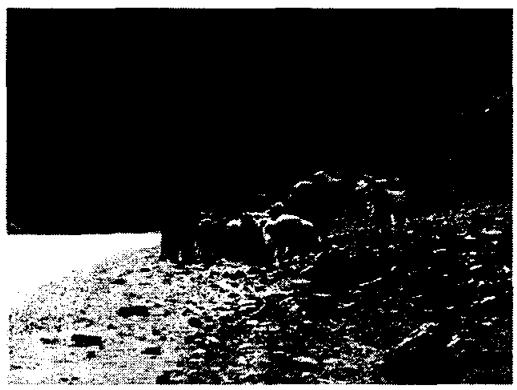


Sharmai Settlement



Siratai Settlement

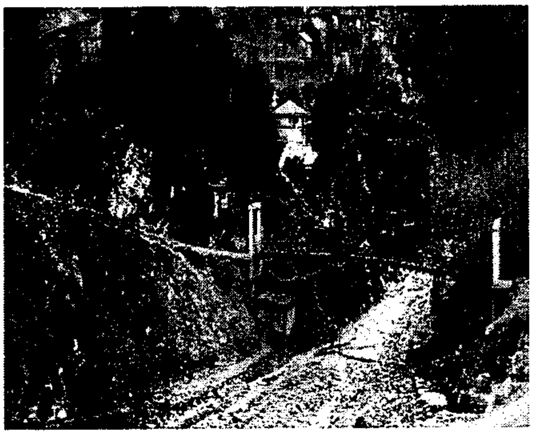
Agricultural Fields at Sharmai settlement



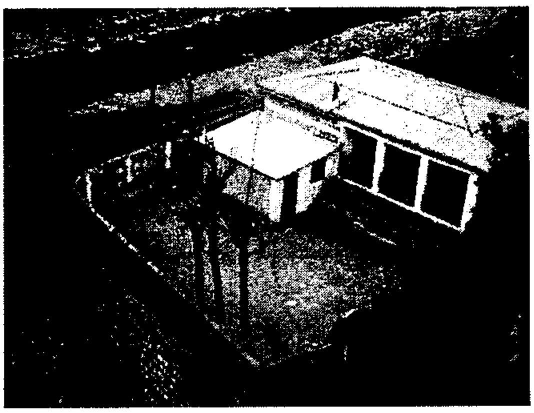
Livestock


Sealed road


Unsealed road


A bridge for Vehicles


Transport


Government Girls Primary School at Serai


Basic Health Unit Daslor

The settlements situated on both sides of Panjkora River in the Socioeconomic Study Area are connected to main towns and cities through sealed and unsealed roads. Main roads in the Socioeconomic Study Area are Mardan Chitral Road, new Sheringal Dir road and old Sheringal Dir road. A traffic survey was undertaken to evaluate the current traffic conditions on routes that could be used for the transportation of equipment, material, and staff to the Project site during construction and operation.

Most of the surveyed settlements are reported to have access to a potable water supply system consisting of a central water storage system, where water collects from a mountain spring and is supplied to the community via a pipeline up to a central point in the community. Distances of the settlements to sources of water range from 1 km to 4 km. Almost all surveyed settlements reported having access to spring water at relatively short distances.

None of the settlements surveyed in the Socioeconomic Study Area are connected to a municipal sewage system. Most human waste is disposed of in septic tanks and all other wastewater eventually runs off into the Panjkora River, affecting water quality. However, dilution rates are high as population is low and quality of river water is relatively unaffected.

The three major fuel sources in the Socioeconomic Study Area include electricity, fuelwood and liquefied petroleum gas (LPG). Natural gas is not supplied in the area. Some of the main towns are connected to the country's landline telephone network in the Socioeconomic Study Area, however the entire area does receive a mobile phone signal. Electricity is available in all the settlements of the Socioeconomic Study Area.

The major sources of income for men are labor (32%), agriculture (16%) private services (11%), government services (10%) and business (9%). For women major sources of income are agriculture (36%), livestock (29%), government services (24%) and private services (8%). Moreover, average income for males is higher in government service sector followed by business sector and private sector. For women average income is higher for government service sector followed by private service and agriculture.

The main river-dependent socioeconomic activity is sediment mining. As observed during the field survey and consultations with the local communities, sediment mining is carried out to some extent throughout the Socioeconomic Study Area. The mineable sediment resource is being extracted to meet small-scale construction demand, involving construction and maintenance of local residential and commercial buildings as well as for roads

There is very little tourism in the Socioeconomic Study Area and recreational dependence on the river was reportedly low in all the settlements. During the survey the survey team did not observe riverside fishing, boating or picnics as a recreational activity or source of income along Panjkora River in the Socioeconomic Study Area. However, tourists pass through the Socioeconomic Study Area going towards Chitral and Sheringal. During their travel people also stay in Timergara and Dir and eat from the road side restaurants.

The Environmental and Social Impacts

What environmental issues were studied?

The ESIA team undertook an extensive assessment exercise to identify and evaluate various environmental issues. The issues that were evaluated included:

- Aquatic ecology—loss of riverine ecosystem due to inundation by Sharmai HPP reservoir; degradation of the river ecosystem in the low flow segment
- Terrestrial ecology—terrestrial habitat loss and impacts of on biodiversity due to construction and operation activities
- Ambient air quality—degradation of air quality due to emission of dust and other gases from construction activities
- Water availability and quality—water resource depletion; changes to groundwater patterns; contamination of water resources; alterations of natural passage of springs due to tunnel construction may disrupt the water availability at mountain springs for local community.
- Noise and vibration—construction equipment noise and vibration from blasting
- Soil, topography, land stability—impact on soil quality and soil erosion
- Livelihood and well-being—employment; training and skill development; enhancement of subsistence and recreational fishing; sand and gravel mining; and land acquisition
- Socio-cultural impacts—pressure on social infrastructure and services; conflicts due to provision of employment to outsiders; conflicting socio-cultural norms; and graveyard management
- Aesthetics and tourism—degradation of aesthetic value of the area due to construction activities; and permanent change in visual character during plant operations
- Traffic and road—impact on highway and community roads
- Climate change—greenhouse gas emissions and climate risk
- Cumulative impact assessment—cumulative impact of the all the hydropower project under construction or planned on the Panjkora River

The study of these issues resulted in a series of mitigation measures that are now incorporated in the design and operation plan of the Project to ensure that the Project impacts are within acceptable limits.

What are the key issues?

The purpose of the ESIA is to identify *all* potential environmental impacts and to propose a comprehensive set of measures to address the concerns associated with them.

Nevertheless the two issues are considered as sensitive requiring particular attention to avert any potential adverse impact.

The cumulative impact of hydropower projects on Panjkora River: Cumulative impacts are those that result from the incremental impact of a project or developments when assessed in combination with other existing or planned projects. The study area selected for this assessment includes the Panjkora River. A total of 5 hydropower projects were considered.

Basin-wide hydropower development will impact migratory fish species, in this case the Alwan Snow Trout, Mahaseer and Suckerhead. Loss of connectivity will confine the populations of these species. The Alwan Snow Trout will be confined to upstream areas while the Mahaseer and Suckerhead will be confined to downstream areas. Isolation will increase in-breeding. Other non-migratory fish species adapted to life in riverine conditions will be impacted due to conversion of part of the river into a lentic or lake habitat.

Unlike other basins such as those in Jhelum and Poonch, impacts on socioeconomic aspects are limited as river-dependent socioeconomic activities are limited. The Project is expected to improve infrastructure and employment conditions in the area.

Environmental Flow (EFlow)

Environmental flows describe the quantity, timing, and quality of water flows required to sustain freshwater and estuarine ecosystems and the human livelihoods and well-being that depend on these ecosystems

This CIA recommends that good practice measures be implemented and followed by all developers and stakeholders for protection of biodiversity in the long term. These include assessing the feasibility of fish ladders on a case by case basis, limiting the number of hydropower projects, requiring all projects to balance economical value with environmental considerations, and holistic environmental flow assessments based on World Bank Guidelines.

The CIA also recommends basin-wide management. This includes standardization of assessment methods using World Bank Guidelines, active regulation by the KP EPA using World Bank Guidelines, increased support to the government departments involved in protection of biodiversity and habitat, greater coordination between developers for achieving synergistic benefits and a balance between environmental and energy requirements in the basin, and support to research for understanding of river ecology and improvement of management practices.

Land Acquisition: The Project will involve acquisition of land for various components. A total of 112.1 hectares of land will be acquired which includes 6.2 hectares of agricultural land, 16.9 hectares of river bed and 89 hectares of uncultivated/ barren land. This will include 43.0 hectares for reservoir, 14.6 hectares for dam, spillway and settling tank, 31.9 hectares for land fill areas, 9.2 hectares for camp, switchyard, batching plant and quarry area, 5.3 hectares for new roads and 8 hectares for upgraded roads.

