Annex B

Hydraulic Structures and Barrage Foundation

ANNEX B Hydraulic Structures and Barrage Foundation

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B.1. Taunsa Barrage Hydraulics Structure

(1) General Features

Taunsa Barrage on River Indus was constructed since 1954 to 1958. The Taunsa Barrage was put into operation in 1958. General feature of major hydraulics structure of the Taunsa Barrage are shown in drawings attached into.

Authorized designed data of Taunsa barrage are following, and all operation and maintenance are conducted on the bases of the same.

1	Design flood capacity	1,000,000 cusecs
2	Maximum flood passed	760,784 cusecs
3	Right under-sluice bays	4 Nos. of 60' each
4	Left under-sluice bays	7 Nos. of 60' each
5	Weir bays	53 Nos. of 60' each
6	Navigation bay on left	1 No of 22'
7	Fish ladder	2 Nos. of 11' each
8	Muzaffargarh canal bays	5 Nos. of 24' each
9	D.G. Khan canal bays	7 Nos. of 24' each
10	T.P. Link canal bays	7 Nos. of 24' each
11	Crest level in under-sluice	R.L. 425.0
12	Crest level in navigation bays	R.L. 425.0
13	Crest level in weir bays	R.L .428.0
14	Crest level in Muzaffargarh bays	R.L. 433.0
15	Crest level in D.G. Khan bays	R.L. 433.0
16	Crest level in T.P. Link bays	R.L. 433.0
17	Upstream floor level in under-sluices	R.L. 419.0
18	Upstream floor level in weir	R.L. 421.0
19	Downstream floor level in under-sluices	R.L. 415.0
20	Downstream floor level in weir	R.L. 416.0
21	Downstream floor level in Muzaffargarh bays	R.L. 430.15
22	Downstream floor level in D.G. Khan bays	R.L. 428.16
23	Downstream floor level in T.P. Link bays	R.L. 428.16
24	Bottom level of bridge	R.L. 452.0
25	Top level of bridge	R.L. 457.5
26	Top level of super-passage	R.L. 500.0
27	Height of gates in under-sluice	22 feet

28	Height of gates in weir	19 feet
29	Maximum pond level upstream	R.L. 446.0
30	Highest flood level upstream	R.L. 447.0
31	Highest flood level downstream	R.L. 444.0
32	Full supply level in off-taking canals	R.L. 444.0
33	Maximum head-across	22 feet (30' designed)
34	Length of impermeable floor	110 feet (U/S)
	of under-sluices/weir	123 feet (D/S)

(2) Structure Observation

Hydraulic structures of the Taunsa barrage and its related regulators were investigated in detail by each bay as follows:

- U/S and D/S floor structures of the Taunsa barrage, such as loose stone, concrete blocks, concrete floor, glacis, and friction blocks, were inspected and investigated these damages. Among the structural components by bays, damages on U/S glacis and friction blocks were prominent. The result is summarized in Table B1.1.1.

- Piers of the Taunsa barrage were observed as being in good condition, while minor chipping was found. No substantial repair works are required. In relation with deterioration of gate grooves, groove contact in the piers were scoured by leaking jet flow through roller guard. These portions should be repaired within the work of groove improvement.

- Canal regulators of D.G. Khan canal, Muzaffargarh canal and T.P. link canal were observed still being in moderate condition. No substantial repair works are required without gate repair.

B.2. Review of Hydraulic Design of the Taunsa Barrage

B.2.1. Review of Hydraulic Design of the Present Facilities

(1) Adequacy of Design Flood Discharge

Design flood discharge of the Taunsa Barrage had been designated at 1,000,000 Cs. Through passing 40 years since construction of the facilities, maximum discharge was observed at 760,784 Cs on July, 1958. Very high floods having peak discharge greater than 65,000 Cs occurred at triple times up to present. Histogram of annual highest floods and result of its frequency analysis is shown in Table B2.1.1 According to the frequency analysis adopted Gumbel Extreme distribution, 100 - year return period flood is obtained at about 823,000 Cs. Taking these results into consideration, the design flood of 1,000,000 Cs is regarded being unquestionable, which is supposed to be in several hundred years return period. As a reference, safe discharging capacity of sub-reach between Taunsa Barrage and Panjand Confluence was estimated at 800,000 Cs. Furthermore, design flood discharge of the Ghazi Ghat Bridge which locates within the sub-reach, coindides with the same of the Taunsa Barrage at 1,000,000 Cs.

The design flood for Taunsa Barrage was based on the data for rim stations available till 1952. Annual flood data series applied in above analysis are different from the data used when it was designed, because of parameters' change like incorporation of the effects of the Tarbela and Chashma Sub-Reservoir, and recent climate change. The purpose of this probability analysis is to evaluate adaptability of the design figure for the future events. In order to follow the purpose, annual flood data series applied in above analysis were limited within recent values without data before construction.

(2) Adequacy of Overall Waterway Provided

Actual overall waterway of the Taunsa Barrage provided is 4,335 feet (1,321.308 m). As the reference indicator of suitable width of waterway, Lacey's wetted perimeter P ($P=4.75 \cdot Q^{1/2}$ in metric unit) is calculated at 2,622.56 feet (799.36 m) applying 1,000,000 Cs of the design flood Q. The actual overall waterway of 4,335 feet being against the Lacey's wetted perimeter of 2,622.56 feet is in safe side with the looseness factor of 1.65 (=4,335 / 2,622.56). Accordingly width of waterway of the Taunsa Barrage is generally regarded as a proper and adequate design.

(3) Dimensions of Hydraulic Structure

Hydraulic design of barrage involves determination of the weir section and the details of its U/S cutoff, crest, glacis, floor, D/s cutoff, U/S and D/S protection works, etc. The hydraulic design of weirs on permeable foundation may be classified into:

- Design for Sub-surface Flow

- Design for Surface Flow

Design for Sub-surface Flow is mentioned in another section. Hereunder, design aspect for surface flow is discussed.

Scouring problem is a highlighted appearance in surface flow aspect. The aspects of surface flow are scour considerations. D/S cutoff or sheet piles at the ends must go below the deepest anticipated scour level. Famous Lacey's extensive investigation on regime channels is much useful for the scouring events. For a discharge intensity q, the normal depth of scour (R) is given by the Lacey's equation, i.e.

$$R = 1.35\ (q^2/f)^{1/3} \qquad \text{(in metric unit)}$$

It is recommended to take each bay section down to the level obtained by measuring the depth of scour R, below the H.F.L. A value of above 1.25 R on U/S side and 1.50 R on D/S is widely accepted.

Item	Unit	Weir Bay	Under-sluice Bay
Discharge intensity (q)	cu.m/s	23.228	29.732
Lacey's silt factor f	-	0.79	0.79
Depth of scour (R)	m	11.930	14.060
Velocity of approch	m/s	2.114	2.295
Designed U/S cutoff depth from U/S W.L.(RL.446.0)	m	10.973	11.582
Designed D/S pile depth	m from D/S W.L.(RL.443.0)	14.630	14.935
D/S pile depth from retrogressed D/S W.L. (RL.434.4 resent value)	m	12.009	12.314

While designed piles or cutoff depth does not hold a sufficient margins, it is in safety range.

The total length of the concrete floor is mainly governed by the exit gradient (GE) considerations. For the safe exit gradient, and also suitable depth of downstream cutoff taking scour considerations into consideration, the length of the horizontal floor "b" can be worked out as $b = \alpha \cdot d$ (being known when GE is fixed). Designing of the Taunsa Barrage, above α is estimated as follows:

α (Weir Bay) = 71.02(233 ft)/ 6.40(21 ft) = 11.10

α (Under-sluice Bay) = 72.54(238 ft)/ 6.40(21 ft) = 11.33

Item	Unit	Weir Bay	Under-sluice Bay
d	feet	21.00	21.00
	m	6.40	6.40
b	feet	233.0	238.0
	m	71.02	72.54
α	-	11.10	11.33
λ	-	6.07	6.19
H (in 30 ft)	m	9.14	9.45
Exit Gradient (GE)	-	0.185	0.189

When foundation soil is categorized in "Coarse sand", safe exit gradient is reneged from 0.20 to 0.17. Considering this criteria on the exit gradient, designing of Taunsa barrage can be regarded as normal assuming "Coarse and Fine sand" of its foundation soil.

Length of D/S concrete floor is much important to keep safe against progress of retrogression. The length of D/S concrete floor shall be designed in consideration with shape of expected hydraulic jump. Present design length of D/S concrete floor as well as other dimensions of protection structures are summerized as follows:

Lengths of D/S Protection Structures

meter(feet)

	Required length when designed	Design length	Remarks
Weir Bay			
D/S Concrete Floor	24.26(79.59)	24.08(79.00)	
C.C.Blocks Apron	17.90(58.71)	17.32(56.83)	1.5D(depth of scour)
Loose Stone	23.86(78.28)	24.57(80.60)	2.0D(depth of scour)
Total length	66.01 (216.58)	65.97 (216.43)	
Under-Sluice Bay			
D/S Concrete Floor	26.03(85.38)	26.82(88.00)	
C.C.Blocks Apron	21.09(69.19)	17.32(56.83)	1.5D(depth of scour)
Loose Stone	28.12(92.26)	27.43(90.00)	2.0D(depth of scour)
Total length	75.23 (246.83)	71.57 (234.83)	

B.2.2. Present Situation of Retrogression

(1) Introductory

Retrogression of levels is a process triggered by construction of pickup weirs or humps across natural streams in alluvial and permeable foundation environments, and results in degradation of stream bed with subsequent lowering of water levels on the downstream of the weir. The construction of a weir and canal head-works across a river effects the river regime and channel equilibrium to bring about the following changes:

- The slope of the river upstream of the weir flattens due to ponding up of the supplies.

