CHAPTER-IV

DESIGN ASPECTS

4.1 Engineering Assessment

As already discussed in the foregoing chapters, the Ken – Betwa link project envisages diversion of water from Ken river to the Betwa river as mandated in the Memorandum of understanding between the state Governments of Madhya Pradesh and Uttar Pradesh. In order to rope in the vast experience of other central orgnisations in relevant fields and also to complete the DPR preparation of this important project within the scheduled time, the works related with the design aspects of various components of the project have been carried out by different concerned Directorates of CWC, based on the inputs from GSI, CEA, NIH, IIT, Delhi, IIT, Roorkee, CSMRS, etc. Design philosophy for the components, their preliminary design, along with underlying consideration, assumptions and parameters and the basis for selecting the parameters form the subject of the chapter. The salient details of the designs carried out by respective directorates of CWC in respect of various main components are briefly discussed in this chapter.

4.1.1 General

The Ken-Betwa Link (KBL) Project mainly envisages construction of Daudhan dam proposed on Ken river, upstream of existing Gangau weir. An upper level link tunnel is planned on the left flank of the reservoir which leads the water into the KBL canal. A lower level tunnel is proposed on the left flank of the reservoir which leads the water into Ken Multipurpose Project (KMPP) canal system after generating power. Two power houses have been proposed. Besides this, dam and barrages are proposed in Upper Betwa which also form part of the Link project.

Hydropower is proposed to be generated from Daudhan storage reservoir at two locations. The first power house is proposed downstream of Daudhan dam and is referred to as Dam-Toe Power House (PH-I), while the second power house is proposed at the exit of the lower level tunnel that carries the water from reservoir to the KMPP canal system and is referred to as Lower Level Tunnel Power House (PH-II). The tailrace water from PH-II is led into KMPP canal system. Central Electricity Authority has carried out the power potential studies for the two power houses and proposed the installed capacity for PH-I as 60 MW (2x30MW) and for PH-II as 18 MW (3x6 MW).

An Index map of the entire Ken – Betwa Link Project (including upper Betwa projects) is given as Plate:1.1 in Drawing volume – III.

Thus, the main components of the Ken Betwa link project comprises of

I Daudhan dam

1. An 1233 m long earth fill dam across Ken River at Daudhan with FRL 288.00 m
2. A concrete dam of 798 m long (including 536 m long over flow section.
3. An upper level tunnel (1.929 km long including length of transition) about 170 m from the left flank of Daudhan dam for taking water to the link canal.
4. A lower level tunnel (1.1 km long) about 100 m from the left flank of Daudhan dam for taking water to the Power House – II and from there to the Ken LBC (proposed in the erstwhile Ken multipurpose Project).
5. Power House-I (2x30 MW) to the right of spillway at the toe of Daudhan dam.
6. Power House-II (3x6 MW) at the outlet of lower level tunnel.

II Canal Systems from Daudhan Dam

1. A link canal 218.695 km long (excluding length of Upper level tunnel) with off take FSL of 257.000 m to existing Barwa sagar reservoir with FRL 219.200 m across Barwa nallah, a tributary of Betwa river.
2. A 7.1 km long Barwa nallah (remodeling only) to carry diverted waters from Barwa Sagar to Betwa river at 12.75 km upstream of existing Parichha weir.
3. A 2.2 km long connecting canal from Power House-II to Ken LBC with an FSL of 246.000 m
4. Ken LBC (57.3 km long, modified) proposed under Ken Multi Purpose Project (KMPP) (which is replaced by Daudhan dam).
5. A bye pass channel form lower level tunnel to meet irrigation requirements of Ken LBC when Power House-II is not in operation.
6. Major Cross-drainage/Cross-masonry structures.

A general layout plan of the Daudhan dam and appurtenant works (KBL-7230-P-1002) is appended as Plate. 4.1 in Drawing volume – III.

4.1.2 Geology, Seismicity and Foundation

(i) Geology

The proposed site of the Daudhan dam site is more or less flat with rolling undulations on both the flanks of the Ken river exhibiting elevations roughly between 216m (deepest river bed level) and 300m (right abutment hill). The abutment slopes are gentle and stable. The dam alignment is occupied by hard, compact and massive beds of sandstone(quartz arenite) exposed on the left flank of the Ken river, thinly bedded sequences of siltstone/sandstone and flagstone with shaly partings in the river section and thinly bedded to laminated black siltstone, shale with porcellanite and chert beds on the right abut hill in succession. A considerable part of dam alignment is occupied by flood plain alluvium forming sandy terraces on either side of the Ken river.

(ii) Seismicity

As already mentioned in the Chapter – II, the studies for site-specific design earthquake parameters for the proposed Daudhan Dam area have been carried out by the Department of Earthquake Engineering, Indian Institute of Technology, Roorkee in 2008. As per these seismotectonic modeling studies, the site specific design earthquake parameter is estimated to be of magnitude 6.5. The peak ground acceleration (PGA) values for Maximum Considered Earthquake (MCE) and Design Basis Earthquake (DBE) conditions are estimated to be 0.11 g and 0.06 g respectively. According to the study, the proposed Daudhan dam site in Chhatarpur district of Madhya Pradesh lies in Seismic zone-II as per the seismic zoning map of India given in the IS 1893(Part I)-2002, 'Criteria for earth quake resistant design of structures'. Zone II indicates low seismicity of the area. However, it is recorded in the history of Gangau weir that a severe earth quake shock occurred on 15th January, 1934. Historically, in the region around Daudhan dam site bounded by latitude 21.6^0 to 27.6^0 N and longitude 76.8^0 to 82.9^0 E, around 90 earthquakes having small, moderate to strong intensity have been recorded up to July, 2006. Among these, 21 earthquakes are of magnitude less than 3, while 3 earthquakes are in the range of more than 6 and the rest are in the range in between.

Necessary provision has been made in the design of embankment dam for earth quake. The full study report of IIT, Roorkee is appended as appendix 2.3.

(iii) Foundation

The foundation investigation of the proposed project site is carried through bore hole drilling and required field and laboratory investigations. Out of total 35 drill holes, 19 drill holes were drilled in the concrete dam & Power House portion and remaining in the Embankment dam portion. Based on the interpretation of subsurface data it appears that competent foundation grade sandstone (litho unit A) is available at much higher elevation on the left flank where location of spillway has been fixed on techno-economic considerations. Also the height of the concrete spillway in case of the flank spillway is considerably lesser than that of the centrally located spillway. On the other hand the bedrock available at shallow depth below the over burden in Ken river section is not sufficiently competent (Litho unit B) due to frequent shaly intercalations. Additional remedial measures for improvement of the foundation grade are required, if concrete dam is to be constructed. The topographic levels in the reservoir area varying between RL 251.0 m and 288.0 m indicate height of water column as about 36.0 m. Interpretation of exploratory data from two drill holes on right flank has reveled that litho units B, C, D continue to form the foundation media in this part of Daudhan dam. The 17 to 20 m explored depth of foundation shows very poor rock condition possibly due to inherent soft nature, higher grades of weathering and structural weakness in the rock mass of above litho units. Adequate investigation of this right abutment is required to be carried out at the pre construction stage for deciding strengthening measures for making impermeable curtain through embankment dam and for ensuring full reservoir competency.

The Geotechnical investigation for foundation strata for earth dam was carried out by CSMRS which includes conducting in situ permeability tests in Auger holes and collection of undisturbed soil samples from the auger holes as well as from the foundation trial pits excavated along the dam axis for foundation characteristics. In all, 5 holes were drilled along the dam axis at reduced distances 650m, 750m, 850m, 900m and 270m. A total number of 16 undisturbed soil samples were collected from different depths in these auger holes for conducting various laboratory tests. In addition, 3 pits were excavated in the foundation overburden along the dam axis at reduced distance 0.00m, 950m, 1100m. One undisturbed soil sample was collected from the pit at RD 950m for conducting various laboratory tests.

Based on the laboratory tests results, it is inferred that the overburden materials in general, possess predominantly silt, clay and fine sand with low to medium plasticity characteristics except few samples which exhibit high plasticity characteristics. The insitu dry density test results indicate that the foundation soil is likely to undergo differential settlement particularly at shallow depths. In view of the above, it is suggested that the soil strata at the levels may be densified by adopting appropriate densification methods. The triaxial Shear test results indicate good shear strength characteristics of the foundation materials. Based on one dimensional consolidation tests results, the foundation materials are likely to undergo low to medium compressibility. The results of Triaxial Shear tests adopted in the design are given in the following table 4.1.

Table 4.1 Results of Triaxial Shear Tests adopted in the design

Sl. No.	Sample No.	In-situ Dry Density (t/m^3)	Moisture Content (%)	Specific Gravity	Effective Cohesion	Effective Angle of Shearing Resistance	Classification as per BIS.
1.	94/1	1.41	25.00	2.68	0.22	26.9	MI
2.	3	1.74	17.30	2.76	0.30	34.8	CL
3.	6	1.59	12.20	-	0.12	31.9	MI
4.	8	1.71	11.30	2.74	0.36	32.9	CI
5.	9	1.48	20.20	2.66	0.17	27.2	CI
6.	11	2.00	16.60	2.78	0.17	25.3	CI
7.	17	1.46	25.20	2.63	0.22	25.7	CH
8.	13	1.66	17.10	2.69	0.22	24.1	MI

Evaluation of design shear parameters of Foundation soils given in the drawing No. KBL-7230-P-203 is appended as Plate. 4.2. in Drawing volume – III.

4.1.3 Alternative Studies Carried out for Selection of Site and type of Structures

i) Daudhan Dam site

The Government of Madhya Pradesh formulated proposals for Ken Multi-purpose project (KMPP) on the Ken river which is also known as Greater Gangau dam and accordingly a detailed project report (1982) had been prepared. As per the report, the project envisaged construction of a dam across the Ken river about 210 m downstream of the existing Gangau weir. The proposed FRL of the Greater Gangau dam was 278.89 m and corresponding live storage was 2062 MCM. However it was found that by the construction of the Greater Gangau dam, the existing Gangau weir, which has been functioning well since 1915, will be submerged. It was, therefore, felt better to locate the dam upstream of this weir with possibility of utilising the arrangement for generation of hydropower as a pumped storage scheme.

It was , therefore, decided to realign the K-B link with higher off take level.at feasibility stage. To ascertain the potential reservoir sites on Ken river, topo sheet studies of Ken basin were carried out in the upper reaches of Greater Gangau dam site. Three reservoir sites are identified viz. Jhalar reservoir (C.A. 18205 Sq km), Ghari Ghat reservoir (C.A. 18055 Sqkm) and Daudhan reservoir (C.A. 19633 Sq km) as shown in Fig.4.1. These sites are identified on the downstream side of the confluence of Sonar and Bearma tributaries with Ken with the obvious advantage of having adequate yield for the proposed irrigation in Ken basin as planned, as well as transfer of the requisite quantity of water through the link canal. Several parameters like submergence area, capacity of the reservoir at different elevations, number of villages affected, forest and Culturable areas under submergence etc. were studied and finally Daudhan dam site was found suitable.

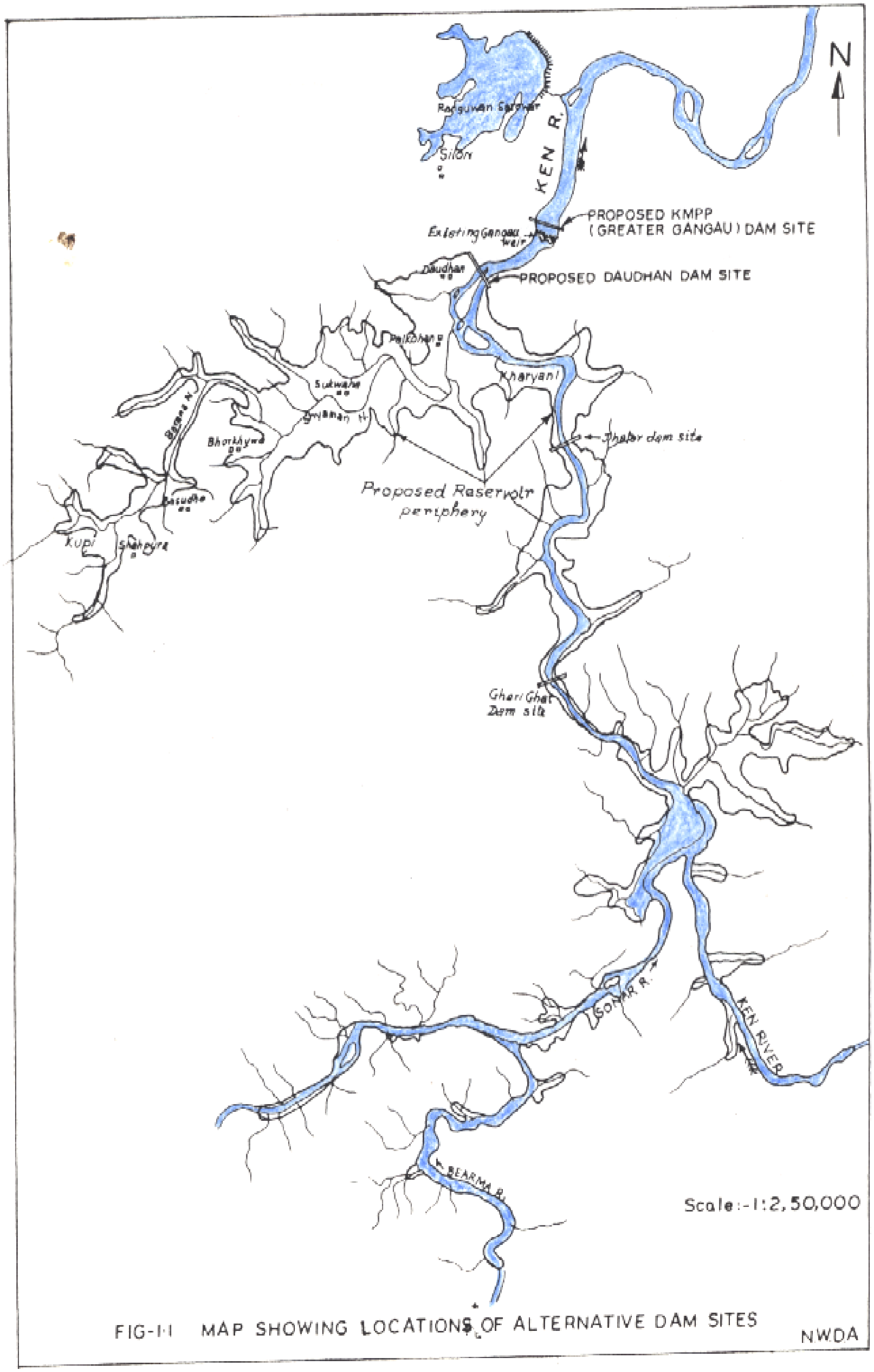

Fig. 4.1 : Map showing locations of alternative dam sites on Ken river

The catchment area of Ken river at Daudhan site is 19633 Sqkm, which is only 0.16% less than that at Greater Gangau. Therefore, the annual yield at Greater Gangau has been taken as valid at Daudhan site also. The FRL of Daudhan dam site has been proposed as 287.0 m and the corresponding gross storage capacity will be 2775 Mm^3. While keeping the general operational features of the original KMPP proposals more or less the same, the power generation is proposed as under. One Power House will utilize the irrigation releases from the reservoir at the tail race water level at 234.75 m. This Power House is proposed to function as a pumped storage power plant i.e. the water released will be further picked up by Gangau weir, which can be pumped back to generate additional power during peak period. The other Power House is planned at right bank of Pukhraha Nalla, 2 km away from the dam with a tail water level 259 m from where Ken-Betwa link canal offtakes. It can be seen that this site is also preferable to the Greater Gangau dam site because (i) it would not submerge the existing Gangau weir, (ii) it would provide enroute irrigation to higher level command along the canal alignment and (iii) additional power generation by pumped storage scheme during peak period.

Considering all the above alternatives, the present proposals as enumerated above have been formulated and finalized.

4.1.4 Choice of Final Layout of All Major Components of the Project and Reason

The location of dam site had again been reviewed jointly by the officials of CWC, GSI and NWDA prior to taken up the detailed survey and investigations for preparation of Detailed Project Report (DPR) of this link project and decided to keep the location of dam axis same as finalized at the time of preparation of feasibility study report of the project. However, during joint visit of officers of CWC, GSI and NWDA to the proposed Daudhan dam site during the month of June, 2006, the Chief Engineer, Design (NW&S), CWC, Director, GSI, Bhopal and other officials of CWC & GSI after having detailed discussions emphasiged to explore the possibility of keeping the spillway at the center of the river gorge instead of left bank of the river as proposed earlier in the feasibility report of this project.

Accordingly, as per the suggestions of GSI, few exploratory drill holes in the river gorge for specified depth were drilled to find out the preconstruction geological conditions for locating the spillway at the center of the river gorge. After carrying out the mapping of the area & logging of the drill cores, GSI vide its preconstruction geological note dated 24.7.2007 recommended on the basis of geological conditions of the area to keep the location of spillway in the left flank itself as proposed at feasibility stage of the project. The general layout plan of Daudhan dam is given in KBL-7230-P-101 is appended as Plate. 4.3 in Drawing volume – III

4.1.5 Design Flood and Sediment Studies

a) Design Flood

The design flood for Daudhan dam has been estimated using two approaches viz., (i) unit hydrograph, and (ii) frequency analysis. The design flood estimated by unit hydrograph method is found to be 57202 cumec with time to peak at 49 hours, and the time base of the hydrograph as 123 hours.

b) Reservoir Sedimentation

The type of Daudhan reservoir is considered as gorge type. For computation of rate of silt load expected at Daudhan reservoir, the silt observations carried out at Banda G & D site located on Ken river on d/s of the proposed dam site are considered as shown below.

Total sediment transport (average of 36 years) = 8356972 tonnes/year (at Banda)
Catchment area of Ken up to Banda G/D site = 25452 sq. km.
Considering 15% bed load, the total sediment = 1.15*8356972 tonnes/year
Sediment transport per year 9610518 tones/year
Average density of silt = 1.145 t/m^3
Silt inflow=total sediment/silt density
=9610518/1.145=8393465 m^3/year
Silt rate=silt inflow/catchment area=8393465/25452=329.8m^3/sq. km.

This sediment rate of 329.8 m^3/sq. km/year has been considered to be appropriate for design purpose. Out of the total catchment area of 19633 sq.km at Daudhan an area of 9439 sq.km is assumed to be intercepted by other projects in the next 15 years. Accordingly, the expected sediment deposit into Daudhan reservoir after a) 50 years and b) 100 years is computed as below.

<u>(a) After 50 years</u>
Rate of silt deposition = 329.8 m^3/sq. km/year
(i) For first 15 years the silt deposition from the catchment area of 19633 sq. km = 329.8*19633*15 = 97.12 MCM.
(ii) For the next 35 years the silt deposition from the catchment area of 9439 sq. km = 329.8*9439*35 = 108.95 MCM.
(iii) Total sediment deposition = 97.12 + 108.95 = 206.1 MCM.

<u>(b) After 100 years</u>
Rate of silt deposition = 329.8 m^3/sq. km/year
(i) For first 15 years, the silt deposition = 97.12 MCM (as estimated above).

(ii) For the next 85 years the silt deposition from the catchment area of 9439 sq. km = 329.8*9439*85 = 264.6 MCM.

(iii) Total sediment deposition = 97.12 + 264.6 = 361.73 MCM.

The total sediment deposition in the reservoir during 50 and 100 years are estimated to be 206.1 and 361.73 MCM respectively as shown above. The sediment distribution is worked out for the two periods, viz., 50 years and 100 years, by Empirical Area Reduction method. For this purpose, the FRL of 288.00 m is adopted and bed level of the reservoir is considered as 216.00 m. The new zero elevations after 50 years and 100 years have been found to be as 230.9 m and 236.4 m respectively. The total sediment during 50 and 100 years will get distributed up to and above various elevations as given in the table 4.2 below.

Table 4.2 The total sediment estimated at Daudhan.

Reservoir level	Sediment deposition in MCM	
	After 50 years	After 100 years
Up to 231 m	49.3	50.3
Above 231 m	156.8	311.4
Up to 236.4 m	85.7	98.0
Above 236.4 m	120.4	263.7
Up to 244 m	127.5	185.7
Above 244 m	78.6	176.0

4.1.1.6 Free Board

The basic requirement of free board for the earth dam is that the dam should not be overtopped under design wind conditions. The procedure adopted is based on recommendations of IS 10635-1993 (Reaffirmed 1998) "Guidelines for free board requirements in Embankment dams". For normal freeboard the wind velocity at the dam site over land has been taken as 39m/s. The computed free board comes out to be 5.05 m for and accordingly the top of dam has been provided as 293.0 m against FRL of 288.0 m. The total height of dam will be to 77 m. above river bed level of 216.0m.

The design computations of freeboard for the Daudhan earth dam have been presented in Drawing No. KBL-7230-P- 201 which is appended as Plate. 4.4 in Drawing volume – III.

4.1.1.7 River Diversion Arrangement

National Institute of Hydrology, Roorkee has analysed the flood characteristics and suggested 1 in 25 year non monsoon flood and 1 in 100 year monsoon flood at the dam site for diversion. In addition, as per the guidelines laid down in the ' Report of Ministry of Irrigation, 1980 for arriving at the diversion design flood according to criteria of risk and damage for different types of dams and barrages' the diversion floods for construction shall be follows.

(i) Diversion capacity for concrete dams and barrages

The capacity of the diversion flood for concrete dams and barrages may be less because flood higher than the designed one could be passed safely over the partly constructed dam. The following criteria would help in deciding the capacity.

a) Maximum non-monsoon flow observed at the dam site.

OR

b) 25 years return period flow, calculated on the basis of non-monsoon yearly peaks.

The higher of the two should be taken as the design flood for diversion.