A resettlement action plan (RAP) will be prepared as a tool to acquire the land and resettle the families living on it in a socially responsible manner. The main objective of the RAP will be to identify social impacts of the Project and to plan measures to mitigate adverse social impacts resulting from loss of assets due to construction of the several Project facilities such as the reservoir, powerhouse, construction camp and offices, access roads etc.

How it will be ensured that all mitigation measures are implemented?

An Environmental Management Plan has been prepared which details the measures that are required to implement the mitigation measures. Responsibilities have been defined for implementation and for monitoring the implementation. Specialized tools such as Site-Specific Environmental Management Plan will be developed to ensure that no all measures are implemented at the project level.

Hagler Bailly Pakistan

Sharmai Hydropower Project

Environmental and Social Impact Assessment

Volume 2 – Executive Summary

HBP Ref.: D8ES3SHR

December 7, 2018

Sapphire Hydro Limited (SHL)

Lahore

Executive Summary

Sapphire Hydro Limited (SHL or Company) intends to develop the 150 megawatt (MW) Sharmai Hydropower Project (the Sharmai HPP or the "Project") on the Panjkora River, near the village of Sharmai in the Upper Dir District of KP. The dam is located 239.9 km from Peshawar. The location of the Project is shown in **Exhibit I.**

Project Background

The conceptual layout of the Project has been subject to various analyses since 1992. The general layout of the Project was initially elaborated by GTZ (German Agency of Technical Cooperation) in 1992. Further information on the design and salient features is available in the Sharmai HPP Project Profile (PEDO, 2016). .[1] Most recently Fichtner carried out an optimization study which was released in March 2018.[2]

The Company contracted the services of Hagler Bailly Pakistan (Pvt.) Ltd. (HBP) to prepare an Environmental and Social Impact Assessment (ESIA) required in line with the applicable national and international laws. It was required that the ESIA shall be prepared in accordance with a) the national legal environmental requirements including that of KP; and b) guidelines and standards of international financing institutions such as the International Finance Corporation's (IFC) Environmental and Social Performance Standards on Social and Environmental Sustainability (PS) and the Asian Development Bank's (ADB) Safeguard Policy Statement.

Study Area

The selection of the Study Area for the ESIA took into account environmentally sensitive receptors that are most likely to be impacted by the Project's development activities during construction and operation. For assessment of cumulative impacts, the Study Area was selected to be large enough to allow the assessment of the Valued Ecosystem Components (VECs) that may be affected by the Project activities. The Study Area defined for the baseline studies and impact assessment is shown in **Exhibit II.**

Policy and Legal Framework

The ESIA process and the environmental and social performance of the Project will be governed by the policies of the GoP, the laws of the Government of KP, and international environmental agreements to which Pakistan is a party. The Project is following IFC's Performance Standards and ADB's guidelines. As the IFC's PSs are the most comprehensive, these are referred to most as assessment criteria. However, where applicable ADB guidelines and World Bank Group guidelines have also been used.

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1 Fichtner, August 2017, Feasibility Study for Sharmai Hydropower Project, Inception Report for Sapphire Electric Company Limited

2 Fichtner, March 2018, Feasibility Study for Sharmai Hydropower Project, Optimization Report for Sapphire Electric Company Limited

Exhibit I: Sharmai Hydropower Project Location

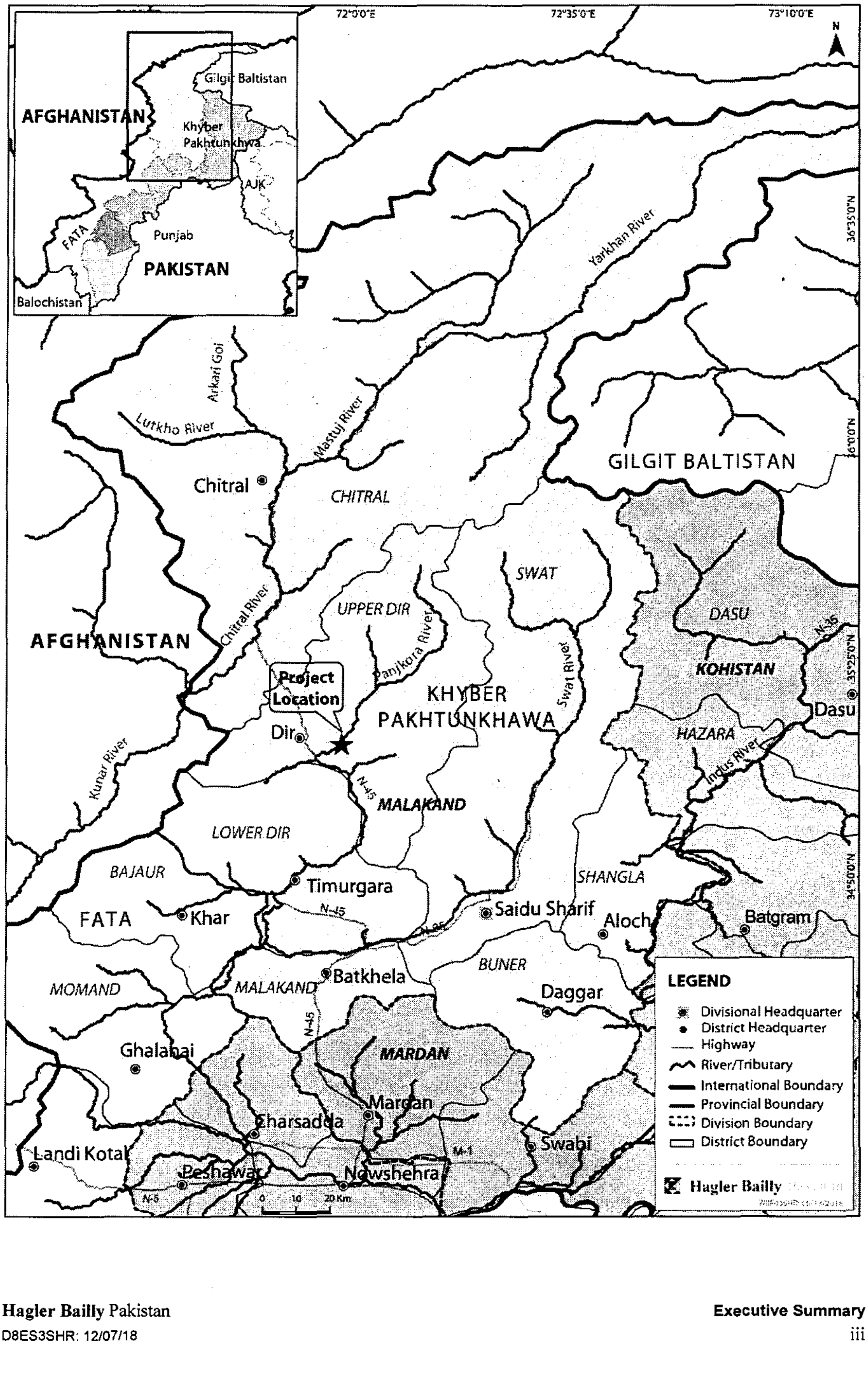
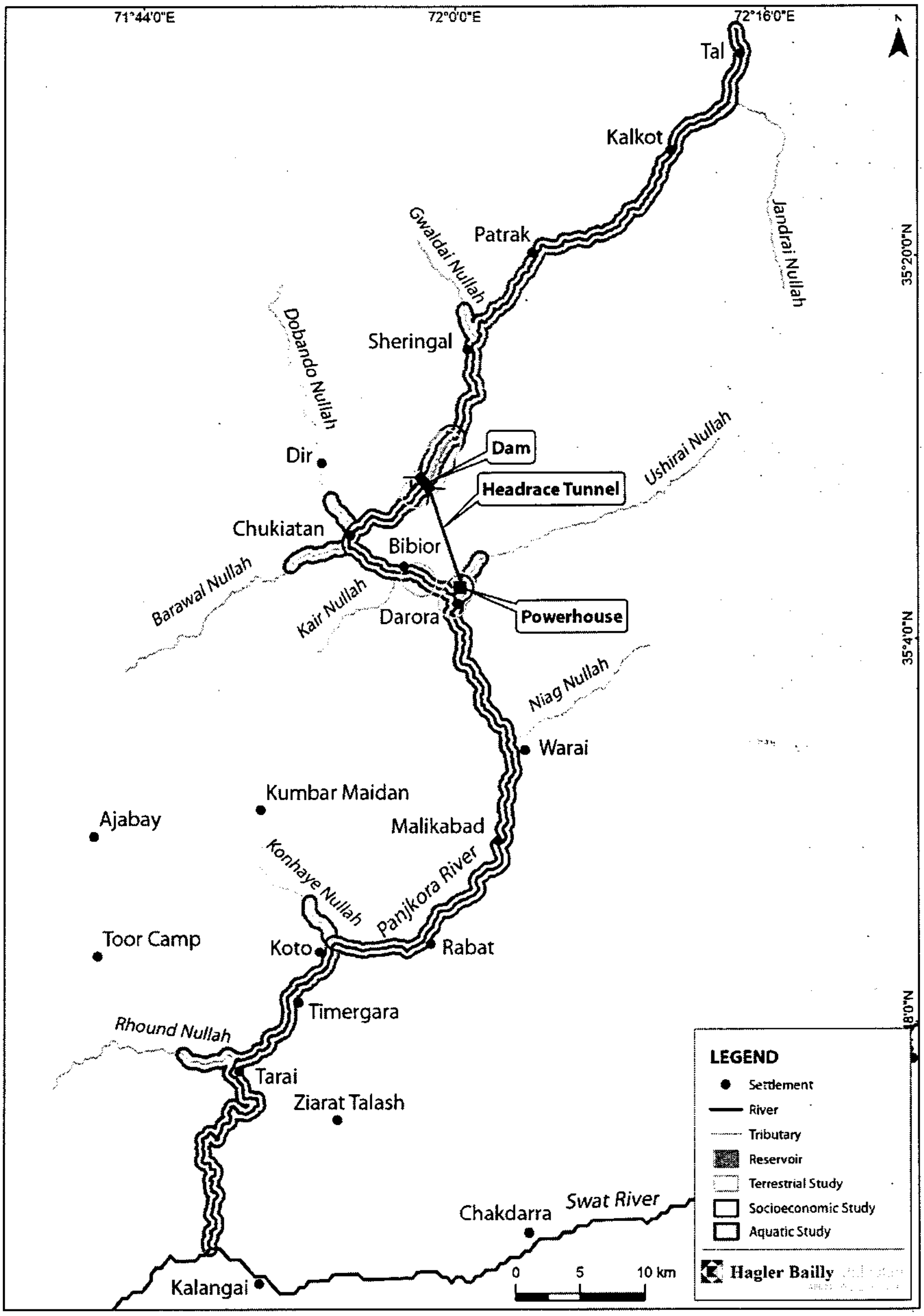