- Due to above the bulk of silt charge of river water deposits in the pond, leading to the formation of irregular shoals.

- A progressive degradation / retrogression of bed levels downstream, due to picking up of bed silt by relatively silt free water escaping over the weir.

- These effects continue for the first few years but later due to continuous silting up of the pond and increasing tortousity, the bed levels of the stream will tend to rise as the bed levels at the weir are fixed. An increase in tortuosity will necessarily enhance the rise f levels due to weir afflux with the result that this rise will be felt higher up the river than would otherwise be the case. As a result of this progressive rise of bed levels, there will be tendency on the part of the river to regain its original slope.

- A stage will come when upstream pond absorbs no further silt burden. Owing to the off-taking canals drawing comparatively silt free water, the excess of silt will go downstream of the weir while the amount of water passing over the weir will be below the normal due to canal withdrawals. The river below the weir, will thus, have to carry an excessive silt change with a lower water discharge. This will result in progressive silting up on the downstream, an increase in tortousity and therefore, a recovery of bed levels downstream.

(2) Retrogression of Levels

In the first few years following the construction of a weir, the retrogression of bed levels downstream is rapid and progressive. Generally this ranges between 4 and 7 feet. This lowering of the bed levels in the early stages if not duly allowed for in design may result in a failure like that of the Islam weir in 1929. The retrogression may undermine the stability of a work by an increase in the exit gradient beyond the safe limits. It will increase the destructive action of the hydraulic jump as with the increase fall and decrease depth of downstream water due to the lowering of the water levels at that end, and jump will tend to travel down to the C.C.blocks and loose stone protection area.

As a result of the retrogression in bed levels, while low water levels have been found to drop from 4 to 7 feet, the maximum flood levels have not been known to have dropped by more than 1.0 to 1.5 feet. The initial magnitude of retrogression depends on crest levels, discharge, sediment load and extent of pond area.

In the course of time the river upstream will regain its original slope which implies that the affect of afflux due to the construction of the weir will not be confined in magnitude and length to the usually accepted distance as determined by back water curve. It will travel further up and will be felt in the full, all that distance.

The process of recovery of downstream bed levels after the initial retrogression, is slow but steady. It may take about 20 years but the bed levels may in some cases rise to higher values than those existing before the construction of the weir.

(3) Provision in Design

Taunsa Barrage had to be located at an un-gauged site. The gauge discharge relationship for design of Taunsa Barrage was determined on the basis of the minimum gauges recorded at Ghazi Ghat during the period 1930 to 1939. Two options were considered and the adopted alternative, provided for a retrogression of 4.7 ft at 10,000 Cs, 2.5 ft at 200,000 Cs and 1.3 ft at 900,000 Cs. as showing a table below.

Statement Showing a Comparison of the Minimum Tail Water Rating Levels and Minimum Retrogression Tail Water Levels

Dischrge in 10^5 Cs	Minimum Rated Tail Water Levels	Minumum Retrogression Tail Water Levels	Magnitude of Retrogression Provided
0.4	432.00	428.35	3.65
0.8	434.40	431.30	3.10
1.0	435.15	432.15	3.00
2.0	437.50	435.00	2.50
3.0	438.90	436.70	2.20
4.0	439.90	438.00	1.90
5.0	440.70	439.00	1.70
6.0	441.30	439.70	1.60
7.0	441.80	440.30	1.50
8.0	442.30	440.90	1.40
9.0	442.70	441.40	1.30
10.0	443.00	441.80	1.20

The calculation model some how erred in asuuming the river slope and rather high water levels were made the base of the barrage design. The barrage therefore has a rether built-in element of lowering of water levels on the downstream. This is confirmed by the fact that water levels experienced during the construction period (1954-1958) were observed to be substantially lower than even designed minimum retrogressed levels. The situation is shown in following table.

Summary of Minimum Gauge-Discharge Relation at Taunsa Barrage Site (From Minimum Envelope Lines)

Dischrge in 10^5 Cs	Calender Years				Designed Minimum Retrogression Levels
	1954	1955	1956	1957	
0.3	429.6	426.5	429.3	430.9	-
0.4	430.3	427.7	430.2	431.8	428.35
0.5	430.9	428.5	430.8	432.5	-
0.8	432.2	430.4	432.2	433.8	431.30
1.0	432.8	431.3	432.8	434.5	432.15
2.0	434.7	434.1	434.8	436.5	435.00
3.0	435.9	435.7	435.9	437.7	436.70
4.0	436.7	436.9	436.7	436.6	438.00
5.0	437.3	437.7	437.3	439.2	439.00
6.0	437.8	438.5	437.9	439.8	439.70
7.0	438.3	439.1	436.3	440.2	440.30
8.0	438.7	439.7	438.7	440.7	440.90
9.0	439.0	440.1	439.0	441.0	441.40
10.0	439.3	440.5	439.3	441.3	441.80

(4) Actual Performance

As explained in the above paragraphs, the retrogression phenomenon is generally experienced at all new barrages in the early period after commissioning. However, after the bed configuration in the upstream pond area stabilizes, the retrogression cycle is reversed and the bed configuration on the downstream of the barrage reverts to normal condition with water levels generally close to design values.

At Taunsa Barrage, the conditions have not varied in the usual / normal course. The barrage is experiencing tail-water levels that are substantially lower than even the minimum designed retrogression levels, although 40 years have elapsed since its commissioning.

This problem attracted the attention of the barrage staff in the Mid-sixties and their reports indicated a perpetually worsening situation. Model tests were therefore conducted in the Lahore Laboratory of Irrigation Research Institute on the Sectional Model and the minimum water levels required for formation of hydraulic jump at the design location were determined for "With-blocks" and "Without-blocks" scenarios both for the undersluices and weir bay sections. These values are presented in a table below. Model tests were restored to because the normal mathematical method of analysis are not fully applicable to the analysis of stilling basins, with energy dissipation appurtenances like impact blocks, deflector blocks etc.

Minimum Tail Water Levels Required for Formation of Stable Hydraulic Jump

Intensity	Undersluices		Weir	
(Cs/ft)	With-blocks	Without-blocks	With-blocks	Without-blocks
25.0	420.8	-	422.3	424.6
50.0	422.0	425.0	424.4	427.3
100.0	426.5	428.5	429.0	431.5
150.0	430.5	431.5	432.2	433.9
200.0	432.6	434.2	434.8	436.0
250.0	434.8	436.4	437.0	438.3
300.0	437.5	438.2	439.9	440.3
350.0	438.4	439.6	-	-

(5) Actual Conditions

As mentioned above the water levels actually observed on the downstream of the barrage have all along been lower than the design levels by varying magnitudes recorded in the barrage gauge records.

The lowering of tail-water levels below the designed minimum levels causes reduction in tail water depths. This naturally results in depths less than the conjugate depths required for formation of a good hydraulic jump at the proper place and level for effective energy dissipation. The difference in actual and designed levels, results in the formation of an unstable jump, much below the required safe levels with a secondary jump at the end of the impervious floor and on the block apron, which causes heavy scour and negative pressures in the block area and loose stone apron and consequently lifting and washing away of the apron stone and settlement of the PCC settling blocks.

To analysis the present retrogression status, the gauges and discharges observed on the downstream were plotted on semi-log graph paper and minimum and maximum level envelope lines drawn which allow for inter / extrapolation of levels for discharges other than actually experienced. Such envelope curves / lines, for the recent years appear as Fig.B2.2.1 to B2.2.5. The minimum tail-water-levels obtained from semi-log plots of downstream gauges from 1954 to 1997 are listed in Table B2.2.1 wherein the levels for discharges higher than those actually experienced are extrapolated values. A comparison with the design minimum retrogressed levels is also exhibited by this table. These levels have been used for plotting and presented in Fig.B2.2.6 and compared with the designed tail water rating levels and designed minimum retrogressed levels, which clearly indicated that, although a couple of recovery cycles have been experienced, still the retrogression problem does exist in substantial magnitude. The magnitude of retrogression for 1997 (most recent) levels is exhibited in Table B2.2.2 and Fig.B2.2.7.

The minimum levels required for formation of stable hydraulic jump appear in table presented above alongwith the designed assumption and latest minimum tail water levels for 1997 in Fig.B2.2.8 and B2.2.9. The point of the inter-section of jump limit curve with actual minimum tail water level line, indicate the present Barrage capacity which comes to 5×10^5 cusecs for stable jump situation ensuring barrage safety. This capacity is very much below the design discharge of 1,000,000 cusecs (only 50 %). In view of the construction of additional storages in the offing, the flood discharge at Taunsa Barrage are likely to be contained to a figure below the design discharge of 1,000,000 cusecs. However, this situation warrants immediate remedial measures to eliminate this persistent retrogression trend.