(ii) For large dams

For large dams, it is desirable that 100-year return flood should be adopted for diversion works.

For the study area, the peak of daily flow data for non-monsoon months ie., from November to May is available for 25 years. Based on this data, as per clause 1(a) mentioned above, the 25- year return period flow using the GEV (PWM) model is calculated as 451 m^3/s. As per clause I(b), the maximum of the non-monsoon flow is 1117.6 m^3/s. As per clause (II), using the AMS, the 100-year return period flood is computed as 30,794 m^3/s by NIH, Roorkee.

Since the earth dam is located in the river section and concrete gravity dam on the left flank, diversion flood of 1 in 100 years return period which works out to 30,794 m^3/s has been considered in the design of diversion arrangements.

The diversion arrangements for Daudhan Dam shall consist of

a. Upstream Coffer dam part of main Earth dam

b. Downstream cofferdam

c. Diversion Channel through concrete gravity dam

The upstream cofferdam is 29 m high earth dam with top of dam as 245 m, which will be part of the main earth dam. The maximum water level shall not be more than 243 m. For seepage control through dam body upstream impervious soil layer of 1.5 m thick at top and 3 m thick at bottom has been provided whereas, for seepage through foundation a cut off trench at upstream end has been provided. In addition, one row of curtain grouting at 3m centre to centre to a depth of h/3 (h is difference of bottom of COT and maximum water level of coffer dam i.e. 243 m.) will be carried out. The down stream coffer dam of about 12 to 18 m height, with 2 (H) to 1 (V) slope both upstream & downstream, will also be provided away from the downstream toe of earth dam to protect all structures including Power House. The exact height shall be decided at the time of preconstruction stage depending upon the existing Gangu weir downstream of the dam.

Daudhan dam being a composite dam and the earthen dam is situated in the deepest level of the river, it has been planned to divert the flood of 30,794 cumecs during the construction period. As quantity of flow is very high it is not possible to pass it through the diversion tunnels, a diversion channel in the area of Spillway has been proposed. Approximate width of channel be 100m at El. 223.00m and 300m at El. 239.0m. The maximum of diversion flood will pass at El. 243.00m.

In the proposed plan, the work of construction of diversion channel will be executed prior to construction of the coffer dam. After completion of construction of coffer dam up to level of 245.00 m , the construction of spillway blocks in the area of the diversion channel be taken up. The diversion flood during construction of spillway blocks can be managed by channelizing the required flood as per reverent I.S. codes.

4.1.1.8 Construction Materials

(i) Daudhan dam

As already discussed in Chapter-II of the report, it is proposed to utilise the soil from the Daudhan reservoir area for construction of Earth dam and accordingly the soil samples have been collected.

For construction material of Earth dam 44 representative soil samples were collected from 4 different borrow areas for conducting various standard laboratory tests in order to ascertain their suitability as construction materials. Daudhan borrow area in general, possess predominantly silt and clay sizes as the other borrow area samples viz. Gangu, Palkohan and Kharyani possess, in general, predominantly silt, clay and fine sand sizes. The tri-axial shear tests results indicate good shear strength characteristics of the borrow area materials. Based on the one dimensional consolidation test results, the borrow area samples in general, possess low to medium compressibility characteristics. The result of laboratory permeability test indicates that these materials are impermeable. The results of the Triaxial shear tests adopted in the design are furnished in the following table 4.3.

Table 4.3 Results of Tri-axial Shear Tests adopted in the design

Location	SL. No.	Sample No.	Maximum Dry Density (t/m^3)	OMC (%)	Specific Gravity	Effective Cohesion	Effective Angle of Shearing Resistance (In degree)	Classification as per BIS.
DAUDHAN BORROW AREA	1.	2007/174	1.69	18.8	2.70	0.22	22.6	CI
	2.	2007/177	1.74	15.8	2.74	0.28	25.1	CI
	3.	2007/187	1.92	11.4	2.71	0.12	30.6	CL
	4.	2007/190	1.80	15.8	2.74	0.22	23.8	CL
	5.	2007/193	1.83	14.8	2.72	0.16	21.3	CL
PALKOHAN BORROW AREA	6.	2007/198	1.81	15.0	2.67	0.14	29.2	CL
	7.	2007/201	1.79	15.0	2.72	0.13	26.4	CL
	8.	2007/207	1.74	15.6	2.66	0.17	25.8	CL-CI
KHARYANI BORROW AREA	9	2007/209	1.81	17.0	2.66	0.18	22.0	CI
	10	2007/215	1.85	13.9	2.68	0.14	29.2	CL
	11	2007/217	1.84	15.9	2.65	0.20	23.2	CI
	12	2007/220	1.85	13.9	2.66	0.15	27.1	CL

All the tested borrow area soil samples in general exhibit plasticity characteristics and are suitable for construction of earthen dam. Based on the Standard Proctor compaction tests, it may be inferred that the borrow area soil samples are capable of achieving good compaction densities. The results of triaxial shear tests conducted on soil samples (remoulded at 98% of standard Proctor maximum dry density) indicate that the tested soil samples possess fairly good shear strength characteristics. Based on one dimensional consolidation test results, the borrow area soil samples tested in general possess low to medium (slightly higher side) compressibility characteristics. The results of falling head laboratory permeability tests conducted on 7 soil samples indicate that the borrow area soil samples exhibit impervious characteristics. The consensus arrived based on the four special dispersivity identification tests indicate that the tested soil samples can be categorized as Non dispersive.

(ii) Ken Betwa Link Canal

Soil samples were collected at a distance of around 5 Km except at locations where sampling was difficult due the presence of rocky strata or some other valid reasons like large depth. At fill sections of canal, representative soil samples in gunny bags were collected around canal alignment for laboratory evaluation for their suitability as construction material. Similarly, wherever significant cutting sections of canal were involved, undisturbed soil samples in core cutters (100 mm dia) were collected for laboratory determination of their Index and engineering characteristics at in situ density.

4.1.1.9 Details of the Model Studies for Important Structures

No model studies work carried at al all stage. However the same may be taken up at preconstruction stage of the project, if necessary.

4.2 Daudhan Dam

4.2.1 Daudhan Earth dam

A homogenous earth dam 1233 m. long with a height of 77 m above river bed level is proposed. The upstream part of the main dam will be used as a cofferdam thereby fully utilizing the available construction material. Daudhan dam of Ken-Betwa Link Project is a 77m high dam with top at EL. 293.00m and having river bed EL. of 216.0m. The left side of the dam is wraped around non-overflow blocks of 120m length. The right flank is resting on hill slope. Diversion arrangement consists of 29 m. high upstream cofferdam which is a part of the earth dam. Downstream cofferdam will not be a part of earth dam. Its height and other details will be finalized at the time of preparation of construction drawings. The layout of the dam and appurtenant structures as available in Drawing No. KBL-7230-P- 101 is appended as Plate. 4.3 in Drawing volume - III. Upstream cofferdam section are given in drawing No. KBL – 7230 –P-208 is appended as Plate. 4.5 in Drawing volume -III.

Earth Dam Section

The geometry of the earth dam has been provided taking into account the structural safety, Seismic aspects, seepage control etc. Detailed investigation for construction material is to be undertaken before actual construction is taken up. The dam section consists of zoned section. The upstream face has slope varying from 2.5(H):1(V) to 3.25(H):1(V) while downstream face has slope varying from 2.5(H):1(V) to 3.5(H):1(V). The upstream face is protected by a hand placed riprap of 0.6m.thickness laid over 0.2 m. thick crushed stone/aggregate and 0.2 m. thick sand filter. There are four berms each 6m. wide on upstream face at EL 275.0m, EL260.0m, EL245.0m has EL230.0m. The downstream face has four berms, two 6m. wide at EL 275.0m & EL260.0m one 15 m wide at El 245.0m and one 7m. wide at EL 230.0m. The material of the embankment dam is mainly CI & CL. Therefore o.6m thick sand drains at 5 m interval have been provided in the upstream draw down portion to relieve pore pressure in the event of draw down. Rock toe of maximum height of 14 m. is provided in the downstream portion from ground level to EL 230.0m.The top width of dam is 8.0m. The typical section of dam has been presented in Drawing No KBL-7230-P-207 is appended as Plate. 4.6 in Drawing volume - III.

Evaluation of Design Parameters.

For estimation of shear strength parameters of borrow area of embankment materials, 75% dependable values of borrow area results were adopted. These are presented in Drawing No. KBL-7230-P-202 is appended as Plate. 4.7 in Drawing volume-III. For estimation of shear strength properties of overburden material in foundation ,100% dependable values were adopted. As already mentioned, the evaluation of shear parameters of foundation overburden are available in Drawing No. KBL-7230-P-203 is appended as Plate. 4.2 in Drawing volume-III.

The properties are summarized below in table 4.4.

Table 4.4 The shear properties of over burden soil at Daudhan dam site

Sl. No.	Material	Saturated Unit Weight	Cohesion C'	Angle of shearing Φ′
1.	Embankment Material	2.13 t/m^3	1.6 t/m^2	24^0
2.	Foundation	2.03 t/m^3	1.7 t/m^2	24^0
3.	Rockfill	2.13 t/m^3	0	40^0

Stability Analysis

The stability analysis has been carried out by a software made in-house based on Swedish slip circle or Fellinious method. The minimum factors of safety obtained for various loading conditions is indicated in the following table and also available in the drawings No. KBL-7230-P-204 to 206 are appended as Plates No. 4.8, 4.9 and 4.10 respectively in drawing volume-III. The factors of safety considered are furnished in the table 4.5

Table 4.5 Minimum Factor of Safety

SL.	LOADING CONDITION	UPSTREAM	DOWNSTREAM With TWL	DOWNSTREAM Without TWL
1.	Steady Seepage without EQ	1.627	1.479	1.690
2.	Steady Seepage with EQ	1.224	1.112	1.347
3.	Sudden Drawdown	1.307	-	1.292

Zoning of Dam Section

The dam section is comprises of 5 kinds of material viz. embankment material, sand drains, filter (Sand layer & crushed stone layer), compacted rockfill in rock toe and hand placed riprap. At the time of construction, material excavated from the gravity dam for making diversion channel, if found suitable, could also be used for the cofferdam construction. This will make the construction faster for coffer dam. The embankment material should be compacted to an average density of 100% of standard proctor density subjected to minimum density equal to 98% of proctor density. Zoning of the dam section would have to be finalized after availability of the strength parameters and gradation of the materials from required excavation at the pre-construction stage.

Drawing showing details of earth dam u/s & d/s wraparound details of Daudhan earth dam drawing No. KBL-7230-P-214 and appended as Plate. 4.11 in Drawing volume - III.

Protection of Slopes of Dam Section.

The upstream face is protected by a hand placed riprap of 0.6m.thickness laid over 0.2m. thick crushed stone/aggregate and 0.2m. thick sand filter. This riprap shall consists of angular rock fragments with 80% fragments heavier than 50 Kg. In the remaining 20% not more than 50% fragments shall be smaller than 75mm in the minimum dimension. However, this will be finalized after detailed survey prior to construction stage. Turfing has been provided on the down stream face. Further for the collection of the rainfall water and seepage water surface drainage and a toe drain has been provided. The details are available in drawings No. KBL-7230-P-213 is appended as Plate. 4.12 in Drawing volume - III.

Filter

The filter shall be compacted to an average relative density of 75% with a minimum acceptable relative density of 70%. Filter material shall satisfy the following criteria:

$$\frac{\text{D15 of Filter material}}{\text{D15 of base material}} < 5$$

$$\frac{\text{D15 of filter material}}{\text{D85 of base material}} < 5$$

Where D15 and D85 are 15% and 85% finer material by weight. Further the grain size curve of the filter material will be parallel to that of the base material. Maximum particle size of the filter material shall not be more than 75mm and it should not contain more that 5% of the material finer than 0.074mm (200 mm Sieve).

Asiesmic Design Considerations

For asiesmic design Earth quake coefficient, α_h as 0.6g and α_v as 0.4g has been adopted for Pseudo-static analysis of the section. However these coefficients shall be recalculated based on site specific response spectra. Adequate factor of safety has been provided against slip failure of slopes during earthquake combined with steady seepage. A wide abutment core-contact will ensure the safety against opening of joint during the earthquake.

Earthquake coefficients adopted will be got approved by the National Committee of Seismic Design Parameters at pre construction stage.

Foundation Treatment

The foundation treatment has been decided based on the geological interpretation carried out from the data of drill holes and other investigations. To control the under seepage, grout curtain in three rows staggered at 3m centre to centre up to a depth half the hydraulic head measured above the base of the cut-off trench up to FRL have been provided. The minimum depth of curtain grouting shall be 16 m. In addition, peripheral grouting shall be carried out at FRL. At the time of pre construction/construction the depth of curtain shall be decided in consultation with Geologist.The post-grouting permeability should be les than 5 lugeons. The foundation treatment of right abutment is based on limited investigation carried out at DPR stage. The detailed investigation is to be carried out at pre construction stage for suggesting treatment to control seepage through right abutment. Based on the investigation foundation treatment shall to be provided. The foundation treatment details are presented in Drawing No KBL-7230-P-209 is appended as Plate. 4.13 in Drawing volume - III.

Drawing showing plan and L-section of Daudhan earth dam (Drawing No.KBL-7230-P-210), and earth dam section at various RD's (Drawing No.KBL-7230-P-211and 212) are appended as Plates No. 4.14, 4.15 and 4.15 respectively in Drawing volume - III.

4.2.2 Daudhan Concrete dam / Non over Flow Section / stability analysis / Spillway section

Daudhan dam consists 30.0m long NOF blocks on the left side, 232.0 m. long on the right side (112m long for Power and 120m long NOF blocks for Earthen dam) with a 536.0m long spillway on the left flank (27 bays of 16.0m width with 26 piers of 4.0m width) to pass a peak flood of 57,202 cumecs (PMF).

(i) Non over Flow Section

The stability analysis for NON OVERFLOW SECTION has been done at the deepest foundation level i.e., 232.0 m as per IS-6512-1984 for all the following seven load combination i.e.:

i) A (construction condition),
ii) B (Normal Operating Condition),
iii) C (Flood Discharge Condition),
iv) D (Combination A with earthquake),
v) E (Combination B with earthquake but no ice).
vi) F (Combination C but with extreme uplift drains inoperative) and
vii) G (combination E but with extreme uplift drains inoperative).

The stresses obtained are compressive in all above mentioned conditions and the factor of safety against sliding are more than 1 as per IS.6512-1984. The following data have been adopted for the design of Non Overflow (NOF) sections.

a) Maximum water Level (MWL)	=	288.00 m
b) Full Reservoir Level (FWL)	=	288.00 m
c) Maximum Tail Water Level (TWL)	=	241.00 m
d) Minimum Tail water Level	=	233.00 m
e) Silt Level in m.	=	236.40 m
f) Top width of dam	=	8.0 m
g) U/S slope (H:1)	=	0.100
h) Horizontal seismic coefficients	=	0.06 g
i) Vertical seismic coefficients	=	0.04 g
j) D/S slope	=	0.85
k) Cohesion	=	500 T/sqm
l) Angle of internal friction	=	55 degrees
m) Width of drainage gallery	=	2.0 m
n) Ht. of drainage gallery	=	2.5 m

The proposed Daudhan dam site falls in zone II indicating low seismicity of the area. The value of horizontal and vertical seismic coeff. adopted are 0.06 g & 0.04 g respectively, which is subject to the approval by National Committee on seismic design parameters. The cohesion and angle of internal friction of the rock has been taken as 500 t/sq.m and 55^0 respectively as furnished by the field investigation engineers. In case of any appreciable change in these values, the stability analysis has to be revised. The drawing showing maximum non-overflow section, zoning of material (NOF section) and water stop details are enclosed as KBL-7230-P-103, KBL-7230-P-104 and KBL-7230-P-107 are appended as Plates No. 4.17, 4.18 and 4.19 respectively in Drawing volume - III.

(ii) Spillway of Daudhan Dam

The overflow section has been designed to pass peak flood of 57,202 cumec (PMF) keeping crest level at 270.00 m and maximum water level 288.00m. The length of the spillway section (over flow section) Vertical upstream face has been provided from El. 260.00 m to 268.40 m and below El. 260.00 m a slope of 1 in 10 in upstream face has been provided up to foundation level. The crest shape, discharge coefficients and d/s water surface and water nappe profiles have been worked out as per relevant I.S. codes.

The stability analysis for OVERFLOW section has also been done at the deepest foundation level i.e. 223.0m as per IS-6512-1984 for all the seven load combination as mentioned above for non over flow section. The data used for design is as given below.

a) Radial gate trunion elevation level	=	277.00 m

b) Bridge weight	=	15 t/m
c) Top width	=	8.0 m
d) Width of block	=	20 m
e) Elevation of T.P.	=	256.33 m
f) Top Bridge level	=	293.00 m
g) Cm	=	0.735
h) Top of crest	=	270.00 m

The stresses obtained are compressive except in "G" condition which is within permissible limit and the factor of safety against sliding are more than 1 as per IS.6512-1984. The drawing showing maximum overflow section, zoning of material (OF section) and Water stop details are enclosed as KBL-7230-P-102, KBL-7230-P-105 and KBL-7230-p-106 are appended as Plate. 4.20, 4.21 and 4.22 respectively in drawing volume-III.

(iii) Daudhan Dam Gates

Radial gates shall be provided on Spillway of Daudhan Dam. There will be Irrigation Sluice Gates and various Intake Gates. The description of various gates are as follows.

A) Spillway Radial Gates

27 Nos. Spillway Radial Gates of size 16000mm X 18500mm will be provided to control the flow of spillway. These gates shall be operated by down stream twin cylinder Hydraulic Hoists. The Hoist cylinders shall be pivoted on the Hoist Support structure mounted on the pier. The power pack shall be installed on the top of the pier. Each gate shall have individual Power Pack. However, provision shall be made to operate the adjacent hoists in case of emergency.

The gate shall consist of curved skin plate stiffened by vertical stiffeners. The vertical stiffeners shall be supported by four Horizontal girders. The load from horizontal girders shall be transmitted to trunnion by three radial arms on each side of gate. The load from trunnion shall be further transferred to concrete by independent anchorage system. Thrust block/Tie Beam shall be provided to transfer the lateral load of radial arms. Suitable bracings shall be provided for Horizontal girder and arms. The sill level of gate shall be 269.50 m and trunnion shall be installed at 277.0 m. The radius of gate leaf shall be kept as 21.5 m. Bottom seal of gate shall be provided as wedge type. The side seals shall be of Z type and will move on curved seal seat. The gate shall be designed as per IS: 4623.

Drawing of Daudhan Dam spillway radial gates are KBL-7320-P-1501 7 KBL-7231-P-1502 are appended as Plate. 4.23 and 4.24 respectively in Drawing volume –III.

B) Spillway Stoplogs

Three sets of stoplogs shall be provided to carry out the maintenance of spillway Radial Gate. Each set of stoplogs shall consist of ten units of 16000mm X 1900mm. Bottom Unit shall be non interchangeable type. All other units shall be interchangeable. The stoplogs shall be operated under balanced water head condition except top most unit which shall be lifted under unbalanced water head condition for one gate unit height water head.

Downstream skin plate and downstream sealings shall be provided. Wedge type bottom seal and solid bulb type side seals shall be provided to make the gate water tight. The Stoplogs shall be operated by gantry crane moving on the bridge. The Stoplogs shall be connected to gantry crane through Lifting Beam and Ramshorn Hook. Drawing of stoplogs for Daudhan Dam spillway radial gate is KBL-7230-P-1503 is appended as Plate. 4.25 in Drawing volume – III.

C) Gantry Crane

The Spillway Stoplogs shall be operated by moving Gantry Crane. The tentative capacity of gantry crane shall be 60T. The Gantry Crane shall have hoist machinery mounted on trolley. The trolley shall of moving type. The crane structure along with trolley shall be capable of moving in longitudinal direction with the help of LT travel mechanism. Suitable counter weight shall be provided to make the crane stable for different stability conditions. The crane shall be designed as per IS: 3177 and IS: 807. 10 T Auxiliary hoist shall be provided to operate the irrigation sluice Stoplogs.

(iv) Energy Dissipation Arrangement

Energy dissipation arrangement envisaged is that of ski-jump bucket type. IS-7365-1985 stipulates that ski-jump bucket is used when the tail water depth is insufficient for the formation of hydraulic jump and when the bed of the river channel on d/s consists of sound rock. The jet coming down the spillway is thrown sufficiently away from the toe of the dam. Considering hard rock at d/s of the bucket, no specific scouring arrangement is being proposed. However, based on site condition and model studies sufficient precaution would be taken in design at preconstruction stage.

(v) Constraints Felt during Concrete Dam Design

Following constraints have been felt during concrete dam design:

1. Studies are based on limited geotechnical data/information and hence detailed geological appraisal needs to be carried out at pre-construction stage.

2. Model studies should be carried out to ascertain the efficiency of energy dissipation arrangement and layout and profile of approach and spill channel at the pre-construction stage.
3. Pre-designed plunge pool with side protection may be provided after carrying out adequate model studies

4.2.3. Openings through Daudhan Dam

In all four openings are provided in the concrete section of the daudhan dam of which 2 openings act as intake for the dam toe Power House(PH-I) and 2 openings act as river sluices to release water to meet down stream requirement when the PH-I is not in operation. While the design aspects relating to Power House intake are discussed under the head Power House, the same relating to river sluices are discussed below. A provision has been kept to pass continuous flow of water for the down stream requirement. It has been planned to release a water of 40 cumec through sluice when the water level in reservoir reduces to below El. 252.00m. However the sluices can be operated when the water is at crest. Accordingly two number of sluices size 1.6mx2.4 m have been proposed at EL.237.2 in block no. 27. Gates for river sluice will be operated through Operation Chamber provided at El 242.60m . There will be direct approach to this gallery from the Elevator shaft and staircase provided in block No. 30

(i) Dam Sluice Gate

Two numbers sluice shall be provided in Dam to reduce the flow in the main river course. Provision of service, emergency and bulkhead gates shall be provided. Service and Emergency gates shall be slide type. These gates shall be operated under unbalance water head condition by Hydraulic Hoists. The hoist chamber shall be sealed by providing bonnet cover. Solid bulb rubber seals shall be provided on the d/s side to make the gate water tight. Suitable air vent shall be provided at the d/s of service gate and emergency gate. Model study for the gate is proposed to be carried out at the time of execution to assess hydro-dynamic forces and air requirement. Steel liners shall also be provided to avoid erosion of concrete in the vicinity of gates.