Exhibit II: Study Areas



KP Environmental Laws

The KP Environmental Protection Agency was established in 1989. The KP Environmental Protection Act 2014 is applicable to a broad range of issues and extends to air, water, industrial liquid effluent, and noise pollution, as well as to the handling of hazardous wastes.

Under national guidelines the Project falls under Schedule II of the 'Review of IEE and EIA Regulations, 2000' as it is a hydroelectric project of more than 50 MW. Therefore, it requires an ESIA.

IFC's PS on Social and Environmental Sustainability

The IFC's PSs are applied to manage social and environmental risks and impacts and to enhance development opportunities in private sector financing in IFC member countries eligible for financing. Together, the eight PSs establish standards that the client is required to meet throughout the project life by IFC or other relevant financial institution. SHL will follow PSs of IFC for this Project and will ensure that the contractors/ subcontracts (subcontractors of the contracts) appointed by SHL all follow the IFC's PS on environmental and social sustainability.

ADB Guidelines

The Asian Development Bank (ADB) manages the environmental and social impacts and risks of projects using its Safeguards Policy Statement (SPS) which is built on safeguard policies for environment, involuntary resettlement and indigenous peoples. It consolidates these into a single policy. The SPS aims to promote sustainability of project outcomes by protecting the environment and people from potential adverse impacts.

Under ADB guidelines the Project is categorized as Category A and requires an ESIA.

Project Description

The Sharmai HPP is a run-of-the-river hydropower project to be constructed on the Panjkora River. The catchment area at the proposed dam site is 1,863 square kilometer (km^2)..Sharmai HPP Dam is located near the village of Sharmai and the Powerhouse is located near the village of Chumra Payen.

Power Generation Capacity

The proposed Project is designed to operate with the reservoir at Full Supply Level (FSL) of 1260 m above mean sea level (amsl) with a reservoir capacity of 3 million m^3. At these conditions, the total installed capacity of the hydropower station will be 150 MW.

Land Requirement

The Project will involve acquisition of land for various components. A total of 112.1 hectares of land will be acquired which includes 6.2 hectares of agricultural land, 16.9 hectares of river bed and 89 hectares of uncultivated/ barren land. This will include 43.0 hectares for reservoir, 14.6 hectares for dam, spillway and settling tank, 31.9 hectares for land fill areas, 9.2 hectares for camp, switchyard, batching plant and quarry area, 5.3 hectares for new roads and 8 hectares for upgraded roads.

Main Components of the Project

Dam and Reservoir

The dam site (35°11'11.645"N, 71°57'42.168"E) is located in a narrow stretch of the river valley and will be 45 m high. The live operating storage will be 3 million m^3. The reservoir will have a length of approximately 3.8 km and an area of 0.3157 km^2 (31.57 hectare).

Reservoir Sediment Flushing

A reservoir water level of 1260 masl has been suggested in an attempt to reduce the environmental and social impacts of the Project to an acceptable level.

The Project will face prominent challenges in regard to sediment flushing due to terrain morphology and the volume and size of the reservoir being relatively modes.

In order to anticipate and manage heavy sediment load routine flushing and sluicing of the reservoir are the best measures to secure the sustainability of the reservoir's active storage over its lifetime. Sediment flushing will be continuous.

Intake Structure

The waterways will consist of an underground low-pressure power tunnel (7 to 9 km long) with a downstream vertical surge tank, pressure shaft (as opposed to an open penstock alternative) and a powerhouse structure. The high flow season corresponds with heavy sediment inflow. The discharge rate at which the fine particles should be removed from the reservoir is about 165 m^3/s.

Headrace Tunnel from Intake to Surge Tank

The Project is envisaged to have a low-pressure headrace tunnel that is approximately 8.4 km.

Tailrace Tunnel

The tailrace tunnel stretches 750 m and the area is geologically set on river bed deposits of alluviums/overburden consisting of boulder gravels.

Powerhouse

The option of cavern and surface-type powerhouse were both considered, however, due to a lower technical burden and limited environmental and social impact of the powerhouse factors, the cavern powerhouse was selected. This is both economically more feasible and environmentally has a lower impact on the local community.

Stakeholder Consultation

The consultation process was designed to be consistent with the relevant national legislation and the IFC's PS on social and environmental sustainability. Consultations with the Project stakeholders were undertaken in May, 2018. A Background Information Document (BID) prepared in English and Urdu that informed the stakeholders about the ESIA process and provided a background about the Project was shared with the stakeholders. The feedback from the communities and institutions was recorded and detailed logs of consultations were prepared. A total number of 31 communities were consulted along the Panjkora River. Local government and officials were also consulted.

Summary of Consultation

Following is a summary of concerns expressed by the communities:

- The community should be given preference in jobs related to the Project.
- Project development will increase job availability which is a positive outcome.
- The community uses the river for water and Project development will result in hindrance of access.
- Traffic in the area will increase and could result in difficulties for locals for example school children.
- The Project should contribute to infrastructure development including repairing damaged roads especially those damaged by the Project, establishment of health facilities and schools etc.
- Electricity should be provided free of cost to the community.
- The government should construct schools and hospitals.
- Local livelihoods associated with the river will be affected due to the Project.
- The Project will have impacts on irrigation from the river.
- Inundation by the reservoir will reduce the grazing area available for livestock.
- Affected households should be properly compensated. A colony should be built for the affected people.
- There will be an increase in the risk of land sliding due to Project construction.
- Influx of workers from outside will affect the mobility of local women.
- There may be a loss of water supply for some locals.

The following is a summary of concerns expressed by the institutional stakeholders:

- Job opportunities will increase and infrastructure will improve. The locals should be given preference in jobs related to the Project.
- Electricity should be provided free of cost to the community.
- There will be an improvement in infrastructure as a result of the Project.
- Water supply for some of the locals will be affected.
- Fish fauna in the main river and in the tributaries will be impacted.
- The risk of landslides may increase.

Overview of the Physical Environment

The physical baseline includes a description of the topography, land use, geology, soils, climate, hydrology and water resources, visual character, air quality, noise levels, and traffic.

Topography

The area in the catchment of the Panjkora River, upstream of the Project ranges between 1,062 m and 5,706 m. The majority (~90%) of the catchment upstream of the Project is below 4,000 meters above mean sea level (m amsl) resulting in limited permanent snow and glacial cover within the catchment, relative to other catchments of the higher Himalaya and Karakorum in Pakistan to the east and north east of the Panjkora catchment, respectively.

Land Use and Cover

Based on the land cover and land use classification in the GlobalLand30 dataset, the land cover upstream of the Project includes only minor built up surfaces (artificial surfaces). This commensurate with the type of construction materials used for housing and outdoor flooring in the area. The majority of land cover is grassland (40.1%), followed by forest (30.3%), barren land (16.4%), snow and ice (8.0%), cultivated land (4.5%), shrub land (0.5%) and water bodies (0.06%).