B.2.3. Hydraulic Jump Analysis

(1) Parameters for Hydraulic Jump Analysis

The performance of stilling basin is a Key element in the assessment of the hydraulic behaviour and safety of barrage with reference to surface flow. Hydraulic jum analysis is conducted hereinafter. The design parameters of Taunsa Barrage relevant to surface flow are:

1) Nomal Pond Level; 446.00

2) Clear Water-way;

Weir	53 x 60 =	3,180 ft
Undersluices	11 x 60 =	660 ft
		3,840 ft

3) Length of Stilling Basin;

Weir	79 ft
Undersluices	88 ft

4) Head-across the Structures;

Designed	30 ft
Permissible(operative)	22 ft

5) Downstream Floor Levels;

Weir	416.0 ft
Undersluices	415.0 ft

6) Crest Levels;

Weir	428.0 ft
Undersluices	425.0 ft

7) Downstream(Tail) Water Levels;

The designed minimum tail water levels, the designed minimum retrogression levels and minimum actual levels taken from Minimum Envelope Line for 1997 appear as the table presented previous para. It is clear from the above mentioned table that the barrage structure is experiencing a severe retrogression of downstream levels of the order of 8.6 ft to 9.8 ft with reference to the Designed Tail water rating levels and 6.15 ft to 7.40 ft with reference to the Designed minimum retrogression Tail-water -levels. Keeping in view the above serious situation, analysis of the surface flow and hydraulic jump characteristics has been conducted for actual minimum Tail-water-levels recorded for the year of 1997 (the most recent).

(2) Result of Analysis

The analyzed data with relevant resultant figures for stilling basin and jump characteristics appear in Table B2.3.1. in which comparison with results of at designed and at present is made. According to the result, hydraulic jump will shift toward downstream from the position calculated by the designer as shown in Fig.B2.3.1. It is worried to harm safety of the barrage.

These table reveal and exhibit clearly the inadequacy of the stilling basin performance level and level of jump exposure which basically caused by retrogression, aggravates the retrogression phenomenon further and jump exposure results in a high percentage of hydraulic energy passing downstream of stilling basin without dissipation, causing serious problems of scour, retrogression and damaging turbulent flow.

Energy dissipation is one of the basic functions of the stilling basin at a barrage. This is normally accomplished by formation of hydraulic jump and the action of stilling basin appurtenances in conjunction with the action of the hydraulic jump. At Taunsa Barrage, energy dissipation is a serious problem. Retrogression of levels and inadequacy of stilling basin result in a pulsating or a completely washed out jump affecting the energy dissipation very adversely. For the Tail water levels of 1997, the energy dissipation is assessed at about 30 % indicating serious threat to downstream protections and consequently the barrage itself. If the retrogression will be left as it is, the situation shall worsen as explained in Fig.B2.3.2.

The stilling basin at Taunsa is shorter than required. Performance is below normal and inefficient. Stilling basin floor level is higher than required. Hydraulic jump remains exposed and pulsating throughout the flow range. Remedial measures are urgently needed to mitigate the present dangerous conditions.

B.2.4. Remedial Measures

(1) Comparison with Alternatives

The remedial measures directed towards eliminating the effects of retrogression mainly aim at improving the tail water levels. The following options can be considered:

1) Remodeling the stilling basin by introduction of devices that could raise the water levels required for improvement in energy dissipation.

2) A Subsidiary weir placed at the end of loose stone apron.

3) A pair of spurs anchored one each to right and left bank to construct the water-way.

4) A low Ogee shape weir crest on piles.

5) A rock weir constructed with stone gabions as the end of impervious floor.

Item 3) formed a part of recommendations made in the late sixties but this dose not serve the purpose because the heading up caused by the spurs will generate severe scour at the spur-head and also in the bed, and the maintenance cost will be too high. Item 4) is not recommended because the construction would pose substantial problems due to the location. And, Item 5) is not recommended also because difficulties in meintenance of the rock weir. Therefore, among above mesures, 1)remodeling the stilling basin and 2)provision of subsidiary weir, are most practicable solutions.

In view of economy and difficulties on construction, as 1) is preferable rather than another, remodeling the stilling basin is proposed in the Study. However, 2) is not thrown out of count until confirming the performance of those functions and applicabilities through model studies to be conducted in further stage. Tail scouring problem on the D/S of the proposed subsidiary weir itself caused probably instead of the same in the existing stilling basin, should be resolved.

(2) Remodeling of Stilling basin

Certain stilling basin should be created for improvement in energy dissipation. In order to form the stilling basin, some devices should be introduced that could raise the water level required.

The present stilling basin of Taunsa Barrage has some inherent structural deficiencies like the damages to skin concrete which has virtually no bond with the low strength base concrete. The impact blocks and the deflector blocks are not properly anchored and efforts to anchor them now will pay off, because the anchors embedded in low strength bas are easily pulled out due to thrust on the block.

An effort to improve the stilling basin in bay No.41,42 made in 1992 did help in improving the flow conditions for sometimes but the damage to skin-concrete and impact blocks, virtually nullified the benefits of this effort, which included re-anchoring of impact blocks, addition of chute blocks at the toe downstream glacis and conversion of last row of cubical blocks into a dentated type end sill.

It was reported that stilling basin length was reduced more than 30 % with the use of appurtenances of adequate installation of blocks. Remodeling of stilling basin is proposed to meet necessary requirement against occurred and expected retrogression. As following previous remodeling in bay No.41,42, anchored chute blocks and strengthen impact blocks will be provided. Furthermore, the remodeling of stilling basin would require major changes, like strengthening of skin concrete, provision of additional energy dissipation devices and consequently strengthening of the downstream C.C.blocks and stone apron.

(3) Design parameters of remodelling of Stilling Basin

Remodeling of the stilling basin which is proposed to comprise the following:

1) Provision of chute blocks at the toe of the downstream glacis

   Dimensions of the proposed chute blocks are determined as following a design of USBR Stilling Basin. The blocks have hight of D1, with top surface on 5 degree slope.

2) Provision of a still at the end of impervious floor

   The type is to be decided through model studies, dimensions are provisionally given having hight of 5.0', with top surface on 2.' width.

3) After removing existing skin-concrete, a skin-concrete with same thick RCC is newly provided by assuring necessary bond to the existing base concrete.

4) Proper anchoring of all the impact blocks

   Corresponding to the above improvment work of skin-concrete, existing blocks and proposed blocks to be newly provided are anchored so as to have surfficient intact with base concrete.

5) Provision of aditional length of permiable block protection

   C.C.blocks protection shall be extent with necessary length against progress of scouring , having adequate registivity and stifness by means of laying sufficient inverted filter with geo-textile.

The above proposal is exhibited in Drawings. Proposed parameters in the Rehabilitation work are shown in the following table.

Present Requisite Lengths of D/S Protection Structures

meter(feet)

	Parameters when designed	Required length	Proposed length as a rehabilitation
Weir Bay			
D/S Concrete Floor	24.08(79.00)	35.23(115.59)*	24.08(79.00)**
C.C.Blocks Apron	17.32(56.83)	17.90(58.71)	30.48 (100.00)
Sub-total		*53.13 (174.30)*	*54.56 (179.00)*
Loose Stone Apron	24.57(80.60)	23.83(78.28)	24.57(80.60)
Total length	65.97 (216.43)	76.99 (252.58)	79.13 (259.60)
Under-Sluice Bay			
D/S Concrete Floor	26.82(88.00)	36.83 (120.83)*	26.82(88.00)**
C.C.Blocks Apron	17.32(56.83)	21.09(69.19)	30.48 (100.00)
Sub-total		*57.92 (190.02)*	*57.30 (188.00)*
Loose Stone Apron	27.43(90.00)	28.12(92.26)	27.43(90.00)
Total length	71.57 (234.83)	86.04 (282.28)	84.73 (278.00)

*: calculated by 4.5 x Ef2 in the Hydraulic Jump Analysis of Table B2.3.1

**: It is proposed to follow the present parameters in consideration with introducing some devices.

B.2.5. Stability of the left wall of rafting bay / Navigation bay

The left wall of rafting bay on the downstream side is unsafe for the design head-across. It is infact so far, only because the head-across the Barrage has been restricted to 22 ft for other reasons and that river traffic intensity reduces to very nominal in low supply periods when the head-across is more.

In order to make this wall safe for the designed head-across, the thickness of the wall upto the under-sluice crest will have to be increased on the under-sluice side by about 5 ft at the base and sloping to 0 at RL 425.0. Due to rarity of traffic intensity and difficulty of ship's approach to downstream of bay by shallow depth of water in low supply periods, present restricted head-across of 22 ft may be kept on practically.

B.3. Barrage Foundation

B.3.1. Chronicle on Accidents and Repair Works

Since commencement of service of the Taunsa barrage on 1958, various accidents has been reported, especially in an early stage of service. Following list shows accidents and repair by year.