Drawing of irrigation sluice gate of Daudhan Dam KBL-7230-P-1504 & 1505 are appended as Plate. 4.26 & 4.27 respectively in Drawing volume - III. One Fixed Wheel Bulkhead Gate shall also be provided to carry out the maintenance of emergency gates. Spring loading guide shall be provided to guide the movement of bulkhead gates. Emergency and Service gates shall be designed to operate water level corresponding to the spillway crest. The Bulkhead gate shall be operated under balanced water head conditions by means of auxiliary hoist of spillway gantry crane. The drawing showing sluice arrangements and Training and divide wall details are given in the drawing volume as KBL-7230-P-108 and KBL-7230-P-109 are appended as Plate. 4.28 and 4.29 respectively in Drawing volume - III.

4.3 Canals from Daudhan Dam

In all two canal systems are proposed from Daudhan dam project viz., A) Ken-Betwa link canal to divert water from Ken river to Betwa river (including en route irrigation) and B) Ken LBC to provide irrigation to the Ken command (contemplated in the erstwhile KMPP, now taken over by Daudhan dam).

4.3.1 Description of Canal Systems including Ridge/Contour/Lift Canal Capacity and Considerations for Fixing Alignments etc.

A) Ken – Betwa link canal

The Ken – Betwa link canal which envisages diversion of water to Betwa river apart from providing en route irrigation starts with a tunnel. The flow control is achieved through a regulating gate at the tunnel entry. A settling basin arrangement is proposed form which the actual canal takes off. To facilitate flow of water into Ken – Betwa link canal, an Upper level tunnel of 1.929 km length is proposed at the Daudhan reservoir. Various design aspects of the Upper level tunnel are discussed in the following paras.

(i) Upper level tunnel

a) Intake

The upper level link tunnel is an irrigation tunnel to divert water from the Daudhan reservoir into the KBL canal. The discharge through the tunnel will vary depending upon the water demand throughout the year. The Intake of Upper Level Tunnel is located on the left bank upstream of Daudhan dam and 90m further upstream of Lower level tunnel intake in the reservoir spread area. An independent inclined intake structure, sloping at around 10º has been provided.

The intake structure mainly comprises of concrete piers, rib beams, metallic trash racks, bell-mouth entry, emergency and service (regulating) gates and a maintenance platform at the top. The concrete piers and rib beams have been provided with suitable cut water and ease water shapes for stream lining the water flow. The intake is designed for a maximum discharge of 76.226 cumecs. The tunnel is 6 m diameter, D-shaped and is laid at a longitudinal slope of 1:800. The length of this tunnel is 1928.91 m. The invert level at the start of the tunnel is EL256.0 m and at the end of the exit portal is EL 253.601 m. The operating platform for mechanical trash rack cleaning is provided at EL 293.0 m, above MWL/FRL for ease of maintenance in all seasons. The trash racks have been provided up to EL 270.229 m. The trash rack grooves extend beyond EL 270.229m up to the top platform.

A metallic trash rack structure is provided to prevent entry of trash, debris and other floating matter into the tunnel. Being a large storage reservoir, the sediments are expected to significantly settle down in the reservoir itself. The invert level at intake is well above the expected reservoir sedimentation level after 100 years. The center to center spacing between the trash bars is provided as 80 mm. The total number of trash rack panels for the intake structure works out to 18. Bell mouth transition has been provided at intake entrance for smooth flow of water. Beyond the bell mouth entry, provision has been made for two gates, emergency gate and service (regulating) gate. Provision for air vent has been made downstream of the regulating gate. After the gates, a transition of 6.60 m length from square to D-shape has been provided. The service gate proposed will be regulating type to supply the required water demand. High velocities in the gate area and hydraulic jump formation are expected depending upon the extent of gate opening. To protect the concrete tunnel lining from abrasion, steel lining is proposed downstream of service gate for a distance of 25 m which is to be firmed up based on model studies. After the exit portal, a transition of 6.6 m length from D-shape tunnel to 6 m x 6 m square concrete box section has been provided which will lead the water further into the KBL canal after crossing Pukhra Nallah. Hydraulic model studies are to be carried out at detailed design stage for firming up the hydraulic details of intake.

b) Rock Support System

The excavation of the upper level link tunnel is proposed to be carried out by conventional drill and blast method from the intake end and exit portal side. Hence no provision for adit is made. GSI has carried out the geological investigations and furnished their report on the likely rock strata to be encountered along the alignment of the tunnel. The tunneling media expected along the tunnel include massive sandstone, gravely-pebbly conglomerate and interbedded sequence of sandstone and slaty shale and foliated quartzite. For more geological details, concerned chapter may be referred. The rock mass has been classified adopting the Geomechanic classification system (CSIR method) criteria which is as follows.

Very good rock	-RMR	= 81-100
Good rock	-RMR	= 61- 80
Fair rock	-RMR	= 41 - 60
Poor rock	-RMR	= 21 – 40
Very poor rock/Squeezing conditions	-RMR	≤ 20

The rock support system has been designed for the above classes of rock mass and basically consists of shotcrete, rockbolts and steel ribs as shown in the drawing. During excavation, 75 mm dia, 4000 mm long drainage holes is to be provided as required in seepage zones. The tunnel is proposed to be lined with M-25 grade concrete lining, 400mm thick. A typical scheme of contact and consolidation grouting has been proposed. Eight drawings pertaining to Upper Level Link Tunnel are appended as Plates No. 4.30 to 4.37 in Drawing volume – III as shown below.

	Details of drawing	Drawing No.	Plate.
1)	Intake Structure - Plan and Sectional details	- KBL-7230-P-1151 (Sheet 1/3)	4.30
2)	Intake Transition Details	- KBL-7230-P-1152 (Sheet 2/3)	4.31
3)	Trash Rack Metal Structure Details	- KBL-7230-P-1153 (Sheet 3/3)	4.32
4)	Longitudinal Section	-KBL-7230-P-1154	4.33
5)	Typical Excavation & Rock Support Details (Drill And Blast Method)	-KBL-7230-P-1155	4.34
6)	Typical Concrete Lining & Grouting Details	-KBL-7230-P-1156	4.35
7)	Exit Portal Details	-KBL-7230-P-1157	4.36
8)	Details of Transition after Exit Portal	- KBL-7230-P-1158	4.37

c) Intake Gates

One number Service Gate and one number Emergency gate of size 6000mm X 6000mm shall be provided in Upper Level Tunnel Intake. The gates shall be Fixed Wheel Vertical Lift type. The gates shall be fabricated in different parts and bolted together. Service gate shall have d/s skin plate and d/s sealing while the Emergency gate shall have u/s skin plate and u/s sealing. The gates structure shall consist of skin plate stiffened by vertical stiffeners and horizontal girders. The horizontal girders shall be supported by End vertical girders provided on each side of gate. The water thrust shall be transmitted to the concrete from horizontal girder by wheels and track embedded in the concrete. The wheels shall be mounted on self lubricating bush bearing. Sill level and top of opening shall be EL 256.00 m and EL 262.05 m respectively. The gates shall be designed to withstand water load corresponding to FRL i.e. 288.00 m. These gates shall be operated by independent rope drum hoist mounted on common support structure. The support structure shall be installed at EL 293.00 m. Each Hoist Machinery shall consist of rope drum, gears and pinions, electric motor and electro – magnetic brakes. Drawing showing details of intake of emergency gate KBL-7230-P-1510 and 1511 are appended as Plates No. 4.38 and 4.39 respectively in Drawing volume - III.

ii) Link canal

The Ken-Betwa link canal is predominantly contour canal except in small reaches where it is aligned as a ridge canal. The Ken – Betwa link canal which takes of from the desilting basin located after the exit portal of the Upper Level tunnel, traverses a total length of 218.695 km and out falls into Barua sagar reservoir. It crosses several streams, minor/major rivers and several roads. The alignment of canal starting with FSL of 256.931 (at RD 500 m) from Dhaudhan Dam to outfall in Barua Sagar (RD 218695 m) at FSL of 221.000 has been divided into four reaches. Each reach has different slope and cross section elements. The alignment has been marked on the strip survey contour sheet for every 2.5 Km along with the corresponding longitudinal section. Total 88 No of sheets have been prepared. Following data has been provided as Longitudinal Section table with every Drawing.

1	R.D (metre)
2	NSL
3	Canal Bed Level
4	Bed Slope (nH:1V)
5	Distributary FSL
6	Distributary Discharge
7	Design Discharge (Cumecs)
8	FSL
9	F S D
10	Bed Width
11	Head Loss (Bed Slope)
12	Head Loss (CD Structure)
13	Manning's Rugosity Coefficient
14	Side Slope (nH:1V)
15	Flow Velocity (at FSD)
16	Free Board
17	Canal Top Level
18	Canal Top Width
19	Depth of Cutting
20	Depth of Filling

Similar details as listed above have been given in all the L.S maps of various canals pertaining to the Ken – Betwa link project.

All the 88 sheets containing the L.S. and strip contour maps of Ken – Betwa link canal are given as KBL-7320-D-2501 to 2588 which are appended as Plates No. 4.40 to 4.127 respectively in Drawing volume -III. The alignment consists of straight lines and circular curves as per Clause 6.4 of IS 5968: 'Guidelines for planning and layout of canal system'. The range of radius are given in table 4.6 below.

Table 4.6 Radii of curves for canals

Discharge(m^3/s)	**Radius, Min**(m)
280 and above	900
Less than 280 to 200	750
Less „ 200 to 140	600
Less „ 140 to 70	450
Less ,, 70 to 40	300
Less ,, 40 to 10	200
Less „ 10 to 3	150
Less „ 3 to 0.3	100
Less „ 0.3	50

Accordingly all the curves in the Ken – Betwa link canal have been proposed with radius of 300 m.

a) Cross Section and Lining

The design of canal cross sections have been done as per provisions of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining'. The details are given in Drg No KBL-7320-D-2589 is appended as Plate. 4.128 in Drawing volume – III.

b) Capacity of K-B Link Canal

Proposed K-B Link Canal has been designed for Capacity varying from 76.23 cumec at head reach to 59.32 cumec at end. The subtractions have been done for minimum discharge in each distributary of a reach and transmission losses are not taken into account for greater flexibility of the system.

c) Shape

The shape of the canal has been selected as trapezoidal with rounded corners as per provisions of IS 3873: 'Laying cement concrete/stone slab lining on canals - Code of practice' for ease in laying of lining. The bed width reduces from head to tail with no change in the water depth so that intended flow velocity is generated with ease in construction.

d) Manning's n

As per Cl 4.1.2.1 of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining' the value of N may be taken as shown in table 4.7 given below.

Table 4.7 Value of ' n ' to be considered in lined canals

Sl No. Surface Characteristics	Value of n
i) Concrete with surface as indicated below:	
a) Formed, no finish/PCC tiles or slabs	0.018-0.020
b) Trowel float finish	0.015-0.018
c) Gunited finish	0.018-0.022
ii) Brick/tile lining	0.018-0.020
iii) U.C.R./Random rubblemasonry with pointing	0.024-0.026
iv) Asphalt	
a) Smooth	0.013-0.015
b) Rough	0.016-0.018
v) Concrete bed trowel/float finish and slopes	
a) Hammer dressed stone masonry	0.019-0.021
b) Coursed rubble masonry	0.018-0.020
c) Random rubble masonry	0.020-0.025
d) Masonry plastered	0.015-0.017
e) Stone pitched lining	0.020-0.030
vi) Gravel bed with side slope characteristics :	
a) Formed concrete	0.02-0.022
b) Random rubble in mortar	0.017-0.023
c) Dry rubble (rip-rap)	0.023-0.033

However the Manning's n is taken as 0.018 to compensate for increased resistance due to curves and for taking into account increased resistance due to deterioration of lining with time.

e) Free board

As per CL 8.2 of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining' the value of free board may be taken as per table 4.8 given below.

Table 4.8 Values of Free board required for lined canals

Canal Discharge	Free Board
More than 10 cumecs	0.75 m
Between 3 to 10 cumecs	0.60 m
1 to 3 cumecs	0.50 m
Less than 1 cumec	0.30 m
Less than 0.1 cumec (Water Course)	0.15 m

However the free board is taken as 920 mm from RD 0 to RD 216250 mm (reach 1 to 3) and 750 mm from RD 216250 mm to end (reach 4). This is provided as per free board provided in major canal systems of India and their performance and to make possible the en route irrigation of area lying between link canal and LBC of KMPP proposed earlier by M.P. WR Dept., which is currently proposed to be served by LBC by lift.

f) Side Slope

As per Cl 8.1.1 of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining' the side slope shown in table 4.9 below.

Table 4.9 Recommended side slopes for the lined canals

SI No.	Type of Soil	Side Slopes (Horizontal : Vertical)	
		In cutting	In embankment
i)	Very light loose sand to average sandy soil	2 : 1 to 3 : 1	
ii)	Sandy loam	1.5 : 1 to 2 : 1	2 : 1
iii)	Sandy gravel/murum	1.5 : 1	1.5 : 1 to 2 : 1
iv)	Black cotton	1.5 : 1 to 2.5 : 1	2 : 1 to 3.5 : 1
v)	Clayey soils	1.5 : 1 to 2 : 1	1.5 : 1 to 2.5 : 1
vi)	Rock	0.25 : 1 to 0.5 : 1	

Note — The above slopes are recommended for depth of cutting/height of embankment up to 6 m. For depth/height in excess of the above, special studies for the stability of slopes are recommended.

However the slope adopted is 1.5 h :1 V for cutting as well as for filling. The cross sectional details of the link canal are given in Drg. No. KBL-7320-D-2590 which is appended as Plate. 4.129 in Drawing volume - III.

g) Bed Slope

Bed slope of the link canal has been kept from 1 in 9,000 to 1 in 12,000 for different reaches.

4.3.2 Study of integrated network of canal system and its operation to utilise the water potential of streams crossed by main canal system by provision of storage / tail tank etc.

Detailed study of integrated network of canal system will be taken up at pre construction stage.

4.3.2.1 Description of soil profile along the canal alignment based on pit / auger holes.

Soil samples were collected at a distance of around 5 Km except at locations where sampling was difficult due the presence of rocky strata or some other valid reasons like large depth. At fill sections of canal, representative soil samples in gunny bags were collected around canal alignment for laboratory evaluation for their suitability as construction material. Similarly, wherever significant cutting sections of canal were involved, undisturbed soil samples in core cutters (100 mm dia) were collected for laboratory determination of their Index and engineering characteristics at insitu density. In all 40 samples, both disturbed and undisturbed samples were collected from different pits except complete filling locations where only disturbed samples were collected. These pits wherever possible, were excavated to various depths along/around the canal by the project authorities for sampling by CSMRS field parties. There is a large variation of soil along/around the canal length, varying from SM (Silty sand), SC (Clayey sand) to CL/CI (Clays of low to intermediate plasticity mostly). Soil is capable of attaining high densities on Proctor compaction. Good Insitu densities are also observed for the samples. Very low values of strength parameters for few samples along the canal alignment at RDs-18.150, 82.751, 108.451 & 211.800 Km was observed. These canal reaches may be further investigated at smaller interval of distance to determine the extent of such soils before construction. Free swell Index values were found from moderate to high in case of some samples. Shrinkage limit was also found to be around 11-12 % in such samples. Swelling pressure is expected in these samples and should be taken into consideration after evaluation in design. Canal sections having soil in semi-pervious range need appropriate measures for seepage control. Densities along with corresponding strength parameters should form the basis of canal design. The canal is generally in full or partial cutting whereas complete filling exits at other locations. Suitable soil available from cutting sections preferably with some plasticity can be used in fill portions of canal.

It is suggested to conduct further soil investigations at lower intervals of distance (Smaller RDs) during construction stage.

4.3.2.2 Evaluation of the Design Parameters Based on the Samples Collected Along the Canal Alignment, borrow area and Suggested Treatment for Problematic Reaches.

A) Evaluation of Design parameters

The Ken Betwa Link canal has been designed in such a way that it carries out its intended design function of conveying the discharge from Dhaudan Dam to Barua Sagar and fulfills the enroute irrigation and drinking water demand with safety, economy and efficiency. For this purpose the following design-criteria based on the relevant provisions of IS codes, and established design practices is proposed.

1. Non silting non scouring velocities should be generated for efficient sediment transport.

2. The system should be flexible to cater to any combination of requirement of irrigation of enroute command and water transfer

3. The Transmission losses should be minimum.
4. The canal should not overtop its lined section in any reach.
5. Designed discharge should be available all the distributaries at designed waterlevel.
6. Deep cutting or high embankments are to be avoided and cutting and filling should be balanced as far as possible.

7. Head loss due to various structure and section resistance should be minimum and compatible with design etc.,

In addition of above, following sub-criteria based on the relevant provisions of IS codes and established design practices have also been adopted.

8. The designed Alignment should involve balanced cutting and filling.
9. Deep cutting or high embankments are to be avoided as far as possible.
10. The canal should not overtop its lined section in any reach.
11. More than intended head loss should not take place.
12. The design should be for better overall economy
13. The water loss due seepage and evaporation should be minimum
14. The bed slope should be such that non silting non scouring velocities are generated.

15. The design should take into account efficient sediment transport.Etc

4.3.2.3 Details of Lining

The lining adopted is concrete lining of thickness 100 mm as per provisions of IS 3873: 'Laying cement concrete/stone slab lining on canals - Code of practice'. It is also proposed to deploy a HDPE geomembrance as per provisions of IS 9698: 'Lining of Canals with Polythene films-code of Practice' behind the lining to further reduce losses. The lining is proposed to be unreinforced. The pressure release arrangements as per provisions of IS 4558 : 1995 'Code of practice for under-drainage of lined canals' are provided to release water pressure behind lining due to rise in ground water level and canal empty condition. In the absence of data on the ground water level variation, it is proposed to provide 30% of the canal lined section with pressure release arrangement. The details of under drainage are illustrated in drawing No. KBL-7320-D- 2591 which is appended as Plate. 4.130 in Drawing volume – III. The details of Mechanical placement of the lining of Ken – Betwa link project are given Drg. No KBL-7320-D-2592 which is appended as Plate. 4.131 in Drawing volume – III. The lining shall however be reinforced at the junction of lining with any CD structure on upstream and downstream for a distance of 50 m.

4.3.2.4 Transmission Losses Assumed for Lined/Unlined Channel (Cumec/million sq.m)

Transmission losses for the entire length of the canal are worked out as 68 MCM at 0.6 cumec/million sq.m of wetted perimeter as per provision in IS Code 4745-1964. The details are given at Annexure 4.3.

4.3.2.5 Cut off statement showing the details of the discharge required from tail to the head considering the irrigation requirement and transmission losses in taking of channel of distributaries out falling from KB Link Canal

Details about the discharge required from tail to the head considering the irrigation requirement and transmission losses in taking of channel of distributaries outfalling from KB Link Canal will be finalised during pre construction stage.

4.3.2.6 Design Calculation for Adequacy of Canal Selections Adopted

Ken Betwa link canal : The water balance studies of Ken and Betwa basins carried out by NWDA show that 1020 MCM of surplus water will be available in the Ken basin upto Daudhan dam, which can be diverted to water short Betwa basin. The reservoir operation studies carried out for Daudhan dam show that the link canal from Daudhan dam will carry a maximum monthly flow of 185 MCM in the month of August and September .As per present proposal, surplus water transfer from Ken to Betwa basin is confined to 8 months (243 days) from July to February. The design discharge of Ken-Betwa link canal will be 76 cumec from RD 0 kms to RD 107kms, 72 cumec from RD 107km to RD 171 km, 68.42 cumec from RD 171km to RD 216.25 km and 59.32 cumec from RD 216.25 km onwards. The section changes because of the withdrawal of water in the enroute command. The length of the K-B link canal will be 231.45 km with FSL at head 259 m. A freeboard of 0.75 m has been provided in the link canal. The side slope of the canal and bed slope will be 1.5 (H) : 1 (V) and 1:10,000 respectively. The link canal will be lined completely with plain cement concrete. Design of Link Canal Cross Sections in various reaches are given at Annexure 4.4.1 to 4.4.4.

The design of canal sections has been checked for passing 25% additional design discharge. It is seen that it is possible to pass this additional discharge with about 0.40 m encroachment in the 0.75 m free board available in the canal.

The details of the canal sections are given in the Table 4.10.

Table 4.10 Reach wise Details of canal sections

Canal reach	**Design discharge (cumec)**	**Bed width (m)**	**Full supply depth (m)**	**Bed slope**	**Velocity (m/sec)**
From RD 0 to 107 km	76.23	13.2	3.44	1 in 9,000	1.09
From RD 107 to 171 km	72.04	13.1	3.44	1 in 10,000	1.03
From RD 171 km to 216.25 km	68.42	13.0	3.44	1 in 11,300	0.97
From RD 216.25 km to Terminal point	59.32	10.75	3.44	1 in 12,000	0.92

4.3.2.7 Design Discharge Data (irrigation requirement, transmission losses, evaportation losees etc) for each Distributaries Supported by Detailed Calculation for a Representative Distributary.

The Ken Betwa Link Canal is designed to provide enroute irrigation and drinking water to command area in its reach. Information on the distributary network is furnished in every L section of 2.5 km of the length of the link canal, for the distributary falling in that reach. Details of distributaries out falling from Ken-Betwa Link Canal are given in Annexure 4.5.