Geology

The Project lies within an active continent-continent convergence margin. The convergence is associated with the Himalayan-Tibetan orogeny and is occupied largely by the east-west trending, high-altitude Himalayan, Karakorum and Hindukush (HKH) ranges in the south and southwest of the vast Tibetan plateau. The HKH orogenic system, created by the Indo-Asian subduction and collision is part of a greater Himalayan-Alpine system that extends from the Mediterranean Sea in the west to the Sumatra arc of Indonesia in the east, over a distance of more than 7,000 km.[3] The system has developed by subduction and closure, as well as uplift, of the Tethys Ocean which existed between the Laurasia and Gondwana super continents. Due to the subduction, the area is associated with widespread volcanism and regional metamorphism.

The geological history of the HKH area is as follows:

- Late Cretaceous emplacement of the Kohistan-Ladakh arc between the Eurasian and Indian plates followed by rapid cooling of the arc between about 85 and 45 Ma[4].[5] This was associated with the early 'soft'[6], i.e. oceanic-continental collision, in the west (i.e. Kohistan-Ladakh part) of the Himalayas.
- Tectonically quiet phase on the northern Indian plate during the Paleocene to early Eocene, when subduction was occurring on the Eurasian margin.[7]

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3 Kirstein, L. A., Sinclair, H., Stuart, F. M. & Dobson, K. 2006. Rapid Early Miocene Exhumation of the Ladakh Batholith, Western Himalaya. Geology, 34, 1049-1052

4 Million years before present

5 Reynolds, P. H., Brookfield, M. E. & Mcnutt, R. H. 1983. The Age and Nature of Mesozoic-Tertiary Magmatism Across the Indus Suture Zone in Kashmir and Ladakh (N. W. India and Pakistan). Geologische Rundschau, 72, 981-1003.

6 Earlier "soft collision" refers to the deformation of the continental margin sedimentary rocks of India and Eurasia whereas the later "hard collision" refers to the collision of the continental crust Shellnut et al, 2013)

7 Reynolds, P. H., Brookfield, M. E. & Mcnutt, R. H. 1983. The Age and Nature of Mesozoic-Tertiary Magmatism Across the Indus Suture Zone in Kashmir and Ladakh (N. W. India and Pakistan). Geologische Rundschau, 72, 981-1003.

- Southward thrusting of the Indian continent margin and associated development of the Ladakh-Deosai Batholiths on the northern Indian margin during the Eocene.[8] The Ladakh-Deosai Batholith intruded the Kohistan-Ladakh arc. The Ladakh-Deosai Batholith is thought to have cooled below 120°C and was actively eroding by the Early Miocene.[9]
- Emplacement of Oligocene Andean arc (Karakorum Batholiths) on the Asian margin, followed by Miocene 'hard' continent-continent collision and intrusions of the "true" granites derived from partial melting of the continental crust.[10]

The Kohistan Ladakh arc and/or Kohistan Batholith[11] is part of a 2,500 km long Cretaceous-Paleogene Andean-type magmatic arc called the Trans Himalayan Plutonic Belt[12]. The Trans Himalayan Plutonic Belt is represented primarily by several major granite batholiths that run parallel to the strike of the Himalayan mountain range. Among the batholiths, the Kohistan-Ladakh arc was sandwiched between Eurasia (the Karakorum terrane) and India during the continental collision in the Early Cenozoic. High-elevation low relief regions are mainly found within the Kohistan-Ladakh arc complex that separates the Indian and Asian plates.[13]

The rock units near the dam and powerhouse site options include metamorphosed sedimentary remnants associated with the Tethys Sea, Eocene acidic to intermediate (granite and diorite) associated with the Kohistan-Ladakh arc. Based on the geology of the area it is likely that acid to intermediate volcanics also exist in the area, or slightly north of the dam site options.

Seismicity

The peak ground acceleration with 10% probability of exceedance in 50 years ranges between 3.2 (0.32 g) and 4.0 m^2/s (0.4 g) for the Project area. Similarly, the Inception Report for the Feasibility Study[14] provides a value for the Maximum Credible Earthquake of 0.46 g.

The two closest registered earthquakes[15] to the Project site (since 1900) were in 2015 (registered M 4.0) and 1995 (registered M 4.9), which are of light magnitude. Nonetheless, the Project area lies between two major tectonically active regions experiencing large and frequent earthquakes, including the main Hindukush region in

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8 Ibid

9 Kirstein, L. A., Sinclair, H., Stuart, F. M. & Dobson, K. 2006. Rapid Early Miocene Exhumation of the Ladakh Batholith, Western Himalaya. Geology, 34, 1049-1052

10 Reynolds, P. H., Brookfield, M. E. & Mcnutt, R. H. 1983. The Age and Nature of Mesozoic-Tertiary Magmatism Across the Indus Suture Zone in Kashmir and Ladakh (N. W. India and Pakistan). Geologische Rundschau, 72, 981-1003.

11 A batholith is a large crystallized and solidified plutonic magma chamber.

12 Kirstein, L. A., Sinclair, H., Stuart, F. M. & Dobson, K. 2006. Rapid Early Miocene Exhumation of the Ladakh Batholith, Western Himalaya. Geology, 34, 1049-1052

13 Beek, V, D., P., Melle, J., Guillot, S., Pecher, A., Reiners, P. W., Nicolescu, S. & Latif, M. 2009. Eocene Tibetan Plateau Remnants Preserved in the Northwest Himalaya. Nature Geosci, 2, 364-368.

14 Ibid

15 Based on USGS data

Afghanistan approximately 90 km northwest of the Project area, as well as the Kashmir region approximately 100 km southeast of the Project area

Soils

The soils of the Project area are composed of piedmont alluvial deposits, where upper layer of the plain/leveled land consists mostly of silty clay loam soils, rich in organic matter content. The subsurface strata are generally sandy loam with gravel. The soils of the hill slopes consist mostly of thin layered sandy loam soils, underlain by rocks or gravelly materials. The valley terraces in-between the mountains are very fertile and used for intensive cropping, while, the hill slopes are used for forest vegetative cover.

Two soil samples; S1 (agricultural land at Rondesh village) and S5 (agricultural land at Alakul village) were collected between July 6 and July 7, 2018 and analyzed for establishing baseline conditions to establish soil fertility and identify any current soil contamination.

Key observations are as follows:

- **Physical:** The pH of soil samples was close to neutral. The EC is observed is low and soils can be classified as non-saline with less than 1 dS/m.
- **Macro-nutrients and Organics:** Organics at all sampling locations don't vary significantly. The maximum organic matter and organic carbon values are observed at S5 as 3.15% and 1.81%, respectively. Phosphates are detected at both sampling locations with maximum at S5 as 928 µg/g. Nitrates are observed at both sampling locations with the maximum value as 943.75 µg/g at S5. Potassium is also high at S5 with the value as 7,522 µg/g. Macronutrient contents shows high fertility at S5.
- **Metals and Major Ions:** Metal contents do not vary significantly through the area sampled, indicating absence of contamination from any industrial activity or spills. Results for Cadmium, Mercury, Selenium and Silver were below the level of reporting.

Climate

A number of different datasets from weather stations operated by the Pakistan Meteorological Department (PMD) and Water and Power Development Authority (WAPDA) as well as multiple gridded datasets are available. Dir station, closest weather station approximately 7 km west of the proposed dam and WorldClim dataset are used to characterize climate of the Project area.

Minimum temperatures around (-10 °C) are observed during the months; December, January and February whereas maximum temperatures around (40 °C) are observed during months; May, June and July.

Minimum or no precipitation is observed during months; October, November, December and January whereas maximum precipitation is observed during months; March and August.

Hydrology

The Panjkora River is a tributary of the Swat River. The key observations based on available flow data are as follows:

- River flows begin to increase with the start of spring in March, when the average temperature at Dir station increases above 4°C.
- Higher flows continue until the recession of the South Asian Monsoon in September. These correspond to increasing temperatures causing snow and glacial melt, as well as the Monsoon rains.
- Following recession of the South Asian Monsoon in September, the flows begins to decrease.
- Lowest monthly flow is observed in January.

Water Resources

Water resources in the area consist of surface water including rivers and groundwater including mountain springs. The information is obtained largely from the hydro census and water quality tests conducted for this study between July 7 and July 11, 2018.

Community Water Supply Census

A hydro-census was carried out to map the community water resources for villages near Project facilities. A 500 m buffer around the Project facilities that may need excavation (including the dam and underground tunnels) was demarcated for the survey to account for the distance to which the impact on ground water might possibly extend.

A total of 24 mountain springs were identified and characterized within the hydro-census area. The total number of households relying on the springs within the area covered by the hydro-census is 708 along with a hotel and a primary school.

These springs are the sole potable water supply for the majority of households. All active springs are used to supply drinking water for humans and livestock.