(1) Accidents

First year of operation (1958):	un-tightened anchor bolts, skin concrete of navigation bay (Bay 8) was damaged,
until the end of 1960 flood:	Bays 56 to 65, trapezoidal blocks uprooted and skin concrete rolled down as a similar type of work to that in 1962- 63 was done,
Until the end of 1961 flood:	Bays 54 to 65, D/S glacis were eroded
1959 - 1961 floods:	Bays 1, 21, 23 & 24, trapezoidal blocks damaged,
1962 flood :	Bay 49, all trapezoidal blocks uprooted and entire skin rolled down,
1962 flood:	Bay 50, three trapezoidal block uprooted and rolled down,
1962 flood:	Bay 52, skin concrete, trapezoidal blocks and cubicle blocks lifted more than half a feet,
until the end of 1962 flood:	Loosed anchor bolts fixing trapezoidal blocks to the mass concrete, as C.E. Bahawalpur inspected and instructed anchor bolts should be fixed as much as possible,
1987 flood:	Bays 41- 42, entire skin concrete rolled up in bay 41 and 50 % of skin concrete rolled up in bay 42, 15 nos. of

trapezoidal blocks and 11 nos. of friction blocks up-rooted,

As of December 1997

Cubicle blocks missing in Bay 10/2 nos., Bay 28/1 no., Bay 36/1 no., Bay 41/7 nos., Bay 47/1 no., Bay 48/4 nos., Bay 65/2 nos.,

Trapezoidal blocks missing in Bay 28/1 no., Bay 34/2 nos.,

Trapezoidal blocks severely damaged in Bay 4/1 no., Bay 5/6 nos., Bay 6/5 nos., Bay 7/2 nos., Bay 9/5 no., Bay 10/4 nos., Bay 11/7 nos., Bay 12/11 nos., Bay 13/11 no., Bay 14/9 no., Bay 15/10 nos., Bay 16/7 no., Bay 17/9 nos., Bay 18/8 nos., Bay 19/11 nos., Bay 20/10 nos., Bay 22/7 nos., Bay 50/9 nos., Bay 51/8 nos., Bay 53/13 nos., Bay 54/12 nos., Bay 55/10 nos., Bay 56/8 no., Bay 62/13 nos.

A record regarding 1962 flood damage describes the reasons and mechanism of accidents as following;

"The skin concrete and the mass concrete are not monolithic as the skin concrete was constructed after a gap of several months. The hooks provided to tie skin concrete with the mass concrete were missing in most of the bays. The trapezoidal blocks and cubical blocks are just placed on the top of the mass concrete and have not been embedded in it, though they are tied with the reinforcement of skin concrete. The construction and expansion joints are leaking badly which transmitted the sub-soil pressure to the bottom of the skin concrete. This uplift pressure and high velocity jet of water badly causes uprooting and rolling down of the trapezoidal blocks and skin concrete. Because all the trapezoidal blocks which were tied with the skin concrete and were just placed on the top of mass concrete."

(2) Repair works

In 1962 - 63:

1. Reconstructing damaged skin concrete and trapezoidal blocks of Bay No. 49 and Navigation Bay No. 8,

2. Guniting Bays No. 9 to 34 and 44 to 53,

3. Fixing trapezoidal blocks to the mass concrete by anchor bolts,

4. Reconstructing damaged blocks in Bays No. 1, 21, 23, 24 and 50,

5. Fixing skin concrete to floor by anchor bolts in Bays No. 1 to 27, 45 to 47 and 50 to 56.

In 1960-61:	Bay 56 to 65, a similar type of work to that in 1962-63 , including grouting and anchoring in these bays,
In 1961-62:	guniting on the eroded D/S glacis in bays 54 to 65
In 1961 - 62:	anchoring and grouting in bay 56 to 65
In 1987 - 88:	Bays 41 & 42, reconstruction of skin concrete, 15 nos. of trapezoidal blocks and 11 nos. of friction blocks,

Repair method of fixing skin concrete to the floor by anchor bolts is mentioned in a paper as following;

The anchor bolts had to be fixed at the rate of 5 anchor bolts per panel of mass concrete. While fixing the position of anchor bolts, special care has been taken to fix them mostly in between the trapezoidal blocks and cubical blocks since lifting of skin concrete is experienced in this zone.

It was instructed to grout the joints by drilling holes up to 2/3 depth of mass concrete at the joints at the rate of one at each corner of the panels of mass concrete. It was later on decided to do only ....(cannot read in a paper) holes at the joints, preferably in the center of the bay and if they take more than 8 to 10 cwt. of cement then only the grouting operation be extended to other places in the bay otherwise not. By the time the revised orders were received most of the grouting work in bays 50 to 56 was either completed or nearing completion. So satisfactory grouting has been done in these bays. No extensive grouting has been done in bays 9 to 27 as time is left as attempt their grouting. In bay ....·(cannot read) and bay No. 1 and 5 have been extensively grouted but these bays had been taken much of grout. The other bays did not take much of ....(cannot read) 45 to 47 have been grouted thoroughly.

Cost for the drilling holes for grouting is different on the right and the left sides, as the bays on the right are more than the grout taken by the left. Similarly, anchor bolt on the right is more costly than on the left, as when the holes were drilled for fixing anchor bolts after grouting the bays on the right side several holes were still giving out water and grouting was carried again through these holes.

After fixing anchor bolts in all these bays, pressure release pipes at the rate of two per bay in the weir and four per bay in the under-sluice are to be fixed in the center of the panels of skin concrete lying between trapezoidal blocks and cubicle blocks where the lifting of the skin concrete has been experienced in the previous years. About 90 nos. of such pipes are to be fixed in 1963.

The pressure release pipe is 1 foot in length. The bottom 2 inches of the pipe will work as filter as this 2 inches' piece has been perforated and surrounded by fine filter mesh. A perforated brass cap has been provided at the top with fine filter mesh welded underneath these perforations. While the bottom of the release pipe will be resting on the mass concrete the top will be flushed with the floor. Threads have been provided at the top to which the brass cap has been fitted. In case any filter does not work in future, it would be possible to clean it by removing the cap and fixing piece of pipe which can be worked with hand pump.

B.3.2. Review on Floor Concrete Design

The report on the 1962 flood damage mentions that "The skin concrete and the mass concrete of the barrage are not monolithic as the skin concrete was constructed after a gap of several months. .... The construction and expansion joints are leaking badly which transmitted the sub-soil pressure to the bottom of the skin concrete."

Inspection in the bay 42 during the feasibility study has revealed serious present status of the skin concrete on the floor and up-rooted and rolled down blocks. About 40 % of skin concrete on the floor downstream of d/s glacis had torn off from the mass concrete and 2 trapezoidal blocks rolled down to the area between trapezoidal blocks and cubicle blocks. Besides, several springs were found out along the joints, especially these between d/s glacis and floor concrete.

(1) Design and construction of floor concrete

Cross sections of the barrage are shown in Fig-B3.1 for under-sluice and for weir, and plan and the type of joint are shown in Fig-B3.2 and B3.3. Two types of joint have been used for the mass concrete, one is with water-stop which is made of steel plate coated with bitumen, and another is without water-stop. The contact surfaces of some joints are coated with bitumen to stop leakage. For the skin concrete, there are two types, which are construction joints and expansion and contraction joints. Shapes of contact surfaces are shown in Fig-B3.2 and the types of joints are tabulated in table B3.1.

Notice in the drawings instructs a sequence of placing mass concrete as following; first sleepers, second compartments, and third walls as shown in Fig-B3.2. A shape of the wall portion where concrete was placed at third stage is trapezium with narrow top width. Mass concrete was a lean mixed concrete of a mix of cement : sand : gravel = 1: 4: 8 containing much sand and gravel. The sequence, the mix of concrete and the shape of wall portion indicate occurrence of unfavorable gaps in the joints of the mass concrete. Gaps also could be developed in the joints of the skin concrete.

(2) Gaps

It is supposed that any joint could not avoid making a gap because of a design and a construction method of concrete, i.e., sequence of placing, quality of concrete and a shape of the wall section. It is assumed that two different types of gaps might be formed, one is a plane-like gap and another is a pipe-like gap along the contact plane between the sleeper and the compartment concrete.

Table B3.1 Types of Joint

Direction	Section	type	Contact
Skin concrete			
u/s- d/s	A-A	C (Construction)	
u/s- d/s	B-B	C	
u/s- d/s	D-D	C	
u/s- d/s	G-G	C	
u/s- d/s	H-H	C	
u/s- d/s	K-K	C	
u/s- d/s	M-M	C	
u/s- d/s	A' -A'	E (Expansion)	Bitumen coating
u/s- d/s	B' -B'	E	Bitumen coating
u/s- d/s	K' -K'	E	Bitumen coating
u/s- d/s	M' -M'	E	Bitumen coating
transverse	A/A'	C	
transverse	C' /C' -C'	E	Bitumen coating
transverse	E/E-E	C	
transverse	F/F-F	C	
transverse	G' /G' -G'	E	Bitumen coating
transverse	J/J-J	C	
transverse	L/L-L	C	
Mass concrete			
u/s- d/s	A-A	w/o (without)	
u/s- d/s	B-B	w/o	
u/s- d/s	D-D	w/o	
u/s- d/s	G-G	w/o	
u/s- d/s	H-H	w/o	
u/s- d/s	K-K	w/o	
u/s- d/s	M-M	w/o	
u/s- d/s	A' -A'	w/ (with)	Bitumen coating
u/s- d/s	B' -B'	w/	Bitumen coating
u/s- d/s	K' -K'	w/	Bitumen coating
u/s- d/s	M' -M'	w/	Bitumen coating
transverse	A/A'	w/o	
transverse	A' /A' -A'	w/	Bitumen coating
transverse	C' /C' -C'	w/	Bitumen coating
transverse	E/E-E	C	
transverse	F/F-F	w/o	Sheet pile
transverse	G' /G' -G'	w/	Bitumen coating
transverse	J/J-J	w/o	
transverse	L/L-L	w/o	

Mechanism of developing gaps could be explained below. Wet concrete for the compartment was placed on a sand layer after placing the sleeper, so that the compartment concrete was supported directly by the sand layer and the sleepers do not work as its supporter. Then wet concrete for the wall was poured on the sleeper between the compartments, using compartment concrete for formwork.