The shape of the distributary section adopted is cup shaped as per codal provisions and established design practices. The respective depths of various triangular portions is shown in the table. The section has been designed for maximum flow and bed slope of 1 in 10000 in the absence of detailed data. The details of cross sections of distributaries Drg No KBL-7320-D- 2599 and appended as Plate. 4.132 in Drawing volume -III.

4.3.2.8 Canal structures across Ken-Betwa Link Canal

(i) Cross regulators

21 cross head regulators will be provided at various locations of Link Canal as shown in Annexure 4.5 referred above, for providing design discharge to distributary at required water level.

The Cross Regulators are provided for the following functions:

1. To ensure pond level in front of head regulators so that the desired quantity of water may be diverted to the distributary.
2. To enable maintenance of a particular stretch of the canal.
3. To facilitate operation of canal escapes.
4. Efforts shall be made to combine bridges with the canal cross regulators to effect economy.

Head regulator has not been proposed for the distributaries in view of the small discharge to be diverted. Instead the water is diverted to distributary through MS pipe of appropriate diameter. To regulate the flow, a valve is proposed to be provided.

Following design criteria has been adopted for design of canal cross regulators:

1. The cross regulator should cause minimum head loss for water flow through the canal. To fulfill this criteria sufficient waterway of the cross regulator shall be provided.

2. The head regulator should be able to dissipate energy satisfactorily in the partial flow. To fulfill this criteria, stilling basin shall be provided.

3. The cross regulator should be able to withstand the uplift pressure. To fulfill this criteria, thickness of floor as well as pressure release valves shall be provided.

4. The cross regulator should be able to prevent piping. To fulfill this criteria sufficient length of floor is provided and cutoff wall is provided. A conservative approach is taken for a later possibility that the canal lining gets damaged at the junction of cross regulator

and there is no water on the downstream side of the cross regulator.

5. Upstream and Downstream protection. The cross regulator should be strong enough to with stand the various hydraulic forces due to water flowing over and below it. To fulfill this criteria, sufficient reinforcement shall be provided. The flexible protection is not provided as the canal is fully lined. However the canal section is reinforced on the upstream and downstream for a distance of 50 m.

The typical cross section of the cross regulator is given as Drawing No. KBL-7230-P-2600 which is appended as Plate. 4.133 in Drawing volume – III. The design of cross regulator is given at Annexure 4.6.

Gates for cross regulators

The cross head regulators shall be at various locations of Link Canal. There will be 21 Cross Regulators. There will be five Service gates of Fixed Wheel Type Vertical Lift Gate in each cross regulator. Size of service gates shall be 3500mm X 4500 mm. The service gates shall be provided with down stream skin plate and down stream sealings. Wedge type seal shall be provided for bottom sealing and music note solid bulb seals shall be provided for side sealings. The seal shall remain in contact with stainless steel seal seats to make the gate water tight. The gate structure shall consist of skin plate stiffened by vertical stiffeners and horizontal girders. The horizontal girders shall be supported by end vertical girders on each side. The water thrust will be transferred to concrete structures from the end vertical girder through wheels and wheel track. The wheels shall be mounted on self lubricating antifriction bearings. The wheel shall be made up of corrosion resistant steel. The BHN of wheel track shall be 50 BHN higher than the wheel material. 20 mm guide and two numbers guide shoes shall be provided on each side to guide the gate in grooves. The gates shall be operated by rope drum hoist of suitable capacity. The Rope Drum Hoist shall consist of hoist machinery mounted on hoist support structure. Each hoist machinery will be equipped with two rope-drums, gears, pinions, couplings, shafts, worm reducer, motor and brakes. The hoist bridge shall be supported on trestles. One set of stoplogs having two units shall be provided to carry out the maintenance of service gates. The stoplogs shall be provided with down stream skin plate and down stream sealings. The stoplogs shall be operated in balanced water head conditions. However, the top most unit can be lifted under unbalanced water head for one unit height water head. The stoplogs shall be operated with monorail crane. The stoplog units shall be stored in stoplog groove above MWL. The stoplog units shall have bronze pad sliding on stainless steel track. 20mm guide shall also be provided to guide the stoplog units. Drawing showing details of cross regulator service gates of 3500mm x 4500mm - general installation sheet 1 of 2 (Drawing No. KBL-7230-P-1515) and stoplog of 1750mm x 4500mm - general installation sheet

2 of 2 (Drawing No. KBL-7230-P-1516) are appended as Plate. 134 and 135 respectively in Drawing volume - III.

(ii) Structure at the beginning and at the end of K-B Link Canal

It is proposed that an energy dissipation cum settling basin may be provided in the beginning of canal for flowing purpose.

1. To dissipate the excess kinetic energy for water when canal is running with high water level in Dhaudhan Dam
2. To settle silt in the water as bed slope is very low
3. To act as canal escape

The details of settling basin are furnished in Drg No KBL-7320-D- 2593 which is appended as Plate. 4.136 in Drawing volume – III. The details of exit regulator which is proposed at the outfall into Barua sagar are given as Drg. No KBL-7320-D- 2594 which is appended as Plate. 4.137 in Drawing volume – III. The design particulares of the exit regulator are given at Annexure 4.7.

(iii) Cross drainage works structures

The Ken-Betwa link canal is predominantly contour canal except in small reaches where it is aligned as a ridge canal. It crosses several streams, minor/major rivers and several roads. The type of cross drainage works depend upon the catchment area of the streams and its bed level with reference to the bed level of the canal it crosses. In general, if bed level of canal is well above the HFL of a drain, an Aqueduct is the obvious choice. Super passage are proposed when the bed level of the stream is much higher than the full supply level (FSL) of the canal at the crossing. Syphon aqueduct has been provided if the HFL of the drain is higher than the canal bed. Canal syphon has been provided if the FSL of the canal is sufficiently above the bed level of the drainage trough, so that the canal can flow under symphonic action under the trough. Following CD/CM works are proposed across the Ken – Betwa link canal.

1. Major Bridges	- 7 Nos
2. Minor Bridges	- 42 Nos
3. Aqueduct	- 10 Nos
4. Syphon Aqueduct	- 12 Nos
5. Canal Syphon (at Dhasan river crossing)	- 1 Nos
6. Canal Falls	- 4 Nos.
7. Escapes	- 4 Nos.
8. Railway bridge	- 1 No.
9.	

Wherever practical, siphon aqueduct is preferred over canal siphon for lower head loss. Only one canal siphon is proposed at river Dhasan crossing.

A list of cross drainage works comprising 10 aqueducts and 12 syphon aqueducts is furnished at Annexure 4.8. Canal falls area provided at RD 202150 m (3.6 m), RD 205531 m(2.0 m) , RD 210080 m (3.0 m) and at RD 216581 m (2.0 m) keeping in view the topographical constraints. Canal escapes are proposed at Starting point (RD500 m), U/S of Dhasan (RD107201 m), at Bhatgora CR (RD 139242 m)and at the End point (RD218.675 m). The design of canal siphon across Dhasan is furnished at Annexure 4.9.

A minor bridge shall be combined with every Aqueduct, Syphon Aqueduct and Cross Regulator to ensure at least one passing over in every 2.5 km. Forty two Bridges Minors (Independent) are proposed across the length of the link canal. One railway bridge is also proposed on the Mauranipur – Jhansi railway line at R.D 187.228 km. The list of CM works i.e., Road bridges and railway bridge across the link canal is furnished as Annexure 4.10.

The design of bridges is adopted as per provisions of IRC standard designs for Bridges. The typical drawings of i) Major bridges (Drg. No. KBL-7320-D- 2595), ii) Aqueduct (Drg no. KBL-7320-D- 2596) iii) Syphon Aqueduct (Drg no. KBL-7320-D- 2596-A), iv) Canal siphon across Dhasan river (Drg no. KBL-7320-D- 2597) and v) Canal falls (Drg No KBL-7320-D-2598) are given at Plates No. 4.138, 4.139, 4.140, 4.141 and 4.142 respectively in Drawing volume- III.

(iv) Canal Outfall at Barwa Sagar .

The existing outlet i.e. waste weir on the extreme right side of Barwa Sagar shall be used to drop link canal water at RL 220.62 m into Betwa river through Barwa river. At the end of canal the structure proposed is an exit regulator to Barua Sagar and an end Cross regulator. Both are proposed with automatic control with manual override for ease in operation. This arrangement is proposed for following purpose

1. The water outfalls to Barua Sagar safely and efficiently. Barua Sagar bed may not be scoured excessively due to incoming water
2. For the case when water is not required in Barua Sagar / when it has become full, the water should be conveyed to Paricha Weir bye passing the Barua Sagar.

(v) Barwa Sagar Outfall Gates

Three Numbers automatic radial gates are proposed to transfer the water of Ken Betwa Link canal to Barua Sagar. The water level of Barua Sagar shall be kept at 220.28M. When the water of Barua Sagar rises above 220.28M, the radial gates shall close automatically. To achieve this, radial gates shall be provided with counter weight at downstream side. When the water level of Barua Sagar exceeds from pre-decided level, the counter weight shall float in the water. The buoyancy of water will reduce the force on gate due to counter weight and the gate shall close due to self weight. Model study of the gate arrangement shall be carried out at design stage to verify the working of gate. The gate structure shall have curved skin plate, stiffened by vertical stiffeners. The vertical stiffeners shall be supported by horizontal girders. The load from horizontal girders shall be transferred to trunnion through parallel arms. Combined anchorage system will transfer the water load from trunnion to concrete in bonding. Hydraulic cylinder may also be provided to avoid hunting of gate and operation of gate. The gate trunnion shall be fitted with self aligned self lubricating bush bearings of standard make. The trunnion shall be mounted above tail water level. One set of stoplogs shall also be provided to carry out maintenance of automatic Radial Gates. The stoplogs shall be operated by Gantry Crane.

(vi) Gates of End Cross Regulator of Link

Automatic radial gates shall be provided to regulate the flow of link Canals. When the water level in Link Canal rises due to closure of Barua Sagar Outfall Gates, the End Cross Regulator Gates will be lifted automatically. This will help in avoiding the spilling over of link canal. These gates shall be radial type. The radius of gate and level of trunnion shall be carefully chosen. The gate leaf will be provided with horizontal flap to give extra uplift force when water level in upstream side increases. Rope Drum Hoist shall also be provided to lift the gate at any other condition. The gate structure shall have curved skin plate, stiffened by vertical stiffeners. The vertical stiffeners shall be supported by horizontal girders. The load from horizontal girders shall be transferred to trunnion through parallel arms. Combined anchorage system will transfer the water load from trunnion to concrete in bonding

4.3.2.9 Description of Canal Systems including Ridge/Contour/Lift Canal Capacity and Considerations for Fixing Alignments etc.

Ken Left Bank canal (including connecting canal)

In the Ken Multi Purpose Project(KMPP) proposed earlier by the state Govt. of M.P. envisaged one Left Bank Canal to provide irrigation facilities in the Ken Command. Since the Daudhan dam project (a major component of the Ken – Betwa link project) replaces the earlier proposed KMPP, the Ken command area proposed to be irrigated by KMPP will now be taken over by the Daudhan dam through the same canal as proposed in the KMPP (Ken LBC). However the length of the Ken LBC will now be 57.3 instead of 67.3 km (as proposed earlier) and a connecting canal of 2.2 km length will join tail race channel of PH-II to the Ken LBC. The other salient design details are kept unaltered. The full supply depth of the Ken LBC at the head is 3.81 m and bed width is 11.585 m. The designed velocity is 1.357 m/sec. The bed slope is 1 in 6000. The condesnsed L- section of the Ken LBC as proposed in the KMPP is given in 4 sheets and appended as Plates No. 4.143,4.144,4.145 and 4.146 in Drawing volume – III.

The details of connecting canal now proposed, are discussed in the following paras.

Connecting Canal to Ken LBC has been proposed from exit of PH-II to convey the water from the Dhaudan Dam to Ken LBC Command. The design details of PH – II and intake (Lower Lever Tunnel) are discussed under the Power House component.

The alignment of the connecting canal has been marked on the strip survey contour sheet. Various data as described in para 4.1.4.1, have been provided as Longitudinal Section table with every Drawing. For details refer drawings sheet No KBL-7320-D- 2680 which is appended as Plate. 4.147 in Drawing volume - III. The KMPP connecting Canal connects the tail race of Powerhouse II at FSL 245.69 to Ken Left Bank Canal (proposed earlier under KMPP) at FSL of 245.34. The connecting canal is designed only for 2.2 km length. No Distributary or CD works are envisaged in this reach.

a) Cross Section and Lining

The design of cross section has been done as per provisions of IS 10430: Criteria for design of lined canals and guidelines for selection of type of lining. The cross section has been designed on Manning's formula. The sectional details are given in Drg No KBL-7320-D-2681 which is appended as Plate. 4.148 in Drawing volume- III.

b) Capacity

Capacity has been taken as 65.194 cumec from head reach starting from tail race of PowerhouseII to its outfaff in KMPP. The subtractions are done for minimum discharge in each distributary of a reach and transmission losses are not taken into account for greater flexibility of the system.

c) Shape

The shape has been selected as trapezoidal with rounded corners as per provisions of IS 3873: Laying cement concrete/stone slab lining on canals - Code of practice for ease in laying of lining. The bed width reduces from head to tail with no change in the water depth so that intended flow velocity is generated with ease in construction.

Other details viz., side slopes, bed slopes, free board etc., for the connecting canal are considered suitably as discussed for Ken – Betwa link canal.

d) Lining

The lining adopted is concrete lining of thickness 100 mm as per provisions of IS 3873: Laying cement concrete/stone slab lining on canals - Code of practice. It is also proposed to deploy a HDPE geomembrance as per provisions of IS 9698: Lining of Canals with Polythene films-code of Practice behind the lining to further reduce losses. The lining is proposed to be unlined. The pressure release arrangements as per provisions of IS 4558 : 1995 Code of practice for under-drainage of lined canals are provided to release water pressure behind lining due to rise in ground water level and canal empty condition. In the absence of definitive data on the ground water level variation, it is proposed to provide 30% of the canal lined section with pressure release arrangement. The lining shall however be reinforced at the junction of lining with structure on upstream and downstream for a distance of 50 m.

e) Structure at the beginning and end of Canal

In the starting and end of canal, arrangement have been proposed to safely inlet the water into canal and to outlet into Ken LBC.

f) CD Structures

No CD Structures are involved in the connecting canal.

4.4 Power House

Hydropower is proposed to be generated from Daudhan storage reservoir at two locations. The first Power House is proposed downstream of Daudhan dam and is referred to as Dam-Toe Power House (PH-I), while the second Power House is proposed at the exit of the lower level tunnel that carries the water from reservoir to the KMPP canal system and is referred to as Lower Level Tunnel Power House (PH-II). The tailrace water from PH-II is led into KMPP canal system. Central Electricity Authority has carried out the power potential studies for the two Power Houses and proposed the installed capacity for PH-I as 60 MW (2x30MW) and for PH-II as 18 MW (3x6 MW).

The overall conceptual planning of the project has been carried out by NWDA taking into consideration the agreement entered into by the two States, results of hydrological and other studies, surveys and investigations. The proposal of two tunnels i.e. upper and lower level tunnels and their elevations have accordingly been finalized by NWDA. NWDA has proposed generation of power by utilizing the water being released for irrigation through the dam and through the lower level tunnel and the head available in the reservoir. However no power is envisaged by NWDA on the Upper Level Link Tunnel.

4.4.1 Alternative studies

Alternative studies limited to finalizing the location of the intake structures and alignment of upper and lower level tunnels as well as the location of the two Power Houses is carried out as discussed in the following paras.

4.4.1.1 Power House - I

Various alternatives have been considered for the location of PH-I keeping in view the point that the tailrace water from PH-1 has to be led into the river, downstream of the dam. The proposal of locating PH-I on the left side of the spillway, downstream of the dam is resulting in too much of excavation of the left bank slopes and requires a long tail race channel. The underground option at the same location is ruled out due to non-availability of adequate rock cover in the area. After evaluating the technical and economic aspects of various alternatives, power house (PH-I) is finally proposed to be located on surface at the toe of the dam between spillway and the river.

4.4.1.2 Power House - II

Various alternatives have been considered for the location of PH-II keeping in view the point that the tailrace water of PH-II has to be led into the KMPP Left Bank canal. In the first alternative (Alt-1), the intake for the head race tunnel is proposed around 180 m upstream of the dam axis along the left bank and the tunnel is aligned ensuring adequate vertical and lateral rock cover. The surge shaft is suitably located ensuring adequate lateral rock cover. From the surge shaft, a pressure shaft leads the water into a surface power house. In the second alternative (Alt-2), attempt is made to shift the above intake location further away from the dam and is placed at around 300m from the dam axis. The headrace tunnel length reduces by around 50 m and pressure shaft length by around 100 m for the same tailrace channel length. A bend in tailrace channel is also avoided. The third alternative (Alt-3) considers locating the power house at the toe of the dam on the left side of spillway. This alternative leads to increased excavation of the left bank slopes downstream of the dam and a very long tailrace channel/tunnel. After evaluating the technical and economic aspects of various alternatives, the second alternative (Alt-2) is preferred and adopted for power house (PH-2).

4.4.1.3 Upper Level Tunnel

Various alternatives have been considered for aligning the upper level link tunnel keeping in view that the water has to be led into the proposed KBL canal. Two alternatives, Alt-1 and Alt-2 have been considered with different intake locations as shown in the drawing. The hydraulics of flow, extent of excavation involved, adequate rock cover availability and geological inputs are some of the main factors considered for comparative study. After evaluating the technical and economic aspects, Alt-2 is finally preferred and adopted.

4.4.2 Dam-toe power house - I

The Dam-Toe Power House (PH-1) is proposed near the toe of the dam around 85m downstream from the dam axis. The power house is located on the right of the spillway and on the left of Ken river. The water from PH-1 is led back into the river through the tail race channel. In the event of sudden shutdown of the power house or during planned maintenance, the water can be released downstream through under sluices/spillway. The FRL/MWL of Daudhan reservoir is EL 288.0 m. The discharge through each unit of the power house is 82.0 cumecs and the total discharge is 164.0 cumecs. The gross maximum and minimum head is 54 m and 18 m respectively. The installed capacity proposed for this power house is 60 MW comprising of 2 units of 30 MW each.

4.4.2.1 Intake

Two independent intake structures have been provided for the two units of power house (PH-1) in blocks 32 and 33 of non-overflow portion of Daudhan dam. The intake structure proposed on the upstream face of the dam body is of semi-circular type. The center to center distance between the two intake structures is 20 m. The intake structure mainly comprises of concrete piers, rib beams, metallic trash racks, bell-mouth entry, emergency and service gates and a maintenance platform at the top. The concrete piers and rib beams have been provided with suitable cut water and ease water shapes for stream lining the water flow.

The minimum draw down level (MDDL) for power generation in Daudhan Reservoir for PH-1 would be EL 252.0 m. The operating platform for trash rack cleaning is provided at EL 293.0 m, above MWL/FRL for ease of maintenance in all seasons. The trash racks have been provided up to EL 252.00 m. Above this, concrete panel wall has been provided in the opening between the concrete piers. However the trash rack grooves extend beyond EL 252.0 m up to the top of the dam.

a) Trash rack structure

A metallic trash rack structure is provided to prevent entry of trash, debris and other floating matter into the penstock. Being a large storage reservoir, the sediments are expected to significantly settle down in the reservoir itself. The invert level at intake is well above the expected reservoir sedimentation level after 100 years. The center to center spacing between the trash bars is provided as 100mm conforming to BIS codes for the proposed Kaplan turbine units. The height of each trash rack panel is kept as 3.28 m. The radius of semicircular trash rack is 6.6 m. The total number of trash rack panels for each intake structure works out to 30.

Each intake is designed for a discharge of 82.0 cumecs. The centre line of penstock has been kept at EL 243.0 m ensuring that minimum water submergence required with respect to MDDL to prevent vortex formation is available. Bell mouth transition has been provided at intake entrance for smooth entry of water. Beyond the bell mouth entry, provision has been made for two gates, emergency and service. Provision for air vent has been made downstream of the service gate. After the gates, a transition of 5.0 m length from rectangular to circular shape has been provided. Hydraulic model studies are to be carried out at detailed design stage for firming up the hydraulic details of intake. The plan and sectional details of intake structure as given in drg. No. KBL-7230-P-1003 (Sheet 1/3) is at Plate. 4.151 in Drawing volume – III. Similarly the intake transition details and Trash metal structure details (drg. No. KBL-7230-P-1004 (Sheet 2/3) and KBL-7230-P-1005 (Sheet 3/3)) are appended as Plates No. 4.152 and 4.153 respectively in Drawing volume – III.

b) Intake Gates

Fixed Wheel Vertical Lift Gate of size 3800mmx4500mm shall be provided at the intake of Dam Toe Power House–I. The gate shall have wedge type bottom seal and hollow bulb side and top seals. The seals shall be provided at the d/s side of the service gate. These gates shall be operated by hydraulic hoist mounted at top of the pier at EL 293.00 M. The hydraulic power pack shall be installed in the chamber provided in the dam body. Sill level of gate shall be 240.75m and shall be designed to withstand a static head of 47.25m corresponding to FRL i.e. 288.00m. The gates shall be closed in unbalanced water head conditions and shall be lifted in balanced water head condition to be achieved by crack opening of gate. The gates shall be capable for fully open or fully closed positions. No regulation shall be provided.