Small tanks are typically built around springs to store water, and act as constant head for water supply pipelines, or such that communities can manually draw water from the tank.

Based on the pH (mean of 7.11), electrical conductivity (mean of 402 µS/cm), the water is fresh and potable. With a mean temperature of 15 °C the water is also cold and refreshing in the summer.

Water Quality

Water quality samples from Panjkora River, tributaries and community springs were collected and analyzed between July 6 and July 7, 2018.

The key observations on the water quality parameters are as follows:

- The electrical conductivity of the river water is low at around 80 µS/cm upstream of the dam site and gradually increases to 104 µS/cm downstream of the tailrace outlet. Similarly, temperature of the river gradually increases from 18°C upstream of the dam site to about 19°C downstream of the tailrace outlet. Tributaries have a higher temperature, closer to 22°C and springs have a cooler temperature of around 16-17°C.

- Aluminum was found at higher concentrations than the NEQS for drinking water in the river water samples. However, other than this metal, the remaining metals were detected within the NEQS for drinking water for all samples from the river, springs and tributaries.
- The turbidity of both the river and tributary water was 2 to 5 times higher than the recommended value for drinking. The turbidity of spring water was suitable for drinking.
- All samples tested had near neutral pH testing between 7.28 and 7.83.
- Spring water was determined to be bacteriologically unfit for human consumption due to the high number of E. Coli and Coliforms detected.

Visual Character

The visual baseline documents the current aesthetic and visual conditions of the proposed Project site as seen from the nearby receptors. A survey was conducted on July 2, 2018 at eight locations and photographs were stitched to form 180° panoramic views in the direction of Project activities.

The mountainous landscape, deep gorges and vegetation greatly restricts visibility to a maximum of 0.5 to 1.5 km at receptor locations.

Ambient Air Quality

Air quality sampling was carried out at eight different locations in the Study Area between June 20 and July 9, 2018.

The summary of the sampling results is as below.

- The annual and 24-hour concentrations of SO_2 and NO_2 comply with both the NEQS and IF-EHS interim target 1 limits. This leaves a wide room to incorporate emissions of the proposed Project. The maximum levels of NO_2 are observed as 34.86 $\mu g/m^3$ at A3 which is along the N-45 and close to the main town of Bibior. This is because of a low number of sources of gaseous pollutants such as combustion from vehicle engines and household stoves.
- The 24-hour PM_{10} concentration comply with both the NEQS and IFC-EHS interim target 1 limits at all sampling locations. The reference location (A1), near Rondesh village and dam site, located at a distance of 100 m from the road and within a low-density settlement, has the lowest PM_{10} concentrations (65.73 $\mu g/m^3$). Nevertheless, the presence of dust at A1 shows that there are natural sources of dust, such as windblown dust from barren mountains.
- The $PM_{2.5}$ concentrations comply with the IFC 24-hour – interim target 1 limits. However, due to the relatively more stringent NEQS standards for $PM_{2.5}$[16], the concentration of $PM_{2.5}$ exceeds at all locations other than at A1.

[16] The IFC 24-hour – interim target 1 limits for $PM_{2.5}$ are 50% of the PM_{10} limits. However, the NEQS/PEQS limit for $PM_{2.5}$ is more stringent at 24% of the PM_{10} limit.

- Similar and high readings of dust were recorded at A3, A4, A5, and A6 which are near the N-45 and the main towns; Darora, Chumra and Bibior.

Noise Levels

The noise levels were measured at seven locations for 24 hours each, using portable Cirrus Research plc.'s sound level meter, Model CR:1720. Noise levels were within NEQS at all three locations during the daytime.

The daytime noise levels are within the limit of 55 dbA at 5 out of 7 locations whereas, the nighttime noise levels exceed the limit of 45 dBA at all sampling locations. The reference location was the lowest during the daytime hours between 11 am and 5 pm and nighttime hours between 12 am and 3am demonstrating limited anthropogenic contributions to noise.

The Panjkora River is a significant source of noise near the dam and powerhouse sites. N6 is closest of all the sampling points to the Panjkora River and hence exposed to the maximum noise levels. The least noise levels were observed at N7 which is approximately 900 m from the Panjkora River. Other sources of noise during the survey may be due to the national election campaign activities that were taking place throughout the country. Moreover, the summer time is tourist season and the area witness an influx of travelers passing through to the northern parts of the country such as Sheringal and Kumrat.

The hourly noise levels indicate that the peaks are formed around 1 pm and 7 pm during day and 4 am during night. Values exceeding the standards are highlighted.

Weather data was measured during the sampling exercise using a Kestrel 5500 weather meter. The summer season is also the rainy season in the area which may lead to increased noise levels.

Traffic

Traffic counts were conducted at two locations on the transport route, T1 near the dam site on Sheringal Road and T2 near powerhouse site on N-45. The key findings are:

- 95% of all vehicles were Light Transmission Vehicles (LTV). Trucks up to 6 axles were observed on the N-45, although limited in number. A similar distribution of LTV to Heavy Transmission Vehicles were observed on all routes.
- Traffic at T2 (Darora town) begins to increase at 5 am, peaking at around 9 am after which it stays relatively constant until subsiding in the evening around 7 pm. Nighttime traffic on the N-45 at Darora town is very low.
- Traffic at T1 (Sharmai village) peaks at 10 am or an hour after that at Darora town for traffic going and coming for Upper and Lower Dir, which then subsites at around 3 pm. Traffic going towards Sharmai village is low and constant throughout the 24 hours.

Overview of Biodiversity Values

Aquatic Biodiversity

The main aspects of the aquatic biodiversity in the Aquatic Study Area include the fish fauna, macro-invertebrates, and riparian vegetation.

Overview of Fish Fauna in Panjkora River

At least 31 fish species have been reported from the Panjkora River.[17] [18] [19] The most abundant and widely distributed fish species in the Aquatic Study Area include Alwan Snow Trout *Schizothorax richardsonii*, Suckerhead *Garra gotyla*, Kashmir Latia *Chrossocheilus diplocheilus*, Chitral Loach *Triplophysa choprai* and Pakistani Baril *Barilius pakistanicus*.[20] [21] Fish species more common in the upper reaches of the Panjora River include Himalayan Catfish *Glyptosternon reticulatum* and Khyber Loach *Schistura parashari*. Dominant fish species in the lower part of Panjkora River include Golden Mahseer *Tor putitura*, Spiny Eel *Mastacembelus armatus,* Ticto Barb *Puntius ticto* and Dwarf Snakehead *Channa gachua.*

Some of the fish reported from the Study Area are of conservation importance. The Mahaseer *Tor putitora* and Alwan Snow Trout are listed as Endangered and Vulnerable respectively included in the IUCN Red List of Threatened Species.[22]. The Wild Common Carp *Cyprinus carpio* is listed as Vulnerable but it is an introduced species in Pakistan, and therefore, not considered of conservation importance.[23] There are three long distance migratory species in the Aquatic Study Area. These are the Alwan Snow Trout, Mahaseer and Suckerhead *Garra gotyla.*[24] One fish species is restricted range species[25] namely the Stone Loach *Schistura naseeri.*

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[17] Hasan, Zaigham, Sana Ullah, S. B. Rasheed, A. Kakar, and N. Ali. "Ichthyofaunal Diversity of River Panjkora, District Dir Lower, Khyber Pakhtunkhwa." *The Journal of Animal & Plant Sciences* 25, no. 3-2 (2015): 550-563.

[18] Wahab, Abdul, and Ali Muhammad Yousafzai. "Quantitative attributes of family Sisoridae (Siluriformes) with a new record of Glyptothorax kashmirensis from River Panjkora, District Lower Dir, Khyber Pakhtunkhwa, Pakistan." (2017).

[19] Perveen, F and din A.U. "The New Record of Nangra Fish, *Nangra robusta* Mirza and Awan (Siluriformes: Sisoridae: Sisorinae) from River Panjkora, Sheringal, Khyber Pakhtunkhwa, Pakistan". International Journal of Research Studies in Zoology, 1 (2015). PP 21-28.

[20] Ullah, Sana, Zaigham Hasan, Shahzad Ahmad, Muhammad Rauf, and Baber Khan. "ichthyofaunal diversity of rhound stream at district Lower Dir, Khyber Pakhtunkhwa Pakistan." *International Journal of Biosciences* 4, no. 8 (2014): 241-247.

[21] Ahmad, Liaqat. "Icthyofaunal Diversity of River Panjkora Upper Dir Khyber Pakhtunkhwa Pakistan." *Journal of Zoology Studies* 1, no. 6 (2014): 27-32.

[22] The IUCN Red List of Threatened Species. Version 2017-3. <www.iucnredlist.org>. Downloaded on 17 June 2018

[23] Ullah, Sana, Zaigham Hasan, Shahzad Ahmad, Muhammad Rauf, and Baber Khan. "ichthyofaunal diversity of Rhound stream at district Lower Dir, Khyber Pakhtunkhwa Pakistan." *International Journal of Biosciences* 4, no. 8 (2014): 241-247.