Settlement of the sleeper must continue during hardening period of the wall concrete since the foundation ground was composed of saturated fine sand layer which could not settle immediately. This might result making a gap between the sleeper and the compartment depending on weight of wall concrete. A plane gap was able to be formed only by settlement of wall concrete. A pipe-like gap could be formed because of difficult compacting of raw concrete of wall by the shape of its section, forming honeycomb by lean mixed concrete, etc.

(3) Settlement of the sleeper

When load applied to the ground through the sleepers, the foundation ground must be compressed depending on its modulus of deformation (D). Since soil was excavated before placing sleeper concrete, the foundation ground had been loosened down to considerable depth, so that the modulus of deformation might become low before placing concrete. Foundation ground was compressed and confined by sleeper concrete and the D must increase to 100 to 200 kg per sq.cm.

Using above mentioned D, extent of settlement is computed. Stress under each sleeper is calculated as shown in table B3.2. Maximum stress was 5.66 psi (3.98 t/sq.m) on the section C' /C' -C' , between d/s glacis and d/s floor concrete.

Settlement is estimated in Table B3.3 assuming D=100 and depth of potential settlement being half of the width of the sleeper. Settlement of the sleeper or the gap between the sleeper and compartment concrete could reach 5 mm at maximum. Width of the gap at the contact surface between compartment and wall concrete estimated about 1 mm by the slope of 5:1.

Table B3.2 Stress on the Foundation Ground of the Sleepers

Section	Wall					Sleeper	Stress
	height	width		Area	Weight	width	
		top	bottom				
	(feet)	(feet)	(feet)	(sq.ft)	(lb.)	(feet)	(psi)
A' /A' -A'	2	4	4.8	8.8	1264	5.8	1.51
B-B/B'-B'	3	4	5.75	14.63	2101	6	2.43
C' /C' -C'	7.3	4	5.56	34.89	5010	6.15	5.66
F/F-F	7.88	4	7.15	43.93	6308	8.15	5.38
G-G	7	4	6.8	37.8	5427	7.8	4.83
G' /G' -G'	6.5	4	5.8	31.85	4573	7.66	4.15
H-H	6	4	6.4	31.2	4480	7.4	4.20
J/J-J	5.5	4	6	27.5	3949	7.2	3.81
K-K/K'-K'	5	4	6	25	3590	7	3.56
L/L-L	4.5	4	5.8	22.05	3166	6.8	3.23
M-M/M'-M'	4	4	5.6	19.2	2757	6.6	2.90

Table B3.3 Settlement of the Sleeper Concrete

Section	Sleeper width	Stress	Affected depth	D	Settlement
	m	t/sq.m	m	kg/sq.cm	mm
A'/A'-A'	1.768	1.064	0.884	100	0.94
B-B/B'-B'	1.829	1.709	0.914	100	1.563
C'/C'-C'	1.875	3.978	0.937	100	3.728
F/F-F	2.484	3.779	1.242	100	4.694
G-G	2.377	3.397	1.189	100	4.038
G'/G'-G'	2.335	2.915	1.167	100	3.403
H-H	2.256	2.956	1.128	100	3.333
J/J-J	2.195	2.678	1.097	100	2.938
K-K/K'-K'	2.134	2.504	1.067	100	2.671
L/L-L	2.073	2.273	1.036	100	2.356
M-M/M'-M'	2.012	2.039	1.006	100	2.051

(4) Plane gap

Judging from the computed width of settlement in the mass concrete, not only water but also sand particles could pass through the gap. Leakage through gaps is estimated for a plane gap and a pipe gap at the end of d/s glacis as following.

Elevation at the bottom of d/s glacis concrete and the outlet of leakage is RL. 406 and RL. 416, respectively, while pond level was RL. 429 in the study period. By Poiseulle's formula, average velocity in the gaps was computed as shown in table B3.4.

$$V=(g I h^2)/(3 n)$$

where, V: average velocity in cm/sec

g: gravity acceleration (=980 cm^2/sec)

I: Hydraulic gradient

2h: opening in cm

n: coefficient of kinematic viscosity (0.009 cm^2/sec at 25 °C)

Table B3.4 Velocity in the plane gap

D (kg/cm^2)	100	150	200
Settlement (mm)	4.69	3.13	2.35
Opening (mm)	0.92	0.61	0.46
Velocity (cm/s)	62.5	27.8	15.6
Q (cm^3/sec)	10350	3067	1293.7

D: Modulus of Deformation (kg/cm^2)

Q: Discharge through gap (gap is 18 m continuous)

Discharge of the springs through the gaps was not more than 200 cm^3 in the bay 42 during investigation and the spring water was carrying small quantity of sand particles. It indicates that the gaps have varying size and are tortuous from one end to another because calculated discharge is much larger than observation. The gaps might be not more than 1 mm judging from bearing capacity of the foundation ground, but it is more than 0.2 to 0.3 mm since sand particles of 0.2 mm diameter could pass through the gaps.

(5) Pipe gap

Pond level and outlet level were as same the above case. Velocity in a pipe flow can be computed by the Poiseulle' s formula when the Reynolds' number (Re) of flow is less than 2300 and when Re is more than 2300, the Colebrook' s formula was used.

A constant in a denominator is changed from 3 to 8 in Poiseulle' s formula in the case of a pipe.

$$V=(g I h^2)/(8 n)$$

Computed velocity in the gap is shown in table B3.5.

Table B3.5 Velocity in the pipe gap

Diameter (cm)	5	2	1	0.5
I	0.95	0.95	0.95	0.95
Velocity (cm/s)	226	143	101	22
Q (cm^3/sec)	4433	448	79	4.2

It is supposed the pipe shaped gaps are tortuous by their origin. Since diameter of the pipes is supposed to be narrower than the average size of the gravel in the concrete, approximately 2 cm of diameter is expected at maximum. In such case, velocity in the pipe could reach 100 cm/sec in 2 cm pipe and sand particles of average diameter of 0.2 mm could be soaked easily into the gap and they could be washed away.

B.3.3. Design Parameters

B.3.3.1. Earthquake

Although the barrage is subject to earthquake activity, the probability of an earthquake is lower in the alluvial plain along the Indus river system than in the mountainous regions. This probability is determined by the number of previous earthquakes in the region and its intensity. In Pakistan, isoseismal map has been prepared as shown in Fig.-B3.4, in which the Taunsa barrage is located in the region between intensity 6 and 7 of Modified Mercalli scale equivalent to 21 to 94 gals of seismic acceleration. It is expected that minor to moderate damage would occur depending on the foundation condition.

Since the foundation consisted of fine sand with dense or relative density being high, an earthquake could cause minor damage to the structure, however no previous earthquake has been reported at the barrage.

B.3.3.2. Wind

Wind generates waves which affect the safety of the guide banks and spur dikes upstream pond of the barrage. Freeboard is provided to prevent over-topping of the guide banks and spur dikes by wave action. The height of waves generated by winds in the reservoir depends on the wind velocity, the duration of the wind, the fetch, the depth of water, and the width of the reservoir. The height of the waves as they approach the face of the dikes may be altered by the increasing depth of the water, or by the decreasing width of the reservoir.

A summary of empirical formulas proposed for determining wave heights is given in an

American Society of Civil Engineers report, from which Table B3.6 was extracted.

"Design of Small Dams" published by USBR describes that "All conditions affecting exposure of the dam to the wind must be considered in selecting the maximum wind velocity. It is believed that no locality is safe from an occurrence of winds up to 100 mi/h at least once during a period of many years, although a particular site may be topographically sheltered so that the reservoir is protected from sustained winds of high velocity. Under these conditions, wind velocities of 75 or even 50 mi/h may be used."

Table B3.6 Wave height versus fetch and wind velocity

Fetch mi	Wind velocity mi/h	Wave height ft
1	50	2.7
1	75	3.0
2.5	50	3.2
2.5	75	3.6
2.5	100	3.9
5	50	3.7
5	75	4.3
5	100	4.8
10	50	4.5
10	75	5.4
10	100	6.1

For the design of small dams with riprapped slopes, it is recommended that the freeboard be sufficient to prevent over-topping of the dam from wave run up equal to 1.5 times the height of the wave as interpolated from the Table B3.6, measured vertically from the still water level. Normal freeboard should be based on a wind velocity of 100 mi/h, and minimum freeboard on the velocity of 50 mi/h. On the basis of these assumptions and on other considerations of the purpose of the freeboard, Table B3.7 lists the least amount recommended for both normal and minimum freeboards on riprapped earth-fill dams.

Table B3.7 Fetch versus recommended normal and minimum freeboard

Fetch mi	Normal freeboard ft	Minimum freeboard ft
<1	4	3
1	5	4
2.5	6	4
5	8	6
10	10	7

At the Taunsa barrage, freeboard of the dikes is designed about 7 feet which is sufficient to the dikes for the barrage pond.

B.3.4. Present Status of the Barrage

B.3.4.1. Geological Features of the Barrage Foundation

The Indus river has about 13 km wide high-flood channel and 2 to 3 km wide low-flood channel with its bed slope of 1/5,000 to 1/10,000 in the Taunsa barrage area. It is assumed that mean annual maximum flood is 466,000 cusecs that is regarded to form a bed slope at equilibrium at recurrence interval of 2 to 3 years. Engineering characteristics of foundation ground are indicated by a density of and an arrangement of particles in the sand layer, which are subjected to the variation of depositing condition. A river morphological process, such as chronological variation on meandering and formation of sand bars, and channel and flow condition of the Indus river suggest a depositing circumstance of fine sand at the site. Since depositing circumstances in the river have kept almost same since the beginning of the Quaternary, meandering river channels and existence of sand bars have been same on every sediment plane down to more than a few hundreds meters deep. Therefore, the foundation could be treated as uniform sand layers geologically with its changes of density varied with depth.