Air vent shall be provided at the d/s of service gate to meet out the air requirement. Drawing of intake service gate and stoplogs Dam toe Power House – I sheet 1 of 2 is KBL -7230-P-1506 and appended as Plate. 4.154 in Drawing volume - III. Two units of Stoplogs shall be provided to carry out the maintenance of Service gate. The Stoplogs shall have u/s skin plate and u/s sealings. The Stoplogs shall be operated by Intake gantry crane and lifting beam. Drawing intake service gate and stoplogs Dam Toe Power House-I Sheet 2 of 2 is KBL-7230-P-1507 is appended as Plate. 4.155 in Drawing volume – III.

c) Gantry Crane

20t Gantry Crane shall be provided to operate the Dam Toe Power House-I Intake Stoplogs. The Gantry Crane shall have hoist machinery mounted on trolley. The trolley shall of moving type. The crane structure along with trolley shall be capable of moving in longitudinal direction with the help of LT travel mechanism. The crane shall be designed as per IS: 3177 and IS: 807. The gantry shall be installed at road level i.e. 293.00 M. These gates shall be operated by independent rope drum hoist mounted on common support structure. The support structure shall be installed at EL 293.00M. Each Hoist Machinery shall consist of rope drum, gears and pinions, electric motor and electro –magnetic brakes.

4.4.2.2 Power Channel

Since the intake to the PH-I is directly from the Daudhan dam, no provision for power channel needed to be considered.

4.4.2.3 Tunnels/Pressure Shafts

No tunnel is proposed for the PH – I. However one Lower level tunnel is proposed for the PH – II, the details of which are dealt under PH – II details.

4.4.2.4 Balancing Reservoir

No balancing reservoir is proposed for PH – I

4.4.2.5 Fore Bay

No fore bay is proposed for PH – I.

4.4.2.6 Penstocks and Surge Shaft

i) Embedded Steel Penstock

The center line of penstock is at EL 243.0 m. The penstock is aligned horizontal from the intake. The steel liner of penstock starts from the end of transition. The horizontal penstock as it leaves the downstream sloping face of the dam body is bent down at an angle of 55º to align almost parallel to the downstream sloping face. After the bottom vertical bend of 55º, the penstock runs horizontal. The center line of the bottom horizontal penstock is EL 222.0 m which is also the center line of spiral casing.

Optimization studies for working out the economic diameter of penstock have been conducted. The internal diameter of penstock provided is 4.5 m which gives a velocity of 5.2 m/s. The total length of the embedded steel penstock along the center line of the penstock from the start of steel liner (after transition) upto D-line of the Power House is 84.345m. At D-line of the Power House, the diameter of the penstock is reduced by a reducer piece from 4.5 m to 4.0 m. The center to center distance between the two penstocks is 20 m. The entire length of penstock is embedded in concrete. The penstock between the downstream sloping dam face and D-line of the Power House is embedded in concrete up to EL 233.50 m (service bay level) for ease of movement around Power House. Provision for a manhole has been kept inside the Power House, upstream of the Main Inlet Valve for maintenance of penstock. Penstock Steel Liner Details as illustrated in drawing Nos. KBL-7230-P-1006 and KBL-7230-P-1007 are given as Plates No. 4.156 and 4.157 respectively in Drawing volume – III.

The steel proposed for the steel liner of the penstock is IS 2002 – Grade 2 with yield strength of 2650 kg/cm2 and ultimate tensile strength of 4200 kg/cm2. The steel liner is designed for full internal pressure including water hammer pressure. The steel liner has been checked for the likely external water pressure and grout pressure. Minimum handling thickness for 4.5 m diameter penstock works out as 12.5 mm. The steel liner thickness provided varies from 16 mm to 20 mm along the length of the penstock.

The length of an erection piece would be 5.0 m formed by welding of two 2.5 m long ferrules in the workshop. All non destructive tests viz. radiography, ultrasonic and hydrostatic testing shall be carried out. All the erection pieces shall be painted from inside.

b) Surge shaft

No surge shaft is provided for the intake to PH-I.

4.4.2.7 Surface Power House (PH-I)

Power House – I is proposed as surface powerhouse referred to as Dam-Toe Power House (PH-I) between the spillway and river portion with its long axis aligned almost parallel to the dam axis. The Power House is located at the toe of the dam in front of non-overflow dam blocks 32 and 33. Geological investigations have revealed that the Power House is located in massive sandstone. For more geological details, concerned chapter may be referred.

The FRL/MWL of Daudhan reservoir is EL 288.0 m. The minimum draw down level for power generation for PH-I would be EL 252.0 m. The maximum tail water level is indicated as EL 234.0 m. The expected high flood level at the Power House site is EL 240.89. The power potential study has been carried out by CEA. The design discharge proposed through each unit of the Power House is 82.0 cumecs and the total discharge is 164.00 cumecs. The gross maximum and minimum head is 54 m and 18 m respectively. The installed capacity proposed by CEA for this Power House is 60 MW comprising of 2 units of 30 MW each.

The Power House structure consists of machine hall, service bay, unloading bay, control rooms and transformer deck area. The structure comprises of RCC columns and beams designed to bear the loads coming from various electro-mechanical equipments. The weight of generator rotor is conveyed as 105 t. The capacity of Electric Overhead Traveling Crane for Power House is 125/30 t. The crane beam has been designed accordingly. A steel truss forms the roof of the Power House

The Electro-Mechanical design and drawings for the Power House have been carried out by CEA. The D-line of the Power House is at a distance of 85 m for the dam axis. The dimensions of the Power House excluding control room and including unloading bay is 88.5 m (long) x 22 m (wide) x 53m (high). The center to center spacing between the units is 20 m. Vertical shaft Kaplan turbines have been proposed by CEA. The center line of spiral casing is EL 222.0 m. The Main Inlet Valve proposed is of butterfly type having 4.0 m diameter. Two draft tubes with separate gates have been provided.

Draft tune gates

The draft tube gates shall be provided at the d/s of power unit. Each power unit of Power House I shall have one draft tube gate of size 5000mm X 4400mm. The draft tube gate shall be lowered to isolate the power unit from d/s side and the maintenance of power unit will be carried out without affecting other units. To achieve this, the draft tube gates shall have sealing and track on power unit side. Spring loaded guide shall be provided to avoid pressuring of power unit. However, additional safety feature in electrical circuit shall also be made. The power unit will not start until the draft tube gate is at lifted position.

The skin plate of gate shall be stiffened by vertical stiffeners and horizontal girders. The load from horizontal girders shall be transferred to end vertical stiffeners provided on each side. Bronze pad shall be fitted to end vertical girders. The load will be transferred to concrete through bronze pad and stainless steel track. The gate shall be closed in balanced water head condition. Lifting of gate shall be done in balanced water head condition which shall be achieved by filling valve. The filling valve shall be linked to lifting arrangement of gates. The initial movement of lifting arrangement shall open the filling valve. The gate shall be lifted after achieving the balance head on both sides of gate. The gate shall be linked to gantry crane through lifting beam. Drawing of draft tube gate for Power House – I(KBL-7230-P-1513) is appended as Plate. 4.158 in Drawing volume – III.

The following drawings in respect of Power House – I are appended in Drawing volume - III at Plates. shown below.

	Details	Drg. No.	Plate.
1.	Cross section of Dam toe PH-I	KBL – 7230 – P – 1008	4.159
2.	M.I.V. Floor Layout Plan	KBL – 7230 – P – 1009	4.160
3.	Turbine Floor Lay out plan	KBL – 7230 – P – 1010	4.161
4.	Generator floor Lay out plan	KBL – 7230 – P – 1011	4.162
5.	Machine hall and Service bay plan	KBL –7230– P – 1012	4.163
6.	Control Room Lay out Plan at El 238	KBL–7230 – P – 1013	4.164
7.	Control Room Lay out Plan at El 243	KBL–7230–P – 1014	4.165
8.	Power House L- section through Centre line of the Unit	KBL – 7230 – P – 1015	4.166

Tail Race Channel

A 32 m wide and 405.0 m long tail race channel will carry the turbine discharge back to the river. As per CEA, the maximum tail water level with two units operating is 234.0 m. The crest level at the start of tailrace channel is kept at 232.06 m. The tail race pool floor is sloping upwards at 1: 5. The tail race channel is a RCC structure with a slope of 1 in 800 towards the river. The top of the retaining walls is proposed at EL 243.0 (transformer deck level) which is above the reported HFL of 240.89 m. From safety considerations, one meter high railing is proposed on the top of the retaining wall. The details of tail race channel from PH-I are shown in drg. No. KBL – 7230 – P – 1016 and is appended as Plate. 4.167 in Drawing volume – III.

4.4.3 Lower Level Tunnel (Power House -II)

4.4.3.1 Intake for Power House - II

The lower level tunnel Power House (PH–II) is proposed at the end of the pressure shaft tunnel as a surface Power House. The water after power generation will finally be released into Ken Left Bank canal. The discharge through each unit of the Power House is 20.33 cumecs and the total discharge is 61.0 cumec. The gross maximum and minimum head is 42 and 14 m respectively. The installed capacity proposed for this Power House is 18 MW comprising of 3 units of 6 MW each.

The Intake of PH-II is located on the left bank, around 300 m upstream of Daudhan dam in the reservoir spread area. An independent inclined intake structure, sloping at around 10º has been provided. The intake structure mainly comprises of concrete piers, rib beams, metallic trash racks, bell-mouth entry, emergency and service gates and a maintenance platform at the top. The concrete piers and rib beams have been provided with suitable cut water and ease water shapes for stream lining the water flow.

The minimum draw down level for power generation in Daudhan Reservoir for PH-II would be EL 260.0 m. The operating platform for mechanical trash rack cleaning is provided at EL 293.0 m, above MWL/FRL for ease of maintenance in all seasons. The trash racks have been provided up to EL 254.525 m. The trash rack grooves extend beyond EL 254.525 m up to the top platform.

a) Trash rack structure

A metallic trash rack structure is provided to prevent entry of trash, debris and other floating matter into the tunnel. Being a large storage reservoir, the sediments are expected to significantly settle down in the reservoir itself. The invert level at intake is well above the expected reservoir sedimentation level after 100 years. The center to center spacing between the trash bars is provided as 80 mm conforming to BIS codes for the proposed Kaplan turbine units. The height of each trash rack panel is kept as 2.624 m. The total number of trash rack panels for the intake structure works out to 10.

The intake is designed for a discharge of 65.194 cumec. The centre line of intake tunnel has been kept at EL247.35 m ensuring that minimum water submergence required with respect to MDDL to prevent vortex formation is available. Bell mouth transition has been provided at intake entrance for smooth flow of water. Beyond the bell mouth entry, provision has been made for two gates, emergency and service. Provision for air vent has been made downstream of the service gate. After the gates, a transition of 6.0 m length from rectangular to modified horse-shoe shape has been provided. Hydraulic model studies are to be carried out at detailed design stage for firming up the hydraulic details of intake. The plan and sectional details of intake structure as given in drg. No. KBL-7230-P-1101 (Sheet 1/3) is at Plate. 4.168 in Drawing volume – III. Similarly the intake transition details and Trash metal structure details (drg. No. KBL-7230-P-1102 (Sheet 2/3) and KBL-7230-P-1103 (sheet 3/3) are appended as Plates No. 4.169 and 4.170 respectively in Drawing volume – III.

b) Intake Gates

One number Service Gate and one number Emergency gate of size 4600mm X 5500mm shall be provided in Lower Tunnel Power House-II Intake. The gates shall be Fixed Wheel Vertical Lift type. The gates shall be fabricated in different parts and bolted together. Service gate shall have d/s skin plate and d/s sealing while the Emergency gate shall have u/s skin plate and u/s sealing. The gates structure shall consist of skin plate stiffened by vertical stiffeners and horizontal girders. The horizontal girders shall be supported by End vertical girders provided on each side of gate. The water thrust shall be transmitted to the concrete from horizontal girder by wheels and track embedded in the concrete. The wheels shall be mounted on self lubricating bush bearing. Sill level and top of opening shall be EL 244.60M and EL 250.10M respectively. The gates shall be designed to withstand water load corresponding to FRL i.e. 288.00m.

Two drawings of intake service and emergency gate for PH-II viz., KBL -7230-P-1508 (Sheet 1 of 2) and 2 KBL -7230-P-1509 (Sheet 2 of 2) are appended as Plates No. 4.171 and 4.172 respectively in Drawing volume - III.

4.4.3.2 Power Channel

Since the intake to the PH-II is directly from the surge shaft located at the exit of Lower Level Tunnel, no provision for power channel needed to be considered.

4.4.3.3 Tunnels/Pressure Shafts

Optimization studies for working out the economic diameter of head race tunnel have been conducted. Modified horse-shoe shaped, 5.5 m finished diameter, concrete-lined, head race tunnel is provided which gives a velocity of 2.65 m/s. The length of head race tunnel is 1008 m and is laid at a slope of 1: 175. The excavation of the tunnel is proposed to be carried out by conventional drill and blast method from two ends, i.e. the intake end and pressure shaft end. Hence no provision for adit is made. The L. section of the head race tunnel (Lower level tunnel) for PH – II (Drg. No. KBL -7230-P-1104) is given as Plate. 4.173 in Drawing volume – III.

GSI has carried out the geological investigations and furnished their report on the likely rock strata to be encountered along the alignment of head race tunnel. The tunneling media expected include massive sandstone, gravely-pebbly conglomerate and interbedded sequence of sandstone and slaty shale. For more geological details, concerned chapter may be referred. The rock mass has been classified adopting the Geomechanic classification system (CSIR method) criteria which is as follows.

Very good rock	-RMR	= 81-100
Good rock	-RMR	= 61- 80
Fair rock	-RMR	= 41 - 60
Poor rock	-RMR	= 21 – 40
Very poor rock/Squeezing conditions	-RMR	≤ 20

The rock support system has been designed for the above classes of rock mass which basically consists of shotcrete, rockbolts and steel ribs as shown in the drawing. During excavation, 75 mm dia, 4000 mm long drainage holes are to be provided as required in seepage zones. The head race tunnel is proposed to be lined with M-25 grade concrete lining, 350mm thick. A typical scheme of contact and consolidation grouting has been proposed. The details of typical Excavation and rock support system and the details of concrete lining and grouting details as shown in Drg. No. KBL -7230-P-1105 and KBL -7230-P-1106 are given as Plates No. 4. 174 and 4.175 respectively in Drawing volume – III.

4.4.3.4 Balancing Reservoir

No balancing reservoir is proposed for PH – II

4.4.3.5 Fore Bay

No fore bay is proposed for PH – II.

4.4.3.6 Penstocks and Surge Shaft

i) Penstocks

Optimization studies for working out the economic diameter of pressure shaft have been conducted. The internal diameter of penstock provided is 2.4 m which gives a velocity of 4.5 m/s. Three 2.4 m finished diameter, steel-lined pressure shafts are emanating from the bottom of the surge shaft. The pressure shaft is laid horizontal from the surge shaft up to the Power House with center line EL 240.0 m. The length of the central steel-lined pressure shaft from surge shaft (after transition) to exit portal is 113.55 m. The length of other two pressure shafts from surge shaft (after transition) up to the exit portal including the horizontal bend is 119.661 m. Thereafter it is exposed for a length of 15 m and embedded in concrete for a length of 25 m. In the exposed reach of 15 m, it is proposed to provide a T-junction in the penstock fitted with a Howell Bunger Valve for bypassing the water to the tailrace downstream whenever the powerhouse is closed. The steel proposed for the steel liner of the penstock is IS 2002 – Grade 2 with yield strength of

2650 kg/cm2 and ultimate tensile strength of 4200 kg/cm2. The steel liner is designed for internal pressure including water hammer pressure and checked for external pressure. No rock participation has been considered in resisting internal pressure. Minimum handling thickness for 2.4 m diameter pressure shaft works out as 8 mm. The steel liner thickness proposed for the pressure shaft up to exit portal is 12 mm and thereafter up to D-line of the Power House is 14 mm. The length of an erection piece would be 5.0 m formed by welding of two 2.5 m long ferrules in the workshop. All non destructive tests viz. radiography, ultrasonic and hydrostatic testing shall be carried out. All the erection pieces shall be painted from inside. The Penstock steel liner details as given in Drg. No. KBL -7230-P-1108 is appended as Plate. 4.176 in the Drawing volume – III.

ii) Surge shaft

A 60 m high, restricted orifice type surge shaft of 18 m finished diameter, open to sky has been proposed at the end of HRT to take care of the surges arising from the various operating conditions of the power plant. Three pressure shafts are emanating from the surge shaft bottom leading to the individual units. For isolating the pressure shaft/penstock for maintenance, separate gates have been proposed for each pressure shaft at surge shaft bottom. The minimum sectional area of surge shaft required for hydraulic stability as per Thoma criteria has been worked out as stipulated in the code. Accordingly, 18 m finished diameter of surge shaft is provided. An orifice of 3.5 m diameter has been provided in the orifice slab of the surge shaft satisfying Calame and Gaden criteria as given in the code. The thickness of orifice slab is 2.0 m. Steel liner, 16 mm thick has been proposed around the orifice in the orifice slab.

Transient analysis has been carried out using a computer programme for various operation conditions of the Power House. The maximum upsurge corresponding to simultaneous full load rejection of all the units works out as EL295.0 m. The maximum down surge corresponding to simultaneous full load rejection followed by acceptance of two units works out as EL 255 m.

The rock support system for the surge shaft shall basically comprise of shotcrete, rock bolts and steel ribs depending on the rock mass encountered. During excavation, 75 mm diameter, 4000 mm long drainage holes are to be provided as required in seepage zones. 1000 mm thick reinforced concrete lining is provided for surge shaft. Consolidation grouting of the rock mass will be carried out to improve the rock conditions around the surge shaft. To ensure proper contact between concrete lining and surrounding rock, contact grouting at a pressure of 2 Kg/cm2 is proposed. Being open to sky, the surge shaft is proposed to be covered by a steel frame structure with barbed wire and fencing all around. The details of Restricted orifice type surge shaft provided at the exit of the Lower level tunnel, as given in drg. No. KBL -7230-P-1107 is appended as Plate. 4.177 in Drawing volume – III.

iii) Surge Shaft Gates:

Three Number Penstocks are proposed to carry water from HRT to generating units. Three numbers of fixed wheel type vertical lift gate of size 2400mm x 3000mm shall be provided in surge shaft. These gates shall be provided to carry out the maintenance of pressure shaft and power unit. The pressure shaft shall be isolated without affecting other power units.

The gates shall be operated by individual rope drum hoist mounted on hoist platform. The gate shall be capable of closing in unbalanced water head condition by its own weight. Suitable size ballast may be provided to make the gate self closing. The gate shall be lifted under balanced water head condition which shall be achieved by filling valve provided in the gate. The filling valve shall be opened by initial movement of hoist. The closing speed of gate shall be so selected to close the gate in 15 seconds in case of emergency situation in penstock or Power House. The speed of gate shall be reduced at the end of closing cycle to avoid hitting of gate at sill.

The gates shall have d/s skin plate stiffened by vertical stiffeners and horizontal girders. The girders shall be supported by end vertical girders. Each end vertical girders shall be supported by two wheels. The wheels shall be mounted on anti-friction bearing to minimise requirement of closing force. Taflon cladded solid bulb rubber seals shall be provided for side and top sealing. The bottom seal shall be wedge type. The lip angle of gate shall be optimized to minimise closing force.

Drawing of surge shaft gate drg. No. KBL-7230-P-1512 is appended as **Plate. 4.178** in Drawing volume - III.

4.4.3.7 Surface Power House (PH-II)

PH-II is proposed as a surface powerhouse after the exit of the pressure shaft tunnel. The D-line of the Power House is at a distance of 40 m from the exit portal. The FRL/MWL of Daudhan reservoir is EL 288.0 m. The minimum draw down level for power generation would be EL 260.0 m. The maximum tail water level for the Power House is conveyed as EL 246.0 m. The power potential study has been carried out by CEA. The discharge through each unit of the Power House is 20.33 cumecs and the total discharge is 61.0 cumecs. The gross maximum and minimum head is 42 and 14 m respectively. The installed capacity proposed by CEA for this Power House is 18 MW comprising of 3 units of 6 MW each. The Power House structure consists of machine hall, service bay, control rooms and transformer deck area. The structure comprises of RCC columns and beams designed to bear the loads coming from various electro-mechanical equipments. The weight of generator rotor is conveyed as 20 t. The capacity of Electric Overhead Traveling Crane for Power House is 30/10t. The crane beam has been designed accordingly. A steel truss forms the roof of the Power House.

The Electro-Mechanical design and drawings for the Power House has been carried out by CEA. Vertical shaft Kaplan turbines have been proposed by CEA. The dimensions of the Power House inclusive of control rooms is 82.5 m (long) x 29 m (wide) x 37.8 m (high). The center to center spacing between the units is 16 m. The center line of spiral casing is EL 240.0 m. The Main Inlet Valve proposed is butterfly type, 2.4 m in diameter. Three draft tubes with separate gates have been provided. In the event of sudden shutdown of the units or during planned maintenance, the water can be released downstream through a bypass valve arrangement proposed just upstream of the Power House.

Draft tune gates

The draft tube gates shall be provided at the d/s of power unit. Each power unit of Power House II shall have one draft tube gate of size 5000mm X 4000mm. The draft tube gate shall be lowered to isolate the power unit from d/s side and the maintenance of power unit will be carried out without affecting other units. To achieve this, the draft tube gates shall have sealing and track on power unit side. Spring loaded guide shall be provided to avoid pressuring of power unit. However, additional safety feature in electrical circuit shall also be made. The power unit will not start until the draft tube gate is at lifted position. Drawing of draft tube gate for Power House – I(KBL-7230-P-1514) is appended as Plate. 4.179 in Drawing volume – III.

The following drawings in respect of Power House – II are appended in Drawing volume - III at Plates. shown below.