[24] FishBase. A global information system on fishes. Accessed on June 17, 2018. Available at http://www.fishbase.org/home.htm

[25] Species in freshwater systems with an extent of occurrence less than 20,000 sq km are considered restricted range species according to IFC's Guidance Note 6.

Some specimens of the Kashmir Catfish *Glyptothorax Kashmirensis* were reported from the confluence of the Panjkora River and Swat River.[26] However, this fish species has neither been seen nor observed in the Aquatic Study Area. According to Dr Muhammad Rafique, a renowned fish specialist of the country, the preferred habitat of the Kashmir Catfish is the Swat River and Panjkora River downstream of its confluence with Swat River, and the Aquatic Study Area does not provide suitable habitat for the Kashmir Catfish.

During the May 2018 surveys a total of 18 fish species recorded from the Aquatic Study Area. The most abundant and widely distribution fish species was Chitral Loach followed by Snow Trout, Khyber Loach, Suckerhead, Pakistani Baril and Golden Mahseer.

Migratory Fish Species

Based on the surveys carried out in May 2018, the Aquatic Study Area contains only two long distance migratory species, the Snow Trout and Golden Mahseer. The Golden Mahseer and Snow Trout is of conservation importance based on the IUCN Red List 2018 as both species are listed as Endangered and Vulnerable respectively.[27]

During winter, the Alwan Snow Trout migrates to lower parts of the Panjkora River to avoid the low water temperature in the upper reaches. With the advent of spring, the Snow Trout starts migrating back to the upper reaches of Panjkora River and tributaries. The Golden Mahaseer shows a similar migratory pattern, however, its habitat ranges are different than Alwan Snow Tout. The Golden Mahaseer has been reported from the lower reaches of Panjkora River. It migrates to lower parts of the main Panjkora River and major tributaries (Konhaye and Rhound Nullah) in winter. In spring, the Mahaseer migrate to upper parts of the Panjkora River up to the town of Rabat (33 km downstream from Powerhouse of Sharmai HPP).

During May 2018 survey, a total of 58 and 47 specimens of the Golden Mahseer and Alwan Snow Trout were observed in the Aquatic Study Area respectively.

Macro-invertebrates

Based on surveys carried out for the ESIA the most abundant macro-invertebrate taxa reported include *Rhithrogena sp.* followed by *Hydropsyche sp.* Most of pollution intolerant genera of macro-invertebrates were observed indicating good water quality. One moderately pollution tolerant genus i.e. *Hydropsyche sp* was also observed.

Riparian Vegetation

The dominant species included *Rumex hastatus, Conyza canadensis, Xanthium strumarium* and *Cannabis sativa.* Vegetation cover was reported as ranging between 2.11% and 3.72%, average plant count was 25.25 and floral diversity was reported as 4.25 species per Sampling Location.

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26 Wahab, Abdul, and Ali Muhammad Yousafzai. "Quantitative attributes of family Sisoridae (Siluriformes) with a new record of Glyptothorax kashmirensis from River Panjkora, District Lower Dir, Khyber Pakhtunkhwa, Pakistan." (2017).

27 Vishwanath, W. 2010. *Schizothorax richardsonii*. The IUCN Red List of Threatened Species 2010: e.T166525A6228314. http://dx.doi.org/10.2305/IUCN.UK.2010-4.RLTS.T166525A6228314.en. Downloaded on **18 April 2018**.

Periphyton Biomass

Periphyton is attached algae on the sediment deposited on stones. It is a source of food for benthos and small fish species. During the May 2018 survey, sampling for periphyton biomass was carried out at a total four sampling locations but no periphyton biomass was collected from any sampling location. This is because of the fast flow of the river in the surveys season, which erodes and washes out periphyton biomass from the cobble stones.

Terrestrial Biodiversity

Terrestrial Flora

A total of 21 plant species were identified in the surveys carried out. None of the species observed are on the IUCN Red List or are globally/nationally threatened species, endemic species or protected species. The locals are dependent on the plants for numerous uses, some of which include food sources, medicinal products, fodder and fuel.

Mammals

A number of mammal species have been reported within the wider area of the Project. Conservational important species in the wider area of the project are includes Musk Deer *Moschus leucogaster,* Asiatic Black Bear *Ursus thibetanus*, Common Leopard *Panthera pardus,* River Otter *Lutra lutra,* Grey Goral *Naemorhedus goral* and Markhor *Capra falconeri.*

During the May 2018 Survey only Asiatic Jackal, Red Fox, Small Asian Mongoose and Indian Crested Porcupine were observed in the Study Area are. None of the observed species is included in the IUCN Red List.

Avifauna

A total of 30 species of birds were observed during surveys carried out as part of the ESIA. Highest abundance was observed in the Agriculture Field habitat while highest diversity was observed in the Oak Forest habitat. Abundant bird species in the Terrestrial Study Area included the, House Sparrow *Passer domesticus*, Jungle Crow *Corvus macrorhynchos*, Common Myna *Acridotheres tristis*, Red-rumped Swallow *Cecropis daurica*, Himalayan Bulbul *Pycnonotus leucogenys*, Long-tailed Shrike *Lanius schach* and Jungle Babbler *Turdoides striata.*

Herpetofauna

A total of seven species of herpetofauna were observed during the surveys carried out as part of the ESIA. The highest reptile abundance and diversity was observed in Agriculture Field habitat type. None of the herpetofauna species observed are of conservation importance based on the IUCN Red List of Species.

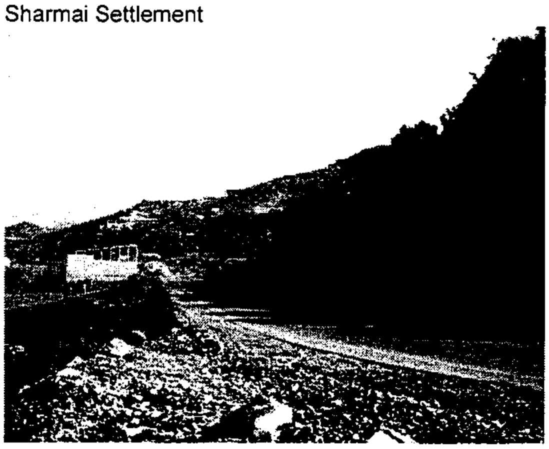
Overview of Socioeconomic Environment

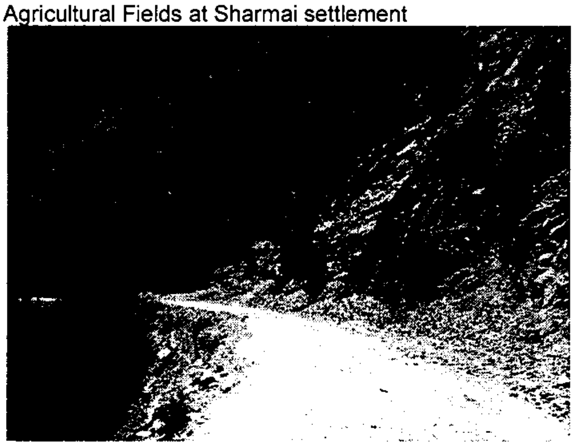
Rural settlement surveys were undertaken in 31 settlements out of 48 settlements with river dependence or within 1 km of Project facilities. Detailed interviews were conducted with key informants to gather information on each settlement's social and economic setup, with focus on infrastructure and livelihoods. Key physical and socioeconomic features of the Study Area are illustrated by the photographs in **Exhibit III.**

Exhibit III: Physical and Socioeconomic Features of the Study Area


Sharmai Settlement


Agricultural Fields at Sharmai settlement


Sealed road


Unsealed road


Government Girls Primary School at Serai


Basic Health Unit Daslor

There are a total of 48 settlements in the Socioeconomic Study Area. All the settlements in the area are located along the river within a radius of 1,000 meter from the center of the river. Out of 31 settlements 29% of the settlements are small in terms of number of households (HHs) ranging from 35 to 80 HHs, 55% settlements are medium settlements ranging from 110 to 450 HHs and 16% settlements are large ranging from 500 to 5,000 HHs. The average household size in the Socioeconomic Study Area is 8.1 individuals. All the settlements are rural and migration into and out of the Socioeconomic Study Area was found to be insignificant over the past 10 years.

Main castes in the Socioeconomic Study Area are Sultan Khel, Painda Khel and Wardig. The predominant language in the Socioeconomic Study Area is Pashto (100%), with Urdu as the main secondary language.

Primary schools are available within a distance of 2.5 km for every settlement while middle and high schools are available within the distance of 15 km in the Socioeconomic Study Area.