Subsoil conditions were explored by drilling bore holes around the barrage as shown in Fig.-B3.5. Eight bore holes were drilled by rotary wash method. Depth of each hole was 40 meters. Bearing capacity of the foundation was determined by standard penetration test (SPT) and the soil samples taken by the SPT were used for particle size distribution test.

(1) Sub-surface exploration

Standard penetration tests (SPT) were performed to obtain in-situ soil conditions and soil sampling. The SPT were performed during drilling at every 0.5 meter depth interval and soil samples recovered. The tests were performed in accordance with ASTM Standard D-1586. in this method, a standard split tube sampler was driven into ground for 0.45 meter (18 inches) with help of 140 lb. hammer, falling freely from height of 75 cm (30 inches). The first 15 cm (6 inches) is considered as preliminary penetration whereas last 30 cm (1 foot) penetration is taken as main penetration. The number of blows for every 15 cm penetration is recorded.

(2) N-values of the foundation ground

The accumulated blows for main 30 cm penetration have been recorded as N-value and given on the bore hole logs. Where 1 foot penetration was not achieved, numbers of blows for maximum penetration were noted and shown on the bore hole logs.

Measured N-value is larger than real value when measuring depth reaches deeper, because the energy given by blowing at the knocking head must be consumed by elastic deformation of longer rod and/or by plastic deformation at the joints of the rods. In this study, following correction was applied when the depth is more than 20 m.

$$N' = N \text{ if } l <= 20 \text{ m}$$

$$N' = N\,(1.06 - 0.003\,l) \quad \text{if } l > 20 \text{ m}$$

where N' : corrected N-value

N : measured N-value

l : length of the rods (m)

Moreover, measured N-value shows larger value than real one where is saturated fine sand or silty sand layer having N-value of more than 15. Terzaghi and Peck have shown the following correlation formula between measured N and corrected N.

$$N' = 15 + (N - 15)/2 \qquad \text{when } N > 15$$

where N' : corrected N-value

N : measured N- value

The measured N-values for each bore hole have been corrected by Terzaghi and Peck's formula. They are shown graphically in Fig.-B3.6. Estimated original ground surface is about EL. 130 m (RL. 428). N-values ranges between 15 and 40 in the depth between EL. 103 to 110 m (RL. 338 to 361). Below the depth between EL. 103 m (RL. 338) and EL. 110 m (RL. 361), N-values range more than 50. At an elevation of RL. 380 (EL. 116 m) to RL. 394 (EL. 120 m) equivalent to that of the bottom of the foundation wells, N-value ranges 15 to 30 and average N-value is 20, then it is categorized "dense" ground.

(3) Size distribution

Particle size distributions were obtained using the SPT soil samples. Particle size distributes almost same over whole drilling holes, 3-16 % of total particle weight is over #50 sieve (0.3 mm), 84-92 % between #100 and #50 sieve (0.15 to 0.3 mm) and 1-10 % smaller than #100 sieve (0.15 mm) as shown in Fig.-B3.7.

(4) Bearing Capacity

Safety for the ground of foundation wells is examined by ultimate equilibrium method. Ultimate bearing capacity "qd" and allowable bearing capacity "qa" are computed 165 psi and 58 psi respectively at the N-value of 20 of the ground composed of cohesionless material. On the other hand, total normal load to the ground is calculated 3,400 tons of which total weight of wells, pier and road bridge is about 3,200 tons and total weight of railway bridge, superstructure and gates is 200 tons. The area of base of the foundation wells is 158 sq.m. Then normal stress "p" to the ground is calculated about 31 psi. Foundation ground has sufficient bearing capacity as the normal stress on the base "p" is 31 psi against the allowable bearing capacity "qa" of 58 psi.

If bearing capacity is insufficient to support the structure, the barrage must be accompanied by excessive settlement and/or uneven settlement in its various parts causing the barrage malfunction. Actual structure, however, shows little sign of excess or uneven settlement. Judging from the bearing capacity and actual status of the barrage, the foundation ground is safe in terms of the bearing capacity at present.

(5) Permeability

Foundation materials are composed of fine sand of their size distributing mostly finer than 1 mm as shown in Fig.-B3.6. Permeability of the foundation material is estimated by various empirical formulae, such as Hazen's, Terzaghi's, Zunker's, Kozeny's and Creager's formulae. Estimated permeability ranges between (1 to 2) x 10^{-2} cm/sec except by Creager's 6 x 10^{-2} cm/sec.

B.3.4.2. Problems in Bay 42 and the Left Abutment

(1) Bay 42

In the study period, inspection was performed in bay 42. Deposited sand of about 10 feet removed. In bay 42, two trapezoidal blocks washed out from central part and skin concrete was torn off also in the central part. Most of trapezoidal blocks remained were moved except near both ends. Washed out blocks were founded just upstream of the cubicle blocks.

Springs were also found on the floor which were located on the joints mostly. Their discharge was not more than 200 cm^3/sec and the spring water was carrying small quantity of sand particles.

In photo-B3.1 shows the status bay 42.

(2) Left abutment

There are depressions and cracks in the pavement in the parking area on the left bank of the Taunsa barrage. There are also collapsed concrete blocks in the transition wall downstream of the under-sluice. The locations are shown in Fig-B3.8.

Plan of the left bank and typical cross section at the retaining wall are shown in Fig-B3.9. Around the depression No. 1, the base of the concrete retaining wall is 50 feet deep from the concrete pavement and 20 feet deep steel sheet piles are placed under the base. Present ground surface around the barrage is RL. 457. Original ground elevation is supposed to be at about RL. 438 to RL. 440. Foundation of the retaining wall is placed at RL. 407.5 by excavating about 30 feet of original sand deposit. After completion of the sheet piling and concrete works, sand was filled behind the retaining walls as back-filling from RL. 407.5 up to RL. 457.

Generally, back-filled material settles depending on its compactness and has quite different soil structure from that of deposited in water. Pure sand layers of the barrage foundation deposited in water are more resistible to piping than artificial back-filled layers.

Settlement of back-fill material initiated development of the depressions. Scenario of developing depressions is following. At first, in the section of the transition, back-filled material including sand and gravel behind the concrete blocks settled since compacting must be difficult, then contact surfaces between the concrete blocks were opened at subsided concrete blocks. The wave action or fluctuation of the water level sucked material behind the blocks out through the openings which accelerated subsiding concrete blocks and developed concentration of sub-surface flow to the openings. Finally undermining or backward piping developed and back-filled material carried out through the channels. Since back-filled sand was cohisionless material which could not keep the channels in the ground, sand particles above the channels fell down and washed away then sand layer above the channels loosened. At last loosened zone extended to the ground surface, then the depression has appeared.

In 1983, it was found similar depression in the parking area and this depression was filled with sand. Present depressions have occurred again at the same location.

B.3.4.3. Survey by Ground Penetrating Radar (GPR)

(1) What is Ground Penetrating Radar

Ground penetrating radar (GPR) was used to explore the foundation condition of the barrage, especially to assess developing undermining. Common way of operating a GPR is in reflection mode. Reflection from geological boundaries or objects is created if the wavelength of the transmitted wave is sufficiently short. This means decimeters to meter size. Converted to frequency it ranges about 10 to 1000 MHz. In this way, radar can give us images of the medium. Also, using waves makes it possible to distinguish and resolve sufficiently small objects like pipes. The images, however, are not geometrically correct in their raw shape and computer software makes it possible to solve.

The resolution is determined by selecting the frequency of the antennas. Higher frequency gives higher resolution, but we loose on the penetration depth. The penetration depth depends on the physical properties of the geological medium. Approximate penetration depth by the antenna frequency is shown in Table B3.8.

Table B3.8 Penetration Depth and Antenna Frequency of GPR

Antenna Frequency(MHz)	Approximate Depth (m)
50	20
100	10
200	7
400	3

Most important issue what we want to know is the boundary between concrete and sand or caves or piping holes in the sand, but the location is saturated and rather deep so that the site very difficult condition to get correct information by the GPR.

Foundation ground of the barrage is fully saturated in its nature, but electromagnetic signal, which is used in GPR, is attenuated in saturated media much more than in unsaturated one. Skin concrete placed on the mass concrete is reinforced concrete, and the steel bars in it attenuate the signal extremely. Those had made it difficult to examine the foundation condition of the barrage.

The GPR used in the study has two antennas, one is for transmitter and the other is for receiver. Transmitted signal passes various routes to receiver as shown in Fig.-B3.10. The shortest route is straight from transmitter to receiver, then the signal is received first and strongest. Since the signal reflects at any boundary of different density, the signal reflects from the concrete walls or boundaries in the ground as it transmitted faster. Printed image of the GPR such as shown in Fig.-B3.11 includes somewhat meaning-less noise, therefore if the noise is stronger than reflected signal, it is impossible to distinguish the reflection from the boundary in the ground.

(2) Survey in Bay 42

After removal of deposited sand in the bay 42, GPR survey was tried on the concrete floor using frequency of 400 MHz. A part of floor concrete was torn off and mass concrete was revealed. Two times of survey had done along the lines shown in Fig.-B3.10.