Details	Drg. No.	Plate.
1. Cross section of Dam toe PH-II	KBL – 7230 – P – 1109	4.180
2. M.I.V. Floor Layout Plan	KBL – 7230 – P – 1110	4.181
3. Turbine Floor Lay out plan	KBL – 7230 – P – 1111	4.182
4. Generator floor Lay out plan	KBL – 7230 – P – 1112	4.183
5. Service bay and auxiliary bay Lay out plan at El 250.20m	KBL – 7230 – P – 1113	4.184
6. Power House L- section through Centre line of the Unit	KBL – 7230 – P – 1114	4.185

Tail Race Channel

A 40 m wide and 255.0 m long tail race channel is proposed to connect to KMPP canal system. As per CEA, the maximum tail water level is EL 246.0 m. The crest level at the start of tailrace channel is kept at EL 244.9 m. The tail race pool floor is sloping upwards at 1:5 slope. The tail race channel is a RCC structure with a slope of 1 in 800 towards the river. The top of the retaining walls of tail race channel is proposed at EL 250.20. From safety considerations, one meter high railing is proposed on the top of the retaining wall. . The details of tail race channel from PH-II are shown in drg. No. KBL – 7230 – P – 1115 and is appended as Plate. 4.186 in Drawing volume – III.

4.5 Upper Betwa Project components

4.5.1 Engineering Assessment of Makodia Dam

The Upper Betwa component of the Ken – Betwa link projects comprises, i) An earth / concrete dam across Betwa at Makodia and corresponding two canals, ii) A barrage (Barari barrage) across Betwa and one right bank canal and iii) Another barrage (Kesari barrage) across Keoton a tributary of Betwa and one right bank canal. The design details of these components are discussed in the following paras. Various provisions of I.S. codes and other standard values which have been dealt with in detail under the Main component i.e., Daudhan dam project above, are not repeated here for simplicity sake.

4.5.1.1 General

The Makodia Dam is one of the three identified Upper Betwa Projects to utilize part of 659 Mm^3 of transferred water through Ken-Betwa link. It will also serve as a storage scheme for Barari barrage to utilize about 316 Mm^3 water including its own demand.

The proposed dam site is located on Betwa river near the village Makoriya in the tehsil of Goharganj, district Rasien of Madhya Pradesh. The latitude and longitude of the dam site are $23^0$14’05” and $77^0$41’00” respectively. The dam site is about 19 km from Raisen and 43 km. from Bhopal. It is proposed to construct a 15.5 m high earthen dam (left and right flank) across river Betwa with a central spillway to pass design discharge of 10384.50 cumecs.

4.5.1.2 Geology, Seismicity and Foundation

(i) Geology

The Makodiya dam site is more or less flat with rolling undulations on both the flanks of the Betwa river displaying elevations roughly between 450 m (right abutment) and 416 m (Betwa river bed). The dam alignment is occupied by thick sequence of hard massive sandstone exposed on both the abutments hills. A considerable part of the dam alignment is covered by soil /alluvium consisting of black clayey soil, sandy silt soil concretionary yellow soil and river borne recent unconsolidated to semi consolidated sediments of sand, kankars and boulders etc., The Betwa River is monsoon fed which remains dry during major part of the year. It flows from south to north with a meandering course. The location and alignment of the proposed dam near Makodia is suitable and geotechnically favourable as the site is devoid of active lineament, faults or shear. Seismically also, the area has not experienced any major geological event in the recent past.

(ii) Seismicity

As already mentioned in the Chapter – II, the studies for site-specific design earthquake parameters for the proposed Makodia Dam area have been carried out by the Department of Earthquake Engineering, Indian Institute of Technology, Roorkee in 2008. As per zones given in the IS 1893-1984, 'Criteria for earth quake resist ant design of structures' reaffirmed in 1998, the proposed Makodia dam site falls in zone II indicating low seismicity of the area. Necessary provision has been made in the design of embankment dam for earth quake.

(iii) Foundation

7 exploratory drill holes were drilled at different chainages along the proposed axis of Makodia dam to precisely assess the subsurface geological condition and water conductivity of bedrocks. Out of seven drill holes two were in the earth dam and five in the concrete dam. The data of drill holes reveals the presence of thick beds of hard, compact and very strong sandstone with occasional shale partings. These continue to occur right from the ground level at both the abutments to the maximum explored depth of 42 m in BH 5. The water loss test indicated a permeability value of bed rock ranging from 0.12 lugeons to 0.54 lugeons signifying impermeable nature of bedrocks.

Based on the interpretation of subsurface data it appears that competent foundation grade sandstone is available in the central portion where location of spillway has been fixed and earth dam on the right as well as left abutment.

4.5.1.3 Alternative Studies Carried out for Selection of Site and Type of Structures

Earlier proposal of Neemkheda-Barari system

After meeting the enroute requirement of link canal, the Ken-Betwa link envisages diversion of 659 MCM surplus waters of ken basin to water deficit areas of Upper Betwa basin by way of substitution. As per feasibility report, the target area of the Upper Betwa project was the command under the four identified projects namely Barari, Kesari, Neemkheda and Richhan in the upper reaches of the Betwa basin proposed by M.P. Govt. An area of 1.27 lakh ha in the Raisen and Vidisha districts of Madhya Pradesh was proposed to be irrigated by utilizing 659 MCM of water annually from aforesaid four projects by way of substitution. However, no survey and investigation was carried out in respect of aforesaid Upper Betwa Project at feasibility stage and information given in Master Plan of Betwa basin of M.P. Govt. was taken for the preparation of Feasibility Report. After preliminary survey & investigation Richhan project has been dropped due to low water availability. Remaining, three project were surveyed and it was found that these three projects are not capable of utilizing 659 MCM of water. However, lot of resistance from local people was found in case of Neemkheda dam resulting in rethinking on locating the site in further upstream. Identification of Upper Betwa Project was, therefore, discussed in three co-ordination committee meetings held on 07.03.2007, 10.08.2007 and 10.03.2008 respectively, between Officers of Water Resources Deptt., Govt. of M.P. and NWDA. Finally, Govt. of M.P. suggested Makodiya dam site in place of Neemkheda dam in last meeting held on 10.03.2008. Prior to this decision, NWDA has carried out study of Neemkheda dam – Barari barrage barrage system.

(i) Neemkheda dam

Neemkheda dam is located on river Betwa near village Neemkheda on National Highway. Based on observed gauge & discharge data of Basoda site on river Betwa, the 75% dependable annual yield available at Neemkheda dam site was worked out as 526 MCM. The balance water available after meeting u/s requirement was 339 MCM for the utilization in the d/s. The FRL and MDDL of Neemkheda dam was proposed at RL 434 m & 424 m respectively. The gross and live storage capacity at Neemkheda dam site were 293 MCM & 240 MCM respectively. In this proposal Neemkheda dam was to work as a storage reservoir and releases from this was proposed to be utilized through Barari barrage about 80 km in the d/s of Neemkheda dam for irrigation in absence of command in nearby area. However, one left bank canal of about 22 km long was planned from Neemkheda dam itself to irrigate about 6036 ha on its left bank through pumping. Command area from Barari barrage lying in Raisen district also involved pumping in this proposal.

(ii) Barari barrage (Lift)

It is located on river Barari near Barrighat village of Gyaraspur tehsil of dist. Vidisha. The pond level at RL 408 m was proposed. Water received from Neemkheda dam was proposed to be lifted about 20 m upto RL 425 m by 4 km long pipe line in the small existing tank located near Gulabganj village. Thereafter one main canal 41 km long was proposed to run towards Ganj Basoda in the northern direction. The tank was to be reconstructed according to requirement of project. Total annual irrigation proposed through this lift system was 49.300 ha utilizing 263 MCM water. The energy required for lifting the water was 282 lakh kwh. Thus these two projects viz. Neemkheda & Barari looked different, but were part of one combined system.

(iii) Kesari barrage

It is located on river Kevtan a tributary of Betwa river, near village Didholi tehsil Basoda of district Vidisha M.P. Earlier is was proposed to utilize about 12.20 MCM by lift on both bank with a gross storage capacity of 17.70 MCM. The pond level was kept at RL 406 m. After field survey the pond level is being kept same as earlier proposal i.e. 406 m But the quantity of water as available now is slightly higher and the gross and live capacities now work out to 20.80 MCM 7 17.50 MCM respectively. The total water is proposed to be utilized through lift on right bank only having lesser static head. The annual irrigation will be now be about 3000 ha in the tehsil Basoda of district Vidisha. This is an independent project and so no change is proposed in this.

4.5.1.4 Choice of Final Layout of All Major Components of the Project and Reason

(i) Modified proposal of Makodiya – Barari system

Government of Madhya Pradesh suggested Makodiya dam site as an alternative site to Neemkheda while the submergence survey of Neemkheda dam was also going on. Therefore, survey works of submergence was extended upto RL 440 m. This new dam site is 7 km upstream of Neemkheda site. Dam axis survey of Makodiya was also completed. After completion of submergence survey work of Neemkheda dam, Govt. of Madhya Pradesh suggested to consider the Makodiya dam site instead of Neemkheda with the FRL & MDDL of Makodiya dam at RL 437.5 m and 432 respectively. In this changed scenario, possibility of taking canal from its both the flanks was explored. It was found that it is possible to run the main canal thorough gravity from right flank of Makodiya dam. As such the main canal is proposed to off-take from right bank at FSL 432 m and run about 90 km to reach near Gulabganj town. The another canal is also proposed to off-take from left bank at FSL 432 m and run about 5 km to reach near Kharvai village the gross and live storage capacities of Makodiya dam site with respect to FRL of 437 m are 276 MCM & 240 MCM respectively. The 75% dependable annual yield available at Makodiya dam site is 503 MCM The balance water available at Makodiya dam site is 316 MCM. To utilize available quantum of water, the level of 437 m was found quite adequate. As such FRL of Makodiya has been proposed as 437 m instead of 437.5 suggested by M.P. State Water Resources Deptt.

Further, out of 316 MCM about 13 MCM water will be utilized for the domestic purpose in the villages of command area, Vidisha and Raisan towns. The balance water of about 303 MCM will be utilized through this canal as well as left bank canal for irrigation purpose. Total annual irrigation proposed is about 56,850 ha. Out of this 9,042 ha area will be irrigated in Raisen and 46,880 ha in Vidisha district through right bank canal. Annual irrigation through left bank canal is 928 ha in Raisen district. The CCA is considered as 90% of GCA. The canal will cross major rivers such as Kuhu, Richhan, Newan, Badal, Seu and Garkhatu nadi and about 48 Major/minor roads crossings.

In addition to this, about 15 MCM water available at RL 408 m of Barari barrage is only to be lifted upto RL 425 m by 4 km long pipe line in the small existing tank located near Gulabganj village to irrigate about 2500 ha in the nearby area of project at right bank.

4.5.1.5 Design Flood and Sedimentation Studies

a) Design Flood

Two approaches are used in this study to estimate the design flood. These are (i) unit hydrograph approach and (ii) statistical approaches i.e. annual maximum series modeling. The unit hydrograph estimated from observed data using Nash method gave inferior results compared to the synthetic unit hydrograph while computing the direct runoff hydrograph at Basoda G&D site. Hence, for three sub-catchments i.e. Makodia, Kesari and Barari, synthetic unit hydrographs were derived based on the geomorphological data and flood estimation report for Betwa Subzone –1(C) (CWC. 1989). The design storm for the project area was determined using the PMP/SPS data supplied by IMD and a critical sequencing procedure recommended in the manual of CWC (2001). On basis of the CWC (1993) guidelines, a 24 hours design storm duration was adopted for the project area.

The peak of design flood hydrograph for Makodia comes out to be 10385 m^3/s. For Barari and Kesari Barrage, design flood peak corresponding to PMP are 17460 m^3/s and 3871 m^3/s, respectively. The design flood peak corresponding to SPS are 12283 m^3/s and 2772 m^3/s for Barari and Kesari, respectively

b) Reservoir sedimentation

The total sediment deposition in the Makodia reservoir during 50 and 100 years are estimated to be 37.5 and 75.0 MCM respectively. The new zero elevations after 50 years and 100 years have been found to be as 428.24 m and 430.0 m respectively. The total sediment during 50 and 100 years will get distributed up to and above various elevations as given in the table 4.11 below.

Table 4.11 The total sediment estimated at Makodia

Reservoir level	Sediment deposition in MCM	
	After 50 years	After 100 years
Up to 430 m	11.35	14.24
Above 430 m	26.16	60.79

The Original and revised (after 50 and 100 years) Elevation-Area-Capacity values for Makodia dam are furnished as Annexure 4.11.

4.5.1.6 Free Board

The basic requirement of free board is that the dam should not be overtopped under design wind conditions. The procedure adopted is based on recommendations of IS 10635-1993 (Reaffirmed 1998) "Guidelines for free board requirements in Embankment dams". For normal freeboard the wind velocity at the dam site over land has been taken as 39m/s. The computed free board comes out to be 4.57 m for and_accordingly the top of dam has been provided as 442.0 m against MWL/FRL of 437.50m. The total height of dam will be to 15.5 m. above ground level of 426.50m at RD 486.0 m. The design computations of freeboard for the Makodia earth dam have been presented in Drawing No. KBL-7230-P- 251 which is appended as Plate. 4.187 in Drawing volume – IV.

4.5.1.7 River Diversion Arrangement

As already explained at 4.1.1.7 above, as per the guidelines laid down in the ' Report of Ministry of Irrigation, 1980 for arriving at the diversion design flood according to criteria of risk and damage for different types of dams and barrages' the diversion floods for construction shall be follows.

(i) Diversion capacity for concrete dams and barrages

The capacity of the diversion flood for concrete dams and barrages may be less because flood higher than the designed one could be passed safely over the partly constructed dam. The following criteria would help in deciding the capacity.

a) Maximum non-monsoon flow observed at the dam site.

OR

b) 25 years return period flow, calculated on the basis of non-monsoon yearly peaks.

The higher of the two should be taken as the design flood for diversion.

(ii) For large dams

For large dams, it is desirable that 100-year return flood should be adopted for diversion works.

For the area, the peak of daily flow for non-monsoon months (Novermber to May) was available for 30 years at Basodo.This data was used to estimate the value s for Kesari, Makodia and Barari in proportionate to the ratio of catchment area. Keeping in view the guidelines, the recommended diversion floods for the three projects are given in table 4.12.

Table 4.12 Diversion flood for Makodia, Barari and Kesari projects.

Project site	Flow (m^3/s)			
	Maximum observed non-monsoon	25-year return period	100-year return period	Recommended diversion flood
Makodia	286	85	116	**286**
Barari	653	194	265	**653**
Kesari	109	32	44	**109**

As shown above, the diversion flood for the Makodia, Barari and Kesari projects is estimated at 286 cumec, 653 cumec and 109 cumec, respectively.

Makodia dam is a composite dam having earthen and Concrete dam. As the concrete dam is proposed in the deepest level of the river, it has been planned to divert the flood of 286 cumecs during the construction period with 3 number of construction sluices at El 416.00m of size 3.00m x 4.50m in block No. 8 and 9. The arrangement of gate operation for the construction sluices have been kept on the u/s face of the dam and it will be operated from the top of dam. In the proposed plan, a Coffer dam for maximum water at El. 427.5 has been considered. The diversion flood during construction of spillway blocks can be managed by channelizing the required flood as per reverent I.S. codes.

The diversion of flood at Makodia dam will pass through construction sluice provided in the concrete gravity dam. Initially water will be allowed to pass through few blocks in the deepest portion and work can be taken up for construction of other blocks including earth dam in both the flanks. The upstream & downstream cofferdams of about 10 m height with a slope of 2(H):1(V) (upstream & downstream) will be constructed when these blocks are to be taken up for construction. A lump sum provision is being made for the cofferdam**.** Details of construction sluice through concrete dam are discussed under concrete gravity dam at Makodia.

4.5.1.8 Construction Materials

Makodia Earth dam

For construction material of Earth dam 20 representative soil samples were collected from borrow areas for conducting various standard laboratory tests in order to ascertain their suitability as construction materials. All the 20 soil samples were found of CI classification (Clay of intermediate plasticity). The tri-axial shear tests results indicate low shear strength characteristics of the borrow area materials. Based on the one dimensional consolidation test results, the borrow area samples in general, possess intermediate compressibility characteristics. The result of laboratory permeability test indicates that these materials are impermeable.The results of the Triaxial shear tests adopted in the design are furnished in the following table 4.13.

Table 4.13 Results of Tri-axial Shear Tests adopted in the design

SL.No	Sample No.	Maximum Dry Density (gm/cc)	OMC (%)	Specific Gravity	Effective Cohesion Kg/cm^2	Effective Angle of Shearing Resistance (In degree)	Classification as per BIS code
1.	2008/34	1.68	19.8	-	0.16	19.6	CI
2.	2008/387	1.69	19.5	-	0.21	19.0	CI
3.	2008/42	1.71	17.2	2.65	0.16	20.7	CI
4.	2008/47	1.68	18.0	2.77	0.18	16.7	CI
5.	2008/50	1.69	19.2	-	0.18	16.7	CI
6.	2008/54	1.72	18.7	2.73	-	-	CI

Laboratory permeability test was conducted on 4 samples. Samples were tested at 98% of MDD. No water flow was observed during the test in all the samples. This indicates that soil samples are impervious in nature. Free Swell index tests were carried out on 4 samples. Degree of expansion for 3 samples is moderate and high in case of fourth sample. The plasticity index values show that samples in general have medium plasticity characteristics

4.5.1.9 Details of the Model Studies for Important Structures

The model studies required to be done for important structures proposed to be taken up at preconstruction stage of the project.

4.5.2 Makodia Dam

4.5.2.1 Makodia Earth Dam

All the components of the Earth dam have been designed as per guidelines/criteria mentioned in relevant BIS codes. Makodia dam of Ken-Betwa Link Project is a 15.50m high earth dam (with central spillway) with top at EL. 442.00m. The left earth dam is from RD 41 to 171 m and right earth dam is from RD 486.0 m to 621 m. The left & right flanks of the non-overflow blocks are wrapped around by earth dam. Wraparound sections consist of key block (conventional type) which will be designed before taking up construction. The Plan & layout of the dam is available in Drawing No. KBL-7230-P-258 and is appended as Plate. 4.188 in Drawing volume - IV.

The details of wraparound has given in drawing No. KBL -7230 P-261 and is appended as Plate. 4.189 in Drawing volume - IV.

Earth Dam Section

The geometry of the earth dam has been provided taking into account the structural safety, Seismic aspects, seepage control etc. Detailed investigation for construction material is to be undertaken before actual construction is taken up.

The dam section consists of homogeneous section. The upstream face has slope varying from 2.75(H):1(V) to 3.00(H):1(V) while downstream face has also slope varying from 2.75(H):1(V) to 3.00(H):1(V) but the downstream slope has a 12m wide berm at El 431.0 m compared to 9.0m at EL 430.0 m on the upstream slope. This is mainly due to high tail water level. 2.0 m thick vertical filter and 1.0 m thick horizontal filter of sand and well graded gravel has been provided to take care of seepage.

The material of the embankment dam is mainly CI. Therefore 0.6m thick sand drains at 5 m interval have been provided in the upstream draw down portion to relieve pore pressure in the event of draw down.

The typical section of dam has been presented in Drawing No KBL-7230-P-256 is appended as Plate. 4.190 in Drawing volume - IV.

Evaluation of Design Parameters.

For estimation of shear strength parameters of borrow area of embankment materials, 75% dependable values of borrow area results were adopted. These are presented in Drawing No. KBL-7230-P-252 is appended as Plate. 4.191 in Drawing volume – IV.

No foundation tests were carried out for the foundation soil. Since the borrow area are close to dam on the upstream side same values have been adopted, for DPR stage, for calculation of shear parameters for foundation material. For estimation of shear strength properties of foundation 100% dependable values were adopted. At the preconstruction stage/ specification stage shear parameters for the foundation material shall be based on the actual samples taken from the foundation. The evaluations of shear parameters, of foundation overburden, are available in Drawing No. KBL-7230-P-253 is appended as Plate. 4.192 in Drawing volume - IV.

The properties are summarized below in table 4.14.

Table 4.14 The shear properties of over burden soil at Makodia dam site

Sl. No.	Material	Saturated Unit Weight	Moist Unit Weight	Cohesion C’	Angle of shearing Φ′
1.	Embankment Material	2.10t/m^3	2.01t/m^3	1.725 t/m^2	18^0
2.	Foundation	2.10t/m^3	-	1.6t/m^2	17^0

Stability Analysis

The stability analysis has been carried out by software made in-house based on Swedish slip circle or Fellinious method. The minimum factors of safety obtained for various loading conditions is indicated in the following table 4.13 and also available in the_drawings No. KBL-7230-P-254 & 255 are appended as Plate. 4.193 and 4.194 respectively in Drawing volume – IV.

The factors of safety considered are furnished in the table 4.15

Table 4.15 Minimum Factor of Safety

SL.	LOADING CONDITION	UPSTREAM	DOWNSTREAM
1.	Steady Seepage without EQ	1.49	1.48
2.	Steady Seepage with EQ	1.13	1.14
3.	Sudden Drawdown	1.31	1.44

The values in the steady seepage condition which are slightly less than the required FOS of 1.5 are below the foundation. Since foundation shear parameters are based on the soil tests of borrow areas, actual values of shear parameters shall be calculated on the bases of foundation soil test results at pre construction stage and section modified if required.

Zoning of Dam Section

The dam section is comprises of 4 kinds of material viz. embankment material, sand drains, filter (Sand layer & crushed stone layer) and hand placed riprap. The embankment material should be compacted to an average density of 100% of standard proctor density subjected to minimum density equal to 98% of proctor density.

Zoning of the dam section would have to be finalized after availability of the strength parameters and gradation of the materials from required excavation at the pre-construction stage.

The details of cross section of various RD's given in drawing No. KBL-7230-P-259 is appended as Plate. 4.195 in Drawing volume - IV.

Protection of Slopes of Dam Section.