Main health facilities in the Socioeconomic Study Area are Basic Health Units (BHU), dispensaries and Lady Health Visitors (LHV)/Lady Health Workers (LHW). Hospitals are available at tehsil and district headquarters. No disease was reported as an epidemic. As expected, the most common illness reported in children, adult male and female populations was flu/fever. Other illnesses reported included dysentery, diabetes and jaundice.

The settlements situated on both sides of Panjkora River in the Socioeconomic Study Area are connected to main towns and cities through sealed and unsealed roads. Main roads in the Socioeconomic Study Area are Mardan Chitral Road, new Sheringal Dir road and old Sheringal Dir road.

Most of the surveyed settlements are reported to have access to a potable water supply system consisting of a central water storage system, where water collects from a mountain spring and is supplied to the community via a pipeline up to a central point in the community. Distances of the settlements to sources of water range from 1 km to 4 km.

None of the settlements surveyed in the Socioeconomic Study Area are connected to a municipal sewage system. Most human waste is disposed of in septic tanks and all other wastewater eventually runs off into the Panjkora River, affecting water quality However, dilution rates are high as population is low and quality of river water is relatively unaffected. Most settlements surveyed reported access to pit latrines of some type, although a significant number of households are still using open latrines.

The three major fuel sources in the Socioeconomic Study Area include electricity, fuelwood and liquefied petroleum gas (LPG). Natural gas is not supplied in the area. Some of the main towns are connected to the country's landline telephone network in the Socioeconomic Study Area, however the entire area does receive a mobile phone signal. Electricity is available in all the settlements of the Socioeconomic Study Area.

Markets and banks are available in main towns in the Socioeconomic Study Area. For major purchases the surveyed settlements depend on markets in the main town i.e. Dir, Bibior, Darora, Warai and Sheringal and Timergara. There are one or two small shops selling basic groceries found in all settlements.

The major sources of income for men are labor (32%), agriculture (16%) private services (11%), government services (10%) and business (9%). For women major sources of income are agriculture (36%), livestock (29%), government services (24%) and private services (8%). A significant portion (13%) of households in the Socioeconomic Study Area earn less than PKR 20,000, and can therefore be considered impoverished.

The average landholding in all settlements ranges from 2.81 to 13.11 kanals (0.14-0.66 hectares) per household. The main winter crop in all settlements is wheat and the main summer crop is maize, vegetables are also grown in the Socioeconomic Study Area. Crop yields are good in the Socioeconomic Study Area. Due to small land holdings quantity of crop production is limited and not even sufficient for the households themselves, almost no crops are sold in the market.

People keep cows and goats for milk production and chicken for eggs and meat. A small number of buffalos and sheep was also reported. Most of the population is engaged in livestock rearing. Trends in livestock rearing were found to be consistent across the settlements, and animals commonly owned include bullocks/buffalos, cows, goats. Livestock owners often engage herders to rear goats, whereas poultry, cows and buffalo are reared at home

The socioeconomic activities investigated in detail included sediment mining from the river, irrigation, fishing, driftwood collection, recreation and tourism.

As observed during the field survey and consultation with the local communities, sediment mining is carried out to some extent throughout the Socioeconomic Study Area but is most prevalent on in Zone 4. The mineable sediment resource is being extracted to meet small-scale construction demand, involving construction and maintenance of local residential and commercial buildings as well as for roads.

As reported by the local communities and observed by the survey team irrigation from Panjkora River is not very common and is only present in Zone 4. Moreover, fishing in the Socioeconomic Study Area is insignificant.

There is very little tourism in the Socioeconomic Study Area and recreational dependence on the river was reportedly low in all the settlements. During the survey the survey team did not observe riverside fishing, boating or picnics as a recreational activity or source of income along Panjkora River in the Socioeconomic Study Area. However, tourists pass through the Socioeconomic Study Area going towards Chitral and Sheringal. During their travel people also stay in Timergara and Dir and eat from the road side restaurants.

Environmental Flow Assessment and Impacts on Aquatic Ecology

The assessment of impact on aquatic ecology presented in the report provides predictions for changes in fish populations due to the Project related change in flow conditions in the Panjkora River.

There will be some impacts that will be transmitted to the Swat River downstream, both in terms of connectivity for the migratory fish species and the impact of daily changes in flow by a peaking operation (if resorted to) of the Project powerhouse. These impacts to the Swat River are expected to be minor in view of attenuation of flow from both the tributaries in the Panjkora River downstream of powerhouse of the Project and the Swat River itself. In addition the presence of the under construction Koto HPP will have a greater impact on the Swat River.

There is no trans-basin diversion, and ecosystems other than river such as estuaries and wetlands are not affected. There is no significant dependence on the river ecosystem.

Predictions for impact on fish populations include the Alwan Snow Trout, Chitral Loach and Himalayan Catfish as these species are indicator species for the five species of conservation importance located in the Panjkora River.

Environmental Flow Release and Peaking Scenario

Environmental flow (EFlow), or flow that must be released from the dam to meet the requirements of the aquatic ecosystem, is of concern in the 18.9 km stretch of the river downstream of the dam up to the tailrace oulet.

Both baseload and peaking operations were considered. EFlow release was varied at 10%, 30% and 40% of the minimum 5 days dry season flow[28] of 9.4 m^3/s along with the EFlow release of 2.8 m^3/s as suggested in the Feasibility Study.

Results and Conclusions

In the Base Case, with peaking operation and an EFlow release of 1 m3/s, overall loss in fish populations compared to present day is estimated at 59% for the Alwan Snow Trout, 26% for the Chitral Loach and 17% for Himalayan Catfish. By increasing EFlow to 2.8 m3/s, overall fish populations can be improved by 7% for the Alwan Snow Trout, 2% for the Chitral Loach and 3% for Himalayan Catfish. Increasing EFlow further to 3.8 m3/s is not recommended as loss in power generation increases with marginal benefit to the river ecosystem and fish populations.

Compared to a peaking operation with an EFlow of 2.8 m3/s, a baseload operation with an EFlow of 2.8 m3/s will result in marginal improvement of fish populations but would result in loss of power generation by approximately 3%. Under current policy and commercial frameworks it is not feasible to operate the plant at baseload. A peaking operation with an EFlow of 2.8 m3/s is recommended. Power generation under this operating mode will decrease by about 2.6% compared to the Base Case peaking operation with an EFlow of 1 m3/s. Compared to the Base Case, fish populations will increase by about 7% for the Alwan Snow Trout, 2% for the Chitral Loach and 3% for Himalayan Catfish on account of higher EFlow release.

Study of Alternatives

No Project Option

Pakistan is going through an acute power shortage. The gap between supply and demand has crossed 7,000 MW. The proposed Project will supply the much needed power to reduce the current gap. Environmentally, this Project will contribute towards improving the air quality as in the long run it will displace fossil fuels used in power generation. The Project also aims to protect fish fauna in the Panjkora River, especially fish species of

[28] Minimum 5 days dry season flow is calculated by DRIFT model on the basis of daily hydrological time series. The model calculates the median value of a running 5 days average during the dry season over the 56 years record. The mean minimum 5 days dry season flow provides a more rational basis for assessment of impacts on river ecology as compared to the absolute minimum flow over the time period considered.

conservation importance. The Project will support government departments in providing protection to fish habitat more effectively.

Alternative Technologies and Scale for Power Generation

The alternatives to the proposed run-of-the-river (RoR) hydropower project include power generation from LNG/imported natural gas based combined cycle gas turbines (CCGTs), coal fired steam plants, and fuel oil based diesel engines. In addition, other technologies such as nuclear, and wind and solar renewable energy power plants could also be considered as alternatives. An analysis of the life cycle average cost of generation shows that cost of power generation for the proposed large size run of river (RoR) hydropower project is presently comparable to that for LNG and coal based options. Cost of power generation for the large hydropower projects is also presently lower than that for wind energy and solar PV projects where power generation is intermittent and weather dependent.