The images on the prof-2 are shown in Fig.-B3.11. The upper dark colored lines such as black, dark blue and brown lines show the surface of concrete. The lowered dark colored line in right side shows depression where the skin concrete tore off. It shows the signal penetrated down to about 1 m deep in Fig.-B3.11 (a). Received echo is very weak from the targets at deeper than 1 m and it is impossible to know the condition below the mass concrete. Fig.-B3.11 (b) shows the image with AGS filter which increases gain of the echoes from the targets. It shows that wave reached down to 2 m deep but too weak to distinguish the condition in the lower part. The dark parallel horizontal lines in the lower part equivalent to about 3.5 m deep show echoes from the concrete ceiling, the slab and beams of the bridge.

Fig.-B3.12 shows the images on prof-3. In the right half of the image, there is depression similar to that on the prof-2.

(3) Survey in Bay 44

GPR survey had made using 100 MHz frequency on the deposited sand. In this paper, only the data of prof-23 is discussed. The survey line is 3.5 m apart from the left pier shown in Fig.-B3.13, starting at 4 m downstream (d/s) from the gate toward up to 66 m downstream.

The section along the line is in Fig.-B3.14. The friction and resistance blocks on the skin concrete are found clearly and the surface of the c.c. blocks downstream looks flat. Local scouring might be developed downstream of loose stone reaching about 1.5 m deep from the surface of the c.c. blocks. The upper layer down to 3 m deep from the surface is consisted of sand. There are several uneven lines in it which show different sand deposition, means different water content or different density.

Oblique lines right-side down are echoes from the gate, piers and bridge. It is analyzed with the help of Fig.-B3.15, black and white image and Fig.-B3.16, stressed image using AGS filter.

B.3.4.4. Pressure Pipes

Uplift pressure under the barrage could be evaluated by reading of the pressure pipes which are arranged as shown in Fig.-B3.17 in each pier.

Since commencement of the barrage operation, readings of the pressure pipes have been taken every fortnight. Original design distributed 579 numbers of pressure pipes in the barrage, but actual number was 540 as of 1958. Pressure pipes in order were 155 in 1976, because grouting in foundation ground choked them. Until August 1997, readings have been recorded for 114 pressure pipe shown in Table 3.9.

Falling head test had performed in the study to confirm the reliability of pressure pipe readings. Then choked pipes were washed for re-activation.

Procedure of the falling head test is as following; getting an initial depth of water surface from the mouth of a pressure pipe, after that filling water up to the mouth of the pipe, then height of the water surface in the pipe was read at regular intervals until the water surface reached down to the original height. Method for re-activation of choked pipes was (1) five-meter long plastic pipe was connected to the mouth of the pressure pipe, (2) water poured into the plastic pipe applying additional pressure of about 5 to 10 m water height, (3) repeat several times of applying pressure.

Results of the falling head test are shown in Fig.-B3.18 that shows the head in the pressure pipe versus time. The results could be categorized into 4 groups: (1) showing incorrect pressure: head in pipe reaches to the initial reading in a short time, within 20 minutes, (2) showing correct pressure: head in pipe reaches to the initial reading within several hours, (3) showing incorrect pressure: head in pipe reaches to the initial reading in more than several hours, (4) choked: head in pipe never reaches to the initial reading. Pipes no. 8 in Bay 23, no. 8 in Bay 40, no. 8 in Bay 51, and no. 9 in Bay 51 fallen in category (1) are too fast to drop the head and they are supposed to be leaking through coming-out of joints of the pipes or holes in the decayed pipe. Pipes no. 9 in Bay 14, no. 8 in Bay 22, no. 8A in Bay 40, no. 9 in Bay 40, no. 8A in Bay 59, no. 4 in Bay 61, and no. 5 in Bay 61 are categorized in group (2) and working well. Pipes no. 8A in Bay 23, no. 8 in Bay 23, no. 2, no. 6 and no. 7 in Bay 61 are categorized in group (3) and indicated pressure might not be correct. Pipe no. 9 in Bay 22, no. 8A in Bay 51, no. 12 in Bay 51, no. 8 in Bay 61, and no. 9 in Bay 61 are categorized to group (4) and those are choked. Performance of other pipes is not clear because of short measuring duration.

Present status of the pressure pipes is shown in table-3.10, which is confirmed by falling head test. At present, 18 % of the readable pressure pipes shows accurate values of the pressure under the barrage. Judging from this percentage, 21 numbers of pressure pipes are in order.

Table B3.9 Status of Pressure Pipes as of 1997

Bay no.	nos. of pipes in order	quantity of pipes readable	quantity of pipes initial	Bay no.	nos. of pipes in order	quantity of pipes readable	quantity of pipes initial
1	3, 22	2	10	35	8, 8A	2	7
2	na	0	8	36	na	0	7
3	na	0	8	37	8, 8A	2	7
4	2, 14	2	1	38	8, 8A, 9	3	7
5	8	1	7	39	na	0	7
6	na	0	8	40	8, 8A, 9	3	79
7&8	1, 4, 5, 6, 9	5	1	41	8, 8A	2	7
9	5, 9	2	1	42	8A	1	7
10	8, 9, 12	3	6	43	9	1	77
11	9	1	7	44	6, 7, 10, 14, 16	5	196
12	na	0	7				
13	8, 12	2	7	45	8, 9, 13	3	7
14	8	1	7	46	8, 9	2	7
15	9	1	7	47	na	0	7
16	9	1	7	48	8, 8A, 9	3	7
17	na	0	7	49	8, 8A, 9	3	7
18	8, 9	2	1	50	4, 8, 8A, 9, 11, 15, 17	7	14
19	8	1	7				
20	8A	1	7	51	8, 8A, 9	3	74
21	na	0	7	52	8, 8A, 9	3	7
22	8, 8A, 9	3	7	53	8	1	7
23	8, 8A, 9	3	7	54	na	0	7
24	8, 8A, 9	3	7	55	na	0	7
25	8, 8A	2	7	56	8	1	7
26	4, 6, 7, 8A	4	1	57	8, 9	2	6
27	na	0	7	58	9	1	7
28	na	0	7	59	8, 8A, 9	3	79
29	na	0	7	60	8, 8A, 9	3	7
30	10	1	7	61	2, 4, 6, 7, 8	5	12
31	12	1	7	62	1, 6, 9, 13, 14, 17	6	16
32	12A	1	7	63	na	0	7
33	na	0	7	64	8	1	7
34	17	1	7	65	7	1	10
				total			114

Table B3.10 Summary of Pressure Test

Bay No.	Pipe No.	Status	Re-activated	Bay No.	Pipe No.	Status	Re-activated
7 & 8	1	choked	no.	40	8A	choked	no.
7 & 8	4	choked	no.	40	9	choked	no.
7 & 8	5	choked	no.	43	8	choked	no.
7 & 8	6	choked	no.	43	8A	choked	-
7 & 8	9	choked	no.	43	9	choked	no.
14	8	choked	-	51	8	**working**	-
14	9	**working**	-	51	8A	choked	no.
18	8	choked	-	51	9	**working**	-
18	8A	choked	-	51	12	choked	no.
18	9	choked	-	59	8	choked	-
22	8	**working**	-	59	8A	**working**	-
22	8A	choked	-	59	9	choked	-
22	9	choked	no.	61	2	choked	no.
23	8	**working**	-	61	4	**working**	-
23	8A	choked	no.	61	5	choked	no.
23	9	choked	no.	61	6	choked	no.
26	8	choked	-	61	7	choked	no.
26	8A	choked	-	61	8	choked	no.
26	9	choked	-	61	9	choked	no.
30	8	choked	-	63	8	choked	-
30	9	choked	-	63	8A	choked	-
40	8	**working**	no.	63	9	choked	-

B.3.4.5. Problems in the Barrage Foundation

(1) Foundation of the Barrage

Foundation structures adopted in the Taunsa barrage are well foundation for supporting piers and retaining walls of left and right banks and steel sheet piles for controlling seepage or sub-surface flow under weir and under-sluice and under the retaining walls and transitions of both banks. Five wells are provided for pier foundation, which are 13.5 feet (4.1 m) to 16 feet (4.9 m) long, 20 feet (6.1 m) wide and 18.5 feet (5.7 m) to 25 feet (7.6 m) deep. Their bottom elevations are RL. 384 to 390 (EL. 117 to 119 m) in under-sluice sections and RL. 387 to 394 (EL. 118 to 120 m) in weir sections, while sheet piles are provided 3 lines under the floor concrete using 15 feet (4.1 m) long sheet pile for upstream and central lines and 20 feet (6.1 m) for downstream line. Elevations at the bottom of the sheet piles are RL. 402 (EL. 122.5 m) for upstream line and RL. 394 (EL. 120 m) for central and downstream lines in under-sluice sections and RL. 405 (EL. 123.4 m) for upstream line and RL. 395 (EL. 120.4 m) for central and downstream lines in weir sections. Sheet piles in both banks reach RL. 390 (EL. 119 m) in upstream of the gates and RL. 387 (EL. 118 m) in downstream of gates using 20 feet (6.1 m) pile and bottom elevations are deeper than those of weir and under-sluice sections. Thickness of concrete is 3 feet (0.9 m) in upstream flat floor, 5 feet to 9.75 feet depending on uplift in downstream floor and in glacis and gate section it is maximum about 10 feet (3 m).

(2) Factors affecting Barrage Safety

Safety of the foundation could be examined by following factors;

i) bearing capacity,

ii) piping / undermining, and

iii) uplift.