The upstream face is protected by a hand placed riprap of 0.6m.thickness laid over 0.15m. thick crushed stone/aggregate and 0.15m. thick sand filter. This riprap shall consists of angular rock fragments with 80% fragments heavier than 50 Kg. In the remaining 20% not more than 50% fragments shall be smaller than 75mm in the minimum dimension. However, this will be finalized after detailed survey prior to construction stage

To take care of high tail water level stone pitching has been proposed on the downstream side up to EL 432.0m. Above EL 432.0 m turfing has been provided on the down stream face. Further for the collection of the rainfall water and seepage water surface drainage and a toe drain has been provided. The details are available in drawings No. KBL-7230-P-260 is appended as Plate. 4.196 in Drawing volume - IV.

Filter

The filter shall be compacted to an average relative density of 75% with a minimum acceptable relative density of 70%. Filter material shall satisfy the following criteria:

$$\frac{\text{D15 of Filter material}}{\text{D15 of base material}} < 5$$

$$\frac{\text{D15 of filter material}}{\text{D85 of base material}} < 5$$

Where D15 and D85 are 15% and 85% finer material by weight. Further the grain size curve of the filter material will be parallel to that of the base material. Maximum particle size of the filter material shall not be more than 75mm and it should not contain more that 5% of the material finer than 0.074mm (200 mm Sieve).

Asiesmic Design Considerations

For seismic design Earth quake coefficient, α_h as 0.6 g and α_v as 0.4 g has been adopted for Pseudo-static analysis of the section. However these coefficients shall be recalculated based on site specific response spectra.

Adequate factor of safety has been provided against slip failure of slopes during earthquake combined with steady seepage. A wide abutment core-contact will ensure the safety against opening of joint during the earthquake. Earthquake coefficients adopted will be got approved by the National Committee of Seismic Design Parameters at pre construction stage.

Foundation treatment

The foundation treatment has been decided based on the geological interpretation carried out from the data of drill holes and other investigations. To control the under seepage, grout curtain in one row at 3m centre to centre up to a depth half the hydraulic head measured above the base of the cut-off trench up to FRL have been provided. The minimum depth of curtain grouting shall be 6 m. In addition, peripheral grouting shall be carried out at FRL. At the time of pre construction/construction the depth of curtain shall be decided in consultation with Geologist. The post-grouting permeability should be less than 5 lugeons.

The foundation treatment details are presented in Drawing No KBL-7230-P-257 is appended as Plate. 4.197 in Drawing volume - IV.

4.5.2.2 Makodia Concrete dam / Non Over Flow Section / stability analysis / Spillway section

A 39.70m high and 581.00 m. long composite dam is proposed across river Betwa near Makodia village in Raisen distt, M.P.

Makodia dam consists 45.75m long NOF blocks on the left side, 45.75m. long on the right side with a 223.50m long spillway on the centre (13 bays of 13.5m width with 12 piers of 4.0m width) to pass a peak flood of 10385 cumec (PMF).

Drawing showing general layout plan of Makodia dam (KBL-7230-P-151) and Plan & U/S elevation (KBL-7230-P-152) are appended as Plate. 4.198 and 4.199 respectively in Drawing volume – IV.

(i) Non overflow section

The stability analysis for NON OVERFLOW section has been carried out at the deepest foundation level i.e., 414.00 m as per IS-6512-1984 for all the seven load combination listed at 4.2.1.2 above.

The stresses obtained are compressive in all conditions and the factor of safety against sliding is more than 1 as per IS.6512-1984.

The following data have been adopted for the design of Non Overflow (NOF) sections.

a) Maximum water Level (MWL)	=	437.50 m
b) Full Reservoir Level (FRL)	=	437.50 m
c) Maximum Tail Water Level (TWL)	=	431.00 m
d) Minimum Tail water Level	=	416.00 m
e) Silt Level	=	427.00 m
f) Top width of dam	=	8.0 m
g) U/S slope	=	0.10
h) D/S slope	=	0.80
i) Horizontal seismic coefficient	=	0.06 g
j) Vertical seismic coefficient	=	0.04 g
k) Cohesion	=	12.5 MPa
l) Angle of internal friction	=	55 degrees
m) Width of drainage gallery	=	2.0 m
n) Ht of drainage gallery	=	2.5 m

The proposed Makodia dam falls in zone II, given in I.S.1893-1984, "Criteria for earthquake resistant design of structure" The value of horizontal and vertical seismic coeff. adopted are 0.06g & 0.04g respectively, which is subject to the approval by National Committee on seismic design parameters.

The cohesion and angle of internal friction of the rock has been taken as 12.5 MPa and 55^0 respectively as furnished by the field investigation engineers. In case of any appreciable change in these values, the stability analysis has to be revised.

Drawing showing maximum non overflow section (KBL-7230-P-154) is appended as Plate. 4.200 in Drawing volume - IV.

(ii) Spillway Makodia dam

The overflow section has been designed to pass peak flood of 10385 cumecs (PMF) keeping crest level at 427.00 m and maximum water level 437.50m. Vertical upstream face has been provided from El. 420.00 m to 425.802 m and below El 420.00 m a slope of 1 in 10 in upstream face has been provided up to foundation level. The crest shape, discharge coefficients and d/s water surface and water nappe profiles have been worked out as per relevant I.S.codes. The stability analysis for OVERFLOW section has also been done at the deepest foundation level i.e. 405.00m as per IS-6512-1984 for all the seven load combination.

Drawing showing maximum overflow section of Makodia dam (KBL-7230-SP-153) and zoning of materials NOF and OF section (KBL-7230-P-155) are appended as Plate. 4.201 and 4.202 respectively in Drawing volume – IV.

The data used for design is as given below

a) Radial gate trunion elevation level	=	432.00 m
b) Bridge weight	=	15 t/m (Aprox)
c) Top width	=	8.0 m
d) Width of block	=	17.50 m
e) Elevation of T.P.	=	419.025 m
f) Top Bridge level	=	443.70 m
g) Cm	=	0.735
h) Top of crest	=	427.00 m

The stresses obtained are within permissible limit and the factor of safety against sliding are more than 1 as per IS.6512-1984.

(iii) Makodia Dam Spillway Radial Gate

Makodia Dam Spillway shall have 13 Numbers Radial Gates. The size of gates shall be 13500mm X 11336mm. Sill level of gate shall be kept at EL 426.164M. The trunnion of gate shall be kept above maximum tail water level i.e. EL 432.00M. Top of gate shall be kept at EL 437.70M. The radius inside skin plate shall be 11.5M. Drawing showing spillway radial gate of Makodia dam (KBL-7230-P-1525) is appended as Plate. 4.203 in Drawing volume – IV.

The gates shall be operated by down stream twin cylinder Hydraulic Hoists. The Hoist cylinders shall be pivoted on the Hoist Support structure mounted on the pier. The power pack shall be installed on the top of the pier. Each gate shall have individual Power Pack. However, provision shall be made to operate the adjacent hoists in case of emergency. The gate shall consist of curved skin plate stiffened by vertical stiffeners. The vertical stiffeners shall be supported by three Horizontal girders. The load from horizontal girders shall be transmitted to trunnion by three radial arms on each side of gate. The load from trunnion shall be further transferred to concrete by independent anchorage system. Tie Beam shall be provided to cater the lateral load of radial arms. Suitable bracings shall be provided for Horizontal girder and arms. Bottom seal of gate shall be provided as wedge type. The side seals shall be of Z type and will move on curved seal seat. The gate shall be designed as per IS: 4623.

Spillway Stoplogs

Two sets of stoplogs shall be provided to carry out the maintenance of spillway Radial Gate. Each set of stoplogs shall consist of eight units of 13500mm X 1440mm. Bottom Unit shall be non interchangeable type. All other units shall be interchangeable. The stoplogs shall be operated under balanced water head condition except top most unit which shall be lifted under unbalanced water head condition for one gate unit height water head. Downstream skin plate and downstream sealings shall be provided. Wedge type bottom seal and solid bulb type side seals shall be provided to make the gate water tight. The Stoplogs shall be operated by gantry crane moving on the bridge. The Stoplogs shall be connected to gantry crane through Lifting Beam and Ramshorn Hook.
Drawing showing water stoped details OF section (KBL -7230-P 156) & water stoped details NOF section (KBL – 7230-P-157) are appended as Plate. 4.204 and 4.205 respectively in Drawing volume – IV.

Gantry Crane

The Spillway Stoplogs shall be operated by moving Gantry Crane. The tentative capacity of gantry crane shall be 30T. The Gantry Crane shall have hoist machinery mounted on trolley. The trolley shall of moving type. The crane structure along with trolley shall be capable of moving in longitudinal direction with the help of LT travel mechanism. Suitable counter weight shall be provided to make the crane stable for different stability conditions. The crane shall be designed as per IS: 3177 and IS: 807.

(iv) Energy dissipation arrangement

Energy dissipation arrangement envisaged is that of Stilling Basin Type-I. IS-4997-1968 (Reaffirmed 1995) stipulates that Stilling Basin Type is used when the tail water rating curve approximately follows the hydraulic jump curve or is only slightly above or below it then hydraulic jump type stilling basin with horizontal apron provides the best solution for energy dissipation. Considering the froude Number of the incoming flow is less than 4.5, the Stilling Basin of Type-I has been adopted. However, based on site condition and model studies sufficient precaution would be taken in design at preconstruction stage.

(v) Constraints felt during concrete dam design

Following constraints have been felt during concrete dam design

1. Studies are based on limited geotechnical data/information and hence detailed geological appraisal needs to be carried out at pre-construction stage.
2 Model studies should be carried out to ascertain the efficiency of energy dissipation arrangement and layout and profile of approach and stilling basin at the pre-construction stage.

4.5.2.3. Openings through Makodia Dam

Two irrigation outlets have been provided one each in the left and right flanks. The right flank double barrel outlet, at RD 570 m., each of size 2.0mx 2.0 m will carry a discharge of 21.241 cumec while left flank pipe out let, at RD 130 m., of 0.6 m dia. will carry discharge of 0.356 cumec.

Details of pipe outlet at various RD's of Makodia dam given in drawing No. KBL-7230-P-262 & 263 are appended as Plate. 4.206 and 4.207 respectively in Drawing volume – IV.

As the proposed canals are aligned at higher elevations, the provision of river sluice have not been kept in the concrete blocks. However, the foundation gallery of size 2.0m x 2.5m has been proposed in the near u/s face of dam through out the all blocks. One Elevator shaft and one Staircase have been proposed to provided in block No. 2 and 17 respectively

Construction Sluice Gates

Two Nos. of construction sluices have been proposed to divert the flow of river. Size of sluice proposed are 3.0m X 4.5 m. Each sluice will be provided with Fixed Wheel vertical lift type gates. The gates shall have spring loaded guide on both sides at the face of dam. The skin plate will be supported by vertical stiffeners & horizontal girders. The water thrust from the horizontal girders will be carried by four wheels on each with the help of end vertical girders. The wheels will be mounted on bush bearings. The load will be transferred to concrete structure by track cum guide. The sealing and skin plate shall be provided on d/s side. The gate shall be closed during lean flow to carry out the plugging operation of sluice and will never be lifted. The gates shall be operated by chain pulley block or any other suitable arrangement. Drawing showing construction sluice details (KBL-7230-P-158) is appended as Plate. 4.208 in Drawing volume - IV. Drawing showing construction sluice gate of Makodia dam general installation (KBL-7230-P-1529) is appended as Plate. 4.209 in drawing volume – IV.

4.6 Barrages / Weirs

Two barrages viz., Barari barrage on Betwa river and Kesari barrage on Keoton, a tributary of Betwa have been proposed under the Upper Betwa component of the Ken – Betwa link project.

4.6.1 Barari Barrage

Barari Barrage has been proposed over River Betwa Downstream of Makodia Dam. Following data has been provided for the design of Barrage.

1.Maximum Observed Non Monsoon 25 year flood	=653 cumec
2.Pond Level	=407.72 m
3.Average River Bed Level	=400.00 m
4.High Flood Level	=412.90 m
5.Standard Project Flood	= 12283 cumec
6.Silt Factor	= 1

Design Flood

As per Cl 5 of IS 6999-Part 1 : 'Hydraulic Design of Barrages and Weirs-Guidelines Part I alluvial Reaches' design flood has been considered. For purposes of design of items other than free board, a design flood of 50 year frequency may normally suffice. In such cases where risks and hazards are involved, a review of this criteria based on site conditions may be necessary. For designing the free board, a minimum of 500 year frequency flood or the standard project flood [see IS 5477 (Part 4): 1971] is desirable. Therefore design flood is proposed as 12283 cumec (SPF). The details of design of the Barari barrage are given at Annexure 4.12.

4.6.1.1 Sediment Data

Since the Barari barrage is located on d/s of the Makodia dam, no significant sediment flow is expected. A silt factor of 1 is adopted for the design of the barrage.

4.6.1.2.1 Assumed Retrogression

The d/s unretrogressed HFL is 412.90 m and an afflux of 0.07 m is assumed initially.

4.6.1.3 Looseness Factor

For the flood discharge estimated at the barrage site, the Laceys water way is worked out as 535.3 m and the total water way provided is 440 m. The looseness factor is worked out as 0.822

4.6.1.4 Scour Depth

With looseness factor less than one the Scour depth is worked out as 12.43 m where as with looseness factor more than one, the scour depth is 10.93 m. As such the scour depth adopted is 12.43 m

4.6.1.5 Intensity of Discharge Under Design Flood Condition

The intensity of discharge under design flood condition is 30.51 cumec/m, against the total length of spillway bays of 250 m and the discharge through spillway equal to 7628.2871 cumec. The discharge through under sluice is 5182.54 cumec.

4.6.1.6 Co-efficient of Discharge

The coefficient of discharge for the under sluices (drowned weir formula) is 1.08 and drowning ratio is 0.99. The coefficient of discharge for the spill way is 1.09

4.6.1.7 Exit Gradient Value

The exit gradient shall be determined from accepted formulae and curves. The factors of safety for exit gradient for different types of soils shall be as follows:

a) Shingle : 4 to 5,
b) Coarse sand : 5 to 6, and
C) Fine sand : 6 to 7.

The exit gradient value is worked out as 0.17465 or 1 in 5.7

4.6.1.8 Stresses Allowed

The residual stresses at various points from u/s sheet pile to d/s sheet pile are 100%(at E1 – top of u/s sheet pile), 85.07% (at D1- bottom of u/s sheet pile) , 80.81 (at C1 – top of u/s sheet pile – inside face), 29.82% (at E2-top of d/s sheet pile – inside face), 20.69% (at D2-bottom of d/s sheet pile) and 0% (at C2 – top of d/s sheet pile).

4.6.1.9 Type (Concrete/Masonry)/Profile Cut Off, Upstream and Down Stream Aprons, Uplift Pressure Relief Arrangements etc.,

Sheet pile cutoff is proposed for safe exit gradient. Various components of the Barrage e.g., raft, piers, flexible apron etc have been designed by the relevant provisions of IS Codes. Various drawings of Barari barrage (drawing no. KBL – 7320 – D 2702 and 2703) are appended as Plate. 4.210 and 4.211 respectively in drawing volume – IV.

4.6.1.10 Gates, Types of Gates and Hoist Bridge and Stop Logs

The Spillway of Berari Barrage shall be provided with 25 numbers fixed wheel vertical lift gates of size 10,000mm X 3720mm. Sill level of the gate shall be EL 404.00m. The gate shall be provided with upstream skin plate and upstream sealings to avoid silt on horizontal girders. Wedge type seal shall be provided for bottom sealing and music note solid bulb seals shall be provided for side sealings. The seal shall remain in contact with stainless steel seal seats to make the gate water tight. The gate structure shall consist of skin plate stiffened by vertical stiffeners and horizontal girders. The horizontal girders shall be supported by end vertical girders on each side. The water thrust will be transferred to concrete structures from the end vertical girder through wheels and wheel track. The wheels shall be mounted on self lubricating bush bearings. The wheel shall be made up of corrosion resistant steel. The BHN of wheel track shall be 50 BHN higher than the wheel material. 20 mm guide and two number guide shoes shall be provided on each side to guide the gate in grooves. The gates shall be operated by rope drum hoist of 25 t capacity. The Rope Drum Hoist shall consist of hoist machinery mounted on hoist support structure. Each hoist machinery will be equipped with two rope-drums, gears, pinions, couplings, shafts, worm reducer, motor and brakes. The hoist bridge shall be supported on trestles. Drawing showing Barari barrage spillway vertical lift fixed wheel gate general installation (KBL-7230-P-1521) is appended as Plate. 4.212 in drawing volume – IV.

Under Sluice Gates

The barrage shall be provided with 10 numbers under sluice gates of size 10000mm x 7720mm. The sill level of gate shall be kept at EL 400.00M. The gate shall be provided with upstream skin plate and upstream sealings to avoid silt on horizontal girders. Wedge type seal shall be provided for bottom sealing and music note solid bulb seals shall be provided for side sealings. The seal shall remain in contact with stainless steel seal seats to make the gate water tight. The skin plate of gate shall be stiffened by vertical stiffeners and horizontal girders. Numbers of girders and their location shall be optimized at design stage. The gate shall be operated by Rope Drum Hoist. The hoist shall be capable of lifting the gate above groove for servicing of gate. The hoist machinery shall be supported on hoist supporting structure.

The gate shall be regulating type and can be kept at desired level between sill and FRL. Drawing showing under sluice gate (KBL-7230-P-1522) is appended as Plate. 4.213 in drawing volume – IV.

Spillway / Under Sluice Stoplogs

Stoplogs are proposed to carry out the maintenance of spillway gates and under sluice gates. Four sets of stoplogs of size 10000mm X 1320mm shall be provided. The stoplogs shall be operated in balanced water head conditions. However, the top most unit can be lifted under unbalanced water head for one unit height water head. The stoplogs shall be operated with gantry crane. The stoplog units shall be stored in stoplog groove above MWL. The stoplog units shall have bronze pad sliding on stainless steel track. 20mm guide shall also be provided to guide the stoplog units. The skin plate and sealing gate shall be kept d/s side. Drawing showing spillway stoplog gate (KBL-7230-P-1523) & under sluice stoplog gate (KBL-7230-P-1524) of Barari barrage are appended as Plate. 4.214 and 4.215 respectivley in drawing volume – IV.

Gantry Crane

20t gantry is planned to be provided to operate the stoplogs. The gantry crane shall be centrally lifted. The crane shall move on gantry girders. Walkway shall also be provided along the gantry girder. The gantry controls shall be provided in operator cabin. The gantry shall be designed as per IS: 3177 and IS: 807.

4.6.2 Kesari Barrage

Kesari Barrage has been proposed over River Keoton a tributary of Betwa river. Following data has been provided for the design of Barrage

1. Maximum Observed Non Monsoon 25 year flood = 109 cumec
2. Pond Level = 403.09 m
3. Average River Bed Level = 399.00 m
4. High Flood Level = 405.60 m
5. Standard Project Flood = 2772 cumec
6. Silt Factor = 1

Design Flood

As per Cl 5 of IS 6999-Part 1 : 'Hydraulic Design of Barrages and Weirs- Guidelines Part I alluvial Reaches' design flood is to be considered . For purposes of design of items other than free board, a design flood of 50 year frequency may normally suffice. In such cases where risks and hazards are involved, a review of this criteria based on site conditions may be necessary. For designing the free board, a minimum of 500 year frequency flood or the standard project flood [see IS 5477 (Part 4): 1971] may be desirable. Therefore design flood is proposed as 2772 cumec (SPF). The details of design of the Barari barrage are given at Annexure 4.13**.**

4.6.2.1 Sediment Data

Since the catchment of the Keoton at Kesari barrage site is marginal no significant sediment flow is expected. A silt factor of 1 is adopted for the design of the barrage.

4.6.2.2 Assumed Retrogression

The d/s unretrogressed HFL is 405.60 m and an afflux of 0.4 m is assumed initially.

4.6.2.3 Looseness Factor

For the given flood discharge, the Laceys water way is worked out as 254.3 m and the total water way provided is 181 m. The looseness factor is worked out as 0.711

4.6.2.4 Scour Depth

With looseness factor less than one the Scour depth is worked out as 8.33 m where as with looseness factor more than one, the scour depth is 6.65 m. As such the scour depth adopted is 8.33 m

4.6.2.5 Intensity of Discharge under Design Flood Condition

The intensity of discharge under design flood condition is 14.74 cumec/m, against the total length of spillway bays of 100 m and the discharge through spillway equal to 1474.17 cumec. The discharge through under sluice is 1297.24 cumec.

4.6.2.6 Co-efficient of Discharge

The coefficient of discharge for the under sluices (drowned weir formula) is 1.36 and drowning ratio is 0.94. The coefficient of discharge for the spill way is 1.47.

4.6.2.7 Exit Gradient Value

The exit gradient shall be determined from accepted formulae and curves. The factors of safety for exit gradient for different types of soils shall be as follows:

a) Shingle : 4 to 5,
b) Coarse sand : 5 to 6, and
C) Fine sand : 6 to 7.

The exit gradient value is worked out as 0.170148 or 1 in 5.9

4.6.2.8 Stresses Allowed

The residual stresses at various points from u/s sheet pile to d/s sheet pile are 100%(at E1 – top of u/s sheet pile), 83.01% (at D1- bottom of u/s sheet pile) , 81.23% (at C1 – top of u/s sheet pile – inside face), 36.46% (at E2-top of d/s sheet pile – inside face), 25.05% (at D2-bottom of d/s sheet pile) and 0% (at C2 – top of d/s sheet pile).

4.6.2.9 Type (Concrete/Masonry)/Profile, Cut off, Upstream and Down Stream Aprons, Uplift Pressure Relief Arrangements etc.,

Sheet pile cutoff is proposed for safe exit gradient. Various components of the Barrage e.g., raft, piers, flexible apron etc have been designed by the relevant provisions of IS Codes. Various drawings of Kesari barrage given in (drawing no. KBL – 7320 – D 2709 to 2710) are appended as Plate. 4.216 and 4.217 respectively in drawing volume – IV.