Project Impacts

During the scoping stage of the ESIA process, several potential environmental and social impacts of the project were identified. The baseline surveys were conducted keeping in consideration the potential impacts. The potential environmental and social impacts were evaluated based on consideration. A summary of Project impacts is presented in **Exhibit IV.**

Exhibit IV: Summary of Significant Impacts

ID	*Aspect*	*Impact*	*Phase*	*Stage*	*Magnitude*	*Timeframe*	*Spatial Scale*	*Consequence*	*Probability*	*Significance*	*+/-*
1.	Aquatic Ecology	Loss of Riverine Ecosystem due to Inundation by Project Reservoir Upstream of the Dam	C, O	Init	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-
				Res	Minor	[illegible]	[illegible]	Medium	[illegible]	Medium	-
2.	Aquatic Ecology	Loss of Aquatic Biodiversity due to Creation of a Low Flow Section Downstream of the Dam	C, O	Init	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-
				Res	Minor	[illegible]	[illegible]	Medium	[illegible]	Medium	-
3.	Aquatic Ecology	Change in Aquatic Biodiversity due to Changes in Ecological Conditions Downstream of the Powerhouse	C, O	Init	Moderate	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-
				Res	Minor	[illegible]	[illegible]	Medium	[illegible]	Medium	-
4.	Terrestrial Ecology	Project operation leading to animal disturbance, displacement and decline.	O	Init	Minor	[illegible]	Small	Medium	Possible	Medium	-
				Res	Minor	[illegible]	Small	Low	Possible	Low	-
5.	Ambient Air Quality	Increase in ambient concentration of air pollutants from construction activities and vehicular movement may cause health impacts on the community.	C	Init	Moderate	Short Term	Intermediate	Medium	Possible	Medium	-
				Res	Minor	Short Term	Intermediate	Low	Possible	Low	-
6.	Vibration from blasting	Vibration from blasting during the construction phase may disturb local communities.	C	Init	Moderate	Short Term	Intermediate	Medium	Possible	Medium	-
				Res	Minor	Medium Term	Small	Low	Possible	Low	-
7.	Hazards of Fly Rock from Blasting	Blasting may pose a safety hazard due to flying debris	C	Init	[illegible]	Short Term	Intermediate	Medium	Possible	Medium	-
				Res	Minor	Short Term	Small	Low	Possible	Low	-

ID	*Aspect*	*Impact*	*Phase*	*Stage*	*Magnitude*	*Timeframe*	*Spatial Scale*	*Consequence*	*Probability*	*Significance*	*+/-*
8.	Construction Nosie	Increase in ambient noise levels due to the operation of construction equipment, movement of construction traffic and blasting may create a nuisance for nearby communities and visiting tourists.	C	Init	Moderate	Short Term	Intermediate	Medium	Possible	Medium	-
				Res	Minor	Short Term	Small	Low	Possible	Low	-
9.	Water Availability and Quality	Alterations of natural passage of springs due to tunnel construction may disrupt the water availability at mountain springs for the local community.	C	Init	[redacted]	[redacted]	Intermediate	[redacted]	Possible	[redacted]	-
				Res	Minor	Medium Term	Intermediate	Low	Possible	Low	-
10.	Water Availability and Quality	Use of local water resources for construction activities may reduce the water availability for local communities.	C	Init	Moderate	Short Term	Intermediate	Medium	Possible	Medium	-
				Res	Minor	Short Term	Small	Low	Unlikely	Low	-
11.	Water Availability and Quality	Seepage from the reservoir will cause an increase in the groundwater level near the dam site and an increase in flow of the springs.	C	Init	Minor	[redacted]	Intermediate	Medium	Possible	Medium	+
				Res	Minor		Intermediate	Medium	Possible	Medium	+
12.	Soil, Topography and Land Stability	Contamination of soil as a result of accidental release of solvents, oils, and lubricants can degrade soil fertility and agricultural productivity.	C	Init	Moderate	Medium	Small	Medium	Possible	Medium	-
				Res	Minor	Medium	Small	Low	Unlikely	Low	-

ID	Aspect	Impact	Phase	Stage	Magnitude	Timeframe	Spatial Scale	Consequence	Probability	Significance	+/-
13.	Soil, Topography and Land Stability	Land clearing, excavation, tunnel boring and other construction activities may loosen the top soil in the Project area resulting in loss of soil and possible acceleration of soil erosion and land sliding, especially in the wet season.	C	Init	Moderate	Medium Term	Intermediate	Medium	[illegible]	Medium	-
				Res	Minor	Short Term	Small	Low	Possible	Low	-
14.	Soil, Topography and Land Stability	Increased erosion and sediment load entering the river from bunds and sediment ponds during the construction phase and as a consequence of the failure of spoil dumping sites.	C, O	Init	Moderate	[illegible]	Intermediate	[illegible]	Possible	[illegible]	-
				Res	Moderate	Medium Term	Intermediate	Medium	Unlikely	Low	-
15.	Traffic and Road	Increased risk to community safety due to increased traffic during the construction phase near communities.	O	Init	[illegible]	Short Term	Small	Medium	Possible	Medium	-
				Res	Minor	Short Term	Small	Low	Possible	Low	-
16.	Livelihood and Well-being	Direct, indirect and induced employment at the local level, resulting in increased prosperity and wellbeing due to higher and more stable incomes of people.	C, O	Init	Minor	[illegible]	[illegible]	Medium	Possible	Medium	+
				Res	Moderate	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	+
17.	Livelihood and Well-being	Increase in the stock of skilled human capital due to transfer of knowledge and skill under the Project resulting in enhanced productivity of local labor.	C, O	Init	Minor	[illegible]	Intermediate	Medium	Possible	Medium	+
				Res	Moderate	[illegible]	[illegible]	[illegible]	Possible	[illegible]	+
18.	Livelihood and Well-being	Loss of income from sediment mining due to inundation and changes in the pattern of sediment deposition following construction of the dam.	O	Init	Minor	Short term	Intermediate	Medium	[illegible]	Medium	-
				Res	Minor	Short term	Small	Low	Possible	Low	-

ID	*Aspect*	*Impact*	*Phase*	*Stage*	*Magnitude*	*Timeframe*	*Spatial Scale*	*Consequence*	*Probability*	*Significance*	*+/-*
19.	Livelihood and Well-being	Loss of assets and livelihood as a result of land acquired for the Project.	D, C	Init	Minor	Short term	Intermediate	Medium	[illegible]	Medium	-
				Res	Minor	Short term	Small	Low	Possible	Low	-
20.	Socio-Cultural Impacts	Increase in population due to in-migration of job seekers (in-migrants) leading to pressure on existing social infrastructure and services in the Study Area.	C	Init	Moderate	Medium	Intermediate	Medium	Possible	Medium	-
				Res	Minor	Medium	Intermediate	Low	Possible	Low	-
21.	Socio-Cultural Impacts	Disputes over the distribution of Project employment within and between Socioeconomic Study Area inhabitants and the in-migrants resulting in social unrest.	C	Init	Moderate	Medium	Intermediate	Medium	Possible	Medium	-
				Res	Minor	Short term	Intermediate	Low	Possible	Low	-
22.	Climate Change	Occurrence of an increase in extreme events (e.g. increase of PMP is likely, with medium confidence, under future climate scenarios; however, extremes events themselves and a dam break even under such conditions are unlikely), that the dam is not designed to withhold, leading to catastrophic dam failure and potential damage to property and loss of life downstream.	C, O	Init	[illegible]	Low Frequency	[illegible]	[illegible]	Possible	[illegible]	-
				Res	Moderate	Low Frequency	[illegible]	Medium	Possible	Medium	-

C: Construction (and pre-Construction); O: Operation; Init: Initial; Res: Residual; Duration: Short (less than four years), Long (beyond the life of the Project)
Frequency: High (more than 10 times a year), Low (less than once a year)

Cumulative Impact Assessment

The methodology used for the CIA of Sharmai HPP has been adapted from the guidelines of the International Finance Corporation. The study area selected for the CIA includes the Panjkora River, on which hydropower projects are planned. The temporal scope of the CIA spans a period of 31 years up till the year 2050.

Unlike in other basins such as those in Jhelum and Poonch, impacts on socioeconomic aspects are limited as river-dependent socioeconomic activities are limited. The Project is expected to improve infrastructure and employment conditions in the area.

Impact on Fish Fauna

The impacts of development of multiple hydropower projects will mainly be on the long-distance migratory fish species, including the Endangered Mahaseer, Vulnerable Alwan Snow Trout and the Suckerhead. Loss of connectivity will confine the populations of these fish species, mainly of the Alwan Snow Trout as it is found in the upper reaches of the Panjkora River where a cascade of hydropower projects is planned. The fragmented populations will be under stress due to their inability to migrate downstream in winters to avoid colder waters upstream. Isolation will also increase in-breeding. Other non-migratory fish species adapted to life in riverine conditions will be impacted due to conversion of part of the river into a lentic or lake habitat.

Management Strategy and Measures

This CIA recommends good practice measures that are important for the protection of biodiversity in the long term to be implemented and followed by hydropower project developers and other stakeholders in the basin. These include:

- Assessing the feasibility of fish ladders on a case by case basis
- Developing Projects to balance economic value with environmental considerations.
- Limiting the number of projects especially in the downstream reaches where the Endangered Mahaseer is present.
- Holistic environmental flow assessments based on World Bank Guidelines

The CIA also recommends basin-wide management. This includes the following actions:

- Standardization of assessment methods using IFC Guidelines,
- Active regulation by the KP EPA using World Bank Guidelines,
- Increased support to the government departments involved in protection of biodiversity and habitat,
- Greater coordination between developers for achieving synergistic benefits and a balance between environmental and energy requirements in the basin and
- Support to research in the basin for advances in understanding of river ecology

Cost Estimate for Environmental Management

The total cost for Environmental Management has been estimated as USD 2,694,316/- for the construction phase and USD 455,744/- annually for the operation phase.