4.6.2.10 Gates, Types of Gates and Hoist Bridge and Stop Logs

The Spillway of Kesari Barrage shall be provided with 10 numbers fixed wheel vertical lift gates of size 10,000mm X 2400mm. Sill level of the gate shall be EL 401.50m. The gate shall be provided with upstream skin plate and upstream sealings to avoid silt on horizontal girders. Wedge type seal shall be provided for bottom sealing and music note solid bulb seals shall be provided for side sealings. The seal shall remain in contact with stainless steel seal seats to make the gate water tight. The gate structure shall consist of skin plate stiffened by vertical stiffeners and horizontal girders. The horizontal girders shall be supported by end vertical girders on each side. The water thrust will be transferred to concrete structures from the end vertical girder through wheels and wheel track. The wheels shall be mounted on self lubricating bush bearings. The wheel shall be made up of corrosion resistant steel. The BHN of wheel track shall be 50 BHN higher than the wheel material. 20 mm guide and two number guide shoes shall be provided on each side to guide the gate in grooves. The gates shall be operated by rope drum hoist of 20 t capacity. The Rope Drum Hoist shall consist of hoist machinery mounted on hoist support structure. Each hoist machinery will be equipped with two rope-drums, gears, pinions, couplings, shafts, worm reducer, motor and brakes. The hoist bridge shall be supported on trestles. Drawing showing spillway vertical lift gates of Kesari barrage (KBL – 7230-P-1517) is appended as Plate. 4.218 in drawing volume – IV.

Under Sluice Gates

The Kesari barrage shall be provided with 5 numbers under sluice gates of size 10000mm x 4900mm. The sill level of gate shall be kept at EL 399.00M. The gate shall be provided with upstream skin plate and upstream sealings to avoid silt on horizontal girders. Wedge type seal shall be provided for bottom sealing and music note solid bulb seals shall be provided for side sealings. The seal shall remain in contact with stainless steel seal seats to make the gate water tight. The skin plate of gate shall be stiffened by vertical stiffeners and horizontal girders. Numbers of girders and their location shall be optimized at design stage. The gate shall be operated by Rope Drum Hoist. The hoist shall be capable of lifting the gate above groove for servicing of gate. The hoist machinery shall be supported on hoist supporting structure. The gate shall be regulating type and can be kept at desired level between sill and FRL. Drawing showing under sluice gate of Kesari barrage (KBL-7230-P-1518) is appended as Plate. 4.219 in drawing volume – IV.

Spillway / Under Sluice Stoplogs

Stoplogs are proposed to carry out the maintenance of spillway gates and under sluice gates. Four sets of stoplogs of size 10000mm X 1300mm shall be provided. The stoplogs shall be operated in balanced water head conditions. However, the top most unit can be lifted under unbalanced water head for one unit height water head. The stoplogs shall be operated with gantry crane. The stoplog units shall be stored in stoplog groove above MWL. The stoplog units shall have bronze pad sliding on stainless steel track. 20mm guide shall also be provided to guide the stoplog units. The skin plate and sealing gate shall be kept d/s side. Drawing showing spillway stoplog gates (KBL-7230-P-1519) & under sluice stoplog gates (KBL-7230-P-1520) of Kesari barrage are appended as Plate. 4.220 and 4.221 respectively in drawing volume – IV.

Gantry Crane

15t gantry is planned to be provided to operate the stoplogs. The gantry crane shall be centrally lifted. The crane shall move on gantry girders. Walkway shall also be provided along the gantry girder. The gantry controls shall be provided in operator cabin. The gantry shall be designed as per IS: 3177 and IS: 807.

4.6.3 Canals of Upper Betwa System

Under the Upper Betwa component four canal systems are proposed viz., Makodia RBC, Makodia LBC, Barari RBC and Kesari RBC.

4.6.3.1 Makodia Dam Canal System - Right Bank Canal

The alignment of Makodia Right Bank Canal has been marked on the strip survey contour sheet for every 1.5 Km. Following data has been provided as Longitudinal Section table with every Drawing

1	R.D (metre)
2	NSL
3	Canal Bed Level
4	Bed Slope (nH:1V)
5	Distributary FSL
6	Distributary Discharge
7	Design Discharge (Cumecs)
8	FSL
9	F S D
10	Bed Width
11	Head Loss (Bed Slope)
12	Head Loss (CD Structure)
13	Manning's Rugosity Coeff
14	Side Slope (nH:1V)
15	Flow Velocity (at FSD)
16	Free Board
17	Canal Top Level
18	Canal Top Width
19	Depth of Cutting
20	Depth of Filling

The alignment of canal with starting with RD 0 m at FSL of 431.000 m to RD 83380 m at FSL of 418.932 has been divided into five reaches. Each reach has different slope and cross section elements. All the 55 sheets containing the L.S. and strip contour maps of Makodia Right Bank canal are given as drawings sheet No KBL-7320-D- 2601 to 2656 which are appended as Plates No. 4.222 to 4.277 respectively in Drawing volume -IV.

Considering relevant BIS codes as discussed in respect of Ken – Betwa link canal, all the curve have been proposed with radii of 200 m for discharge upto 10 cumecs and 150 for other reaches.

a) Cross Section and Lining

The design of cross section has been done as per provisions of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining' . The cross section has been designed on Manning's formula. The FSD at head is 2.5 m and bed width is 5.4 m. The length of the canal is 83.38 km. The details are given in Drg No KBL-7320-D-2657 and 2658 which are appended as Plate. 4.278 and 4.279 in Drawing volume – IV.

b) Capacity of RBC canal

Capacity has been taken as 21.241 cumec from head reach to 2.998 cumec at end. The subtractions are done for discharge in each distributary of a reach and transmission losses are not taken into account for greater flexibility of the system.

c) Shape

The shape has been selected as trapezoidal with rounded corners as per provisions of IS 3873: 'Laying cement concrete/stone slab lining on canals - Code of practice' for ease in laying of lining. The bed width reduces from head to tail with change in the water depth reach wise so that intended flow velocity is generated with ease in construction.

d) Manning's n

As already discussed considering the provisions of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining' the value of N is adopted 0.018 to compensate for increased resistance due to curves and for taking into account increased resistance due to deterioration of lining with time.

e) Free board

As per CL 8.2 of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining' the value of free board has been taken as 0.75 m from RD 0 to RD 72700 (reach 1 to 4) and 0.60 m from RD 72700 to end (reach 5).

f) Side Slope

As per Cl 8.1.1 of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining' the side slope may be adopted as 1.5 h :1 V for cutting as well as for filling.

g) Bed Slope

Bed slope has been provided so that following purposes are fulfilled:

1. The section is able to safely carry the design discharge from Makodia Dam to end of canal.
2. The required head is available for each distributary for irrigation / drinking
3. The cutting and filling is balanced as far as possible

It has been observed that due to topography the canal is running into cutting for most of the reach.

4.6.3.2 Study of Integrated Network of Canal System and its Operation to Utilise the Water Potential of Streams crossed by Main Canal System by Provision of Storage / Tail Tank etc.

Detailed study of integrated network of canal system will be taken up at pre construction stage.

4.6.3.3 Description of Soil Profile

Representative soil samples have been collected approximately at every 5.0 km interval along the Makodia RBC. Based on the results of tests, It is suggested to conduct further soil investigations at lower intervals of distance during construction stage.

4.6.3.4 Evaluation of Design Parameters, Suggested Treatment for Problematic Reaches.

For this purpose the following design-criteria based on the relevant provisions of IS codes, and established design practices is adopted.

1. Non silting non scouring velocities should be generated for efficient sediment transport.

2. The system should be flexible to cater to any combination of requirement of irrigation of enroute command and water transfer

3. The Transmission losses should be minimum.
4. The canal should not overtop its lined section in any reach.
5. Designed discharge should be available all the distributaries at designed waterlevel.
6. Deep cutting or high embankments are to be avoided and cutting and filling should be balanced as far as possible.

7. Head loss due to various structure and section resistance should be minimum and compatible with design etc.,

4.6.3.5 Details of Lining

The lining adopted is concrete lining of thickness 75 mm from RD 0 to RD 72700 (reach 1 to 4) and 60 mm from RD 72700 to end (reach 5)as per provisions of IS 3873: 'Laying cement concrete/stone slab lining on canals - Code of practice'. It is also proposed to provide CNS layer as per provisions of IS 9451: Guidelines for Laying of Canals in Expansive Soils'. IS 9451: Guidelines for Laying of Canals in Expansive Soils' IS 9451: Guidelines for Laying of Canals in Expansive Soils' which may be further optimized or altered on the basis of swelling index tests of the soil encountered. The lining is proposed to be unreinforced. The pressure release arrangements as per provisions of IS 4558 : 1995 'Code of practice for under-drainage of lined canals' are provided to release water pressure behind lining due to rise in ground water level and canal empty condition. In the absence of data on the ground water level variation, it is proposed to provide 30% of the canal lined section with pressure release arrangement. The lining shall however be reinforced at the junction of lining with any CD structure on upstream and downstream for a distance of 50 m.

4.6.3.6 Transmission Losses

As explained earlier transmission losses are not taken into account for greater flexibility of the system.

4.6.3.7 Cut off Statement Showing the Details of the Discharge Required from Tail to the Head

Details about the discharge required from tail to the head considering the irrigation requirement and transmission losses in taking of channel of distributaries outfalling from Makodia RBC will be finalised during pre construction stage.

4.6.3.8 Design Calculation for Adequacy of Canal Sections Adopted

Design of Makodia RBC Cross Sections in various reaches are given at Annexure 4.14.1 to 4.14.5. The design of canal sections has been checked for passing 25% additional design discharge. It is seen that it is possible to pass this additional discharge with in free board available in the canal.

4.6.3.9 Design Discharge Data for Each Distributaries

The Makodia RBC is designed to provide irrigation and drinking water to command area in its reach. Information on the distributary network is furnished in every L section of 1.5 km of the length of the canal, for the distributary falling in that reach. Details of distributaries outfalling from Makodia RBC are given in Annexure 4.15 . The shape of the section adopted is cup shaped as per codal provisions and established design practices. The depth of triangular portion is shown in the annexure referred above. The section has been designed for maximum flow and bed slope of 1 in 10000 in the absence of detailed data. The details of distributary are given in Drg No KBL-7320-D- 2666 which is appended as Plate. 4.280 in Drawing volume – IV.

4.6.3.10 Canal Structures across Makodia Right Bank Canal

(i) Cross regulators

15 cross head regulators will be provided at various locations of the Canal as shown in Annexure 4.15 referred above, for providing design

discharge to distributary at required water level. Typical drawings of provided Cross regulators viz., CR1, CR2 and CR3 are given as Drg No. KBL-7320-D-2667, 2668 and 2669 are appended as Plates No. 4.281, 4.282 and 4.283 respectively in the Drawing volume – IV. Head regulator has not been proposed for the distributaries in view of the small discharge to be diverted. Instead the water is diverted to distributary through MS pipe of appropriate diameter. To regulate the flow valve is proposed to be provided.

Gates for cross regulators

The cross head regulators shall also be provided at various locations of the Canal. There will be 11 Nos. of Cross Regulators. Four Cross Regulator shall have three gates of size 4000mm X 2500mm. Six Nos. Cross Regulators shall have two gates of size 5000mm X 2250mm. One Cross Regulator shall have two gates of size 4000mm X 1300mm.There will be one Service gate of Fixed Wheel Type Vertical Lift Gate in each opening. The gates shall be provided with upstream skin plate and upstream sealings. Wedge type seal shall be provided for bottom sealing and music note solid bulb seals shall be provided for side sealings. The seal shall remain in contact with stainless steel seal seats to make the gate water tight. The gate structure shall consist of skin plate stiffened by vertical stiffeners and horizontal girders. The horizontal girders shall be supported by end vertical girders on each side. The water thrust will be transferred to concrete structures from the end vertical girder through wheels and wheel track. The wheels shall be mounted on self lubricating antifriction bearings. The wheel shall be made up of corrosion resistant steel. The BHN of wheel track shall be 50 BHN higher than the wheel material. 20 mm guide and two number guide shoes shall be provided on each side to guide the gate in grooves. The gates shall be operated by rope drum hoist of suitable capacity. The Rope Drum Hoist shall consist of hoist machinery mounted on hoist support structure. Each hoist machinery will be equipped with two rope-drums, gears, pinions, couplings, shafts, worm reducer, motor and brakes. The hoist bridge shall be supported on trestles. Drawing showing typical cross regulator service gate of RBC Makodia dam (KBL-7230-P-1526) is appended as Plate. 4.284 in Drawing volume - IV.

ii) Structure at the beginning and end of Canal

The Canal shall originate from the sluice provided in the earthen Makodia dam section. Therefore there is no need for any other regulating structure in the beginning.

Makodia dam - Right Bank Canal - Intake Gates

Service and Emergency gates shall be provided in canal Intake. The gates shall be Fixed Wheel Vertical Lift type. The gates shall be fabricated in one unit. Service gate shall have d/s skin plate and d/s sealing while the Emergency gate shall have u/s skin plate and u/s sealing. The gates structure shall consist of skin plate stiffened by vertical stiffeners and horizontal girders.

The horizontal girders shall be supported by End vertical girders provided on each side of gate. The water thrust shall be transmitted to the concrete from horizontal girder by wheels and track embedded in the concrete. The wheels shall be mounted on self lubricating bush bearing. The gates shall be designed to withstand water load corresponding to FRL. These gates shall be operated by independent rope drum hoist mounted on common support structure. The support structure shall be installed at pier top. Each Hoist Machinery shall consist of rope drum, gears and pinions, electric motor and electro –magnetic brakes. Drawing showing emergency service gate for Makodia dam RBC intake general installation (KBL-7230-P-1527 & 1528) are appended as Plate. 4.285 and 4.286 respectively in Drawing volume - IV.

No structure has been proposed in the end also as the canal is meant entirely for serving the command and no water transfer is envisaged.

iii) Cross drainage structures

Below every 1.5 km L – Section of the Makodia RBC given as Plates No.4.222 to 4.277, data on CD works proposed in the corresponding reach is given. Following CD works are proposed

1. Major Bridges
2. Minor Bridges
3. Aqueduct
4. Syphon Aqueduct
5. Super Passage
6. Canal Syphon
7. Escapes

Wherever practical, siphon aqueduct is preferred over canal siphon for lower headloss. The design of bridges is adopted as per provisions of IRC standard designs for Bridges. The typical design details of bridge is given as Drg No KBL-7320-D- 2660 and the same is appended Plate. 4.287 in Drawing volume - IV. The design details of typical Syphon Aqueduct (Drg no. KBL-7320-D- 2663) and Super passage (Drg no. KBL-7320-D- 2665) are appended as Plates No. 4.288 and 4.289 respectively in Drawing volume – IV. A minor bridge shall be combined with every Aqueduct, Syphon Aqueduct and Cross Regulator to ensure atleast one passing over in every 1.5 km. The Bridges are given in the L section for details refer the L- Section Drawings. The design criteria for the above is as per applicable IS standards. The list of CD/CM structures proposed across the Makodia RBC is furnished as Annexure 4.16.

4.6.4 Makodia Dam - Left Bank Canal

The alignment has been marked on the strip survey contour sheet for every .5 km. The alignment of canal with starting with RD 0 at FSL of 431.000 to RD 6700 at FSL of 430.256 has design discharge of 0.356 cumecs only. Therefore only one reach has been designed. The L-section of the canal is given as Drd. No. KBL-7320-D- 2671 to 2675, which are appended as Plate. 4.290 to 4.294 respectively in Drawing volume – IV.

The radius of the curves along the Makodia LBC has been proposed to be 60 m as per BIS code provisions.

i) Cross Section and Lining

The design of cross section has been done as per provisions of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining'. The cross section has been designed on Manning's formula. The full supply depth at head is 0.786 m and FSL at head is 431.00m. The total length of the MLBC is 6.70 km. The design details of the cross section are given in Drg No KBL-7230-D-2676 which is appended as Plate. 4.295 in Drawing volume - IV. The lining adopted is concrete lining of thickness 60 mm as per BIS code provisions.

ii) Capacity

Due to small discharge of 0.356 cumecs only, the section of Makodia Left bank canal has been considered with uniform section throughout its reach of 6.7 Km. At the time of detailed planning for construction, this may be optimized by tapering the section from head to tail.

iii) Shape

The shape has been selected as cup shaped with rounded corners as per provisions of IS 3873: 'Laying cement concrete/stone slab lining on canals - Code of practice' for ease in laying of lining.

iv) Manning's n

The Manning's N is taken as 0.018 to compensate for increased resistance due to curves and for taking into account increased resistance due to deterioration of lining with time in accordance with Cl 4.1.2.1 of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining'.

v) Free board

As per CL 8.2 of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining' the value of free board has been taken as 0.30 m for all the alignment of 6.7 km.

vi) Side Slope

As per Cl 8.1.1 of IS 10430: 'Criteria for design of lined canals and guidelines for selection of type of lining' the side slope adopted is 1.5 h :1 V for cutting as well as for filling.

vii) Bed Slope

In the absence of distribution data and considering the small size of canal, the bed slope has been taken as 1 in 9000 for all alignment

viii) Lining

The lining adopted is concrete lining of thickness 60 mm as per provisions of IS 3873: 'Laying cement concrete/stone slab lining on canals - Code of practice'. It is also proposed to deploy a HDPE geomembrance as per provisions of IS 9698: 'Lining of Canals with Polythene films-code of Practice' behind the lining to further reduce losses. The lining is proposed to be unreinforced. The pressure release arrangements as per provisions of IS 4558 : 1995 'Code of practice for under-drainage of lined canals' are provided to release water pressure behind lining due to rise in ground water level and canal empty condition. In the absence of data on the ground water level variation, it is proposed to provide 20% of the canal lined section with pressure release arrangement.

ix) Structure at the beginning and end of Canal

The Canal shall originate from the sluice provided in the earthen Makodia dam section. Therefore there is no need for any other regulating structure in the beginning.

No structure has been proposed in the end also as the canal is meant entirely for serving the command and no water transfer is envisaged.

x) CD structures across Makodia LBC

Below every L – Section data on CD work proposed in the corresponding range is given. Since the canal section is very small, no heavy CD works are proposed. The canal may be taken through a pipe for any crossing as per site conditions.

xi) Distribution Network

Makodia Left Bank Canal is designed to provide irrigation and drinking water to its command area. Due to small command area, further distributaries are small and are designed as cup shaped.

xii) Cross Regulator

In view of the small size of the canal Cross regulators with simple arrangements are proposed. Head regulator has not been proposed for the distributaries in view of the small discharge to be diverted. Instead the water is diverted to distributary through MS pipe of appropriate diameter

4.6.5 Barari Barrage - Right Bank Canal

The canal taking off from the barrage is from right bank only. Initially the water will be lifted through a height of 21 m (static) over a length of 4.0 km through a pipe line and there after it runs by gravity for 4.7 km. Due to low discharge of diversion canal (1.12 cumec only) and lifting of water to command, Head Regulator has not been proposed with Barrage. If more command or water is made available at a later stage due to change if any in overall planning of the Project, diversion arrangements may have to be modified. The full supply depth of the canal in its head reach is 1.18 m and FSL at head is 424.00 m. The bed slope adopted is 1 in 10000. Since the canal is aligned as a ridge canal no C.D/ C.M works are proposed. The details of L section of the canal and its cross section is illustrated in Drg no KBL-7320-D-2705 to 2707 which are appended as Plates No. 4.296, 4.297 and 4.298 respectively in Drawing volume – IV.

4.6.7 Kesari Barrage - Right Bank Canal

The canal taking off from the Kesari barrage is from right bank only. Initially water will be lifted through a height of 8 m (static) over a length of 2.9 km through a pipe line and there after it runs by gravity for 9.6 km Due to low discharge of diversion canal (1.07 cumec only) and lifting of water to command, Head Regulator has not been proposed with Barrage. If more command or water is made available at a later stage due to change if any in overall planning of the Project, diversion arrangements may have to be modified. The full supply depth of the canal at its head is 1.16 m and FSL at Head is 409.50 m. The bed slope adopted is 1 in 10000. Suitable CD/CM works are proposed along The details of L section of the canal and its cross section is illustrated in Drg no KBL-7320-D-2720 to 2726 which are appended as Plates No. 4.299 to 4.305 respectively in Drawing volume – IV.

4.7 Infrastructure Studies

Almost entire area of the project is well connected with road and rail net work. No constraints on transportation of heavy equipment upto project site are envisaged as the area is well connected by NH-76 which can bear class-a loading. The weights of generating units, power transformers and other components etc to be transported by road or from the nearest railhead to project site shall be within the transport limits of 70 tonnes. For transportation of heavy machinery to the dam site some of the road bridges and culverts need to be strenghted at the time of preconstruction stage.

4.8 Industrial and urban use

The domestic and industrial requirement of the project area is proposed to be met from the proposed canal system. However necessary storage and distribution system are to be developed by the local authorities

4.9 Instrumentation

The requirement of special instruments for the construction of dam, Power House and canals are described in other chapters.

4.10 Navigation and Tourism development

There is no provision for development of navigation aspect in the project. As far as the Development of Tourism is concerned this project has got full potential, particularly because of it close proximity of Daudhan dam with khajuraho and that of tail end of link canal with orcha temple near Jhansi. Provision for development of tourist huts, picnick spot has been made on the periphery of Daudhan reservoir, Baruwa sagar and Makodiya reservoir.

4.11 Operation and maintenance

The proposed organizational set up at the construction stage can be made available for operation and maintenance of the project. The entire operation and maintenance will be looked after by one chief Engineer along with sufficient organizational set up. A suitable operation and maintenance programme has to be developed for meeting the various objects of the project.

4.12 Other studies

The other studies which are not coverd in the DPR stage will be planned at the preconstruction stage. No other study was required do be done at DPR stage. However, if any study is suggested by Design organization of CWC, the same may be carried out at pre-construction stage.