1) Bearing capacity

Safety of foundation ground for wells has examined by ultimate equilibrium method. Ultimate bearing capacity "q_d" of the ground composed of cohesionless material is computed 165 psi as N-value of 20 in the foundation ground and allowable bearing capacity "q_a" is estimated 58 psi. On the other hand, calculated total normal load to the ground is 3,400 tons of which weight of wells, pier and road bridge is about 3,200 tons and weight of railway bridge, superstructure and gates is 200 tons. The area of the foundation well base is 158 sq.m. Then normal stress "p" to the ground is about 31 psi. The foundation ground has judged having sufficient bearing capacity "qa" of 58 psi against active normal stress "p" of 31 psi.

2) Piping/Undermining

Terzaghi showed that piping failure might occur in two different manners as follows;

(A) By seepage pressure - in accordance with the equation of potential flow, exit-gradient criterion, and all the relevant theory.

(B) By "backward erosion" from springs - a slow process, which may take years to develop and is unpredictable.

The first (A) type failure occurs as soon as the critical head is applied. The behavior of pervious granular material conforms to this case. On the other hand, the critical head for the second type of failure cannot be predicted because this type (B) failure depends on specific geological details, such as the position, sequence and continuity of the individual layers of the soil in the foundation. Unpredictability of this type of failure is due to the fact that average size of particles and structure of sedimentation may vary within extremely wide limits.

The time factor introduces other different characteristics between the two types of failure, for experiments show that the first category failure occurs soon after destruction of the internal equilibrium of the soil particles, while the second type failure does take many years until occurrence of the failure by a subsurface erosion because of its very slow process. The second type failure develops backwards from the downstream end to the upstream end in the deeper subsurface layers of the granular soil formation. The soil is removed from beneath the barrage along one or several tortuous tunnels until a continuous channel is formed and finally water rushes through it carrying the bulk of the granular foundation.

Safety to piping failure was examined by applying Lane' s formula in the study. Lane, on the basis of his analysis carried out on about 200 dams over the world, concluded that the horizontal creep is less effective in causing loss of head than the vertical creep. For the Taunsa barrage, Lane' s creep coefficient ' C' could be recommended 7.0 for its fine sand foundation.

Safety to piping failure has evaluated comparing present creep coefficient calculated by Lane' s formula with recommended values of Lane' s creep coefficient. As shown in table B3.11, in both cases, under the weir and under-sluice sections of the barrage, values of creep coefficient at present are large enough to be safe to piping failure.

Table B3.11 Bligh's Coeffcient and Recommended Head Loss

	Under sluice	Weir
Lane' s creep length: Ll	195.3	192.3
Total head at preent: H	31	30
Lane' s creep coefficient : C	7.7	7.8
Recommended C for fine sand	7.0	7.0
Safety to piping	Safe	Safe

3) Springs

Springs were found on the floor in bay 42 during inspection of the barrage. Though it is necessary to escape from developing springs on the concrete floor, construction and expansion joints could hardly have avoided developing an unfavorable gap in the skin concrete and the mass concrete by its design and the work procedures. Since the mass concrete confines pressurized water in the foundation ground, the water tends to pass through gaps in the joints making springs on the floor surface. The spring or leakage through the joints, however, releases uplift through gaps. Thus the springs might keep the barrage be safe as regard uplift on one hand, but on the other hand it induces the barrage dangerous in terms of progression of undermining.

Here, it is examined whether the gaps along the joints could cause for occurring backward erosion. Since leaked water is filtered during traveling through the sand and it contains no particles finer than silt when it reached at the entrance of the gap, occurrence of piping can be observed whether the spring carries sand particles out. Piping occurs if sand particles were washed away by leaking water when the moving force acting on the sand particle exceeds the resisting force of it. Here, it is examined whether piping occurs under the barrage by the springs.

4) Movement of sand particle

When the force acting on the sand particles exceeds the critical force, sand could move. Here, moving force or driving velocity is checked by different assumption; one is sand particle of specified diameter moves when flow velocity exceeds fall velocity of sand particle in a quiescent column of water, and another is sand particle moves when the force by seepage flow acting the particle exceeds critical velocity in a soil using Justin' s formula.

For each case, velocity is computed by Rubey' s, Stokes' and Justin' s formulae for a sand particle having different diameter. Judging from critical velocity to move sand particles in Table B3.12, a sand particle of 0.2 mm diameter, equivalent to average sand diameter of D_{50} at the site, can be transported when flow velocity exceeds 2.7 cm/sec. Critical velocity for a particle of 0.1 mm diameter, equivalent to D_{10} of the foundation sand, is expected 0.9 cm /sec, that is adopted as a critical velocity for the foundation material because extraction of a fine sand particle from the sand layer might cause deprivation of internal equilibrium of the layer in the foundation.

Table B3.12 Critical velocity driving sand particles

Diameter (cm)	Velocity (cm/sec) by Rubey' s	by Stokes'	by Justin' s
0.01	0.92	1.00	3.28
0.02	2.68	4.01	4.64
0.03	4.17	9.01	5.69

Velocity near an entrance of the gap in the sand layer reduces by distance from the entrance, which is computed supposing an area of flow section is in inverse proportion to the distance from the entrance of the gap. Two typical types of gap were supposed for the calculation, they are a plane-like gap and a pipe-like gap. For each case, velocity and discharge are computed by size of the gaps.

For a plane gap, velocity near the entrance is calculated in Table B3.13 supposing the area of flow section being 1/4 of circumference of the cylinder, of which radius is distance from the entrance, since the entrance of the gap is located at the joint of compartment and sleeper.

If the gap is pipe-like, the pipe might be tortuous and its diameter must vary with the location. In this case, an area of flow section is supposed 1/4 of a surface of sphere having its center at the entrance, and very high roughness coefficient adapted to the tortuous pipe. Calculation results that velocity is much faster than that in a plane gap, then the distance where sand particles can be sucked out is enlarged. Following Table B3.14 is a calculation of this case.

Table B3.13 Velocity in the sand by distance from the entrance in plane gap

Distance from the entrance	Gap width (mm) 0.92	0.61	0.46
(cm)	Velocity (cm/sec)		
1	8.55	2.53	1.07
2	2.14	0.63	0.27
3	0.95	0.28	0.12
4	0.53	0.16	0.07
5	0.34	0.10	0.04
10	0.09	0.03	0.01

Table B3.14 Velocity in the sand by distance from the entrance in pipe gap

Distance from the entrance	Pipe diameter (cm) 5.0	2.0	1.0	0.5
(cm)	Velocity (cm/sec)			
1	1411.26	142.70	25.18	1.35
2	352.81	35.67	6.29	0.34
3	156.81	15.86	2.80	0.15
4	88.20	8.92	1.57	0.08
5	56.45	5.71	1.01	0.05
6	39.20	3.96	0.70	0.04
7	28.80	2.91	0.51	0.03
8	22.05	2.23	0.39	0.02
9	17.42	1.76	0.31	0.02
10	14.11	1.43	0.25	0.01
15	6.27	0.63	0.11	0.01
20	3.53	0.36	0.06	0.00
30	1.57	0.16	0.03	0.00

In the case of a plane gap, velocity is 0.95 cm/sec at 3 cm apart from the entrance if the width of a gap is 0.9 mm, and it is 1.1 cm/sec at 1 cm from the entrance when the width of a gap is 0.46 mm. Sand particles of diameter D_{10} could move within 3 cm from the entrance of the gap when the width of gap is 0.9 mm.

In the case of a pipe-like gap, velocity is 1.6 cm at 30 cm apart from the entrance if the diameter of the pipe is 5 cm. If the diameter is 2 cm, flow velocity is 1.4 cm/sec at 10 cm apart from the entrance, and it is 1 cm/sec at 5 cm apart from the entrance when the diameter of the pipe is 1 cm. Five centimeters (2 inches) diameter of pipe, however, is difficult to be formed in joints considering the concrete placing process, but it could be made a pipe-like gap with its diameter of 1 or 2 cm along the contact surface of the joint. Therefore sand particles of diameter D_{10} in the area within 10 cm from the entrance of the gap might be sucked out when diameter of pipe gap is 2 cm.

By above examination, it is supposed sand particles might move into the gaps. Then velocity in the gap determines whether the sand particles can be washed away from foundation ground to surface of the floor concrete through the gap. The table below shows velocity in the gaps, in which velocity in the pipe gap is high enough to carry sand particles out through the gap.

Table B3.15 Velocity in the gap

Plane gap				
Width (mm)	0.92	0.61		0.46
Velocity (cm/s)	62.5	27.8		15.6
Q (cm^3/sec/m)	575	170		72
Pipe gap				
Diameter (cm)	5	2	1	0.5
Velocity (cm/s)	225.8	142.7	100.7	21.6
Q (cm^3/sec)	4433.6	448.3	79.1	4.2

It is supposed that pipe-like gaps of 2 cm diameter could be formed in the sand layer under the mass concrete. It is possible that there could be formed considerable wide loosened sand layer or opening between the concrete and the surface of sand layer under the mass concrete because fine sand in a flowing water moves easily and becomes stable making very gentle slope in water.

5) Exit Gradient and Scour Depth

Exit gradient is computed by Khosla' s formula and shown in table B3.16. Under existing condition, exit gradient is smaller than recommended value for fine sand by Khosla which is between 0.14 and 0.17. Depth of sheet pile required by Khosla' s formula is 4 to 15 feet longer than existing one. Extension of the sheet piles, however, results increase of uplift under downstream floor concrete.

Table B3.16 Exit Gradient

Under sluice				Weir		
	Existing	Recommended		Existing	Recommended	
Pond level (RL feet)	446			446		
Floor level (RL feet)	415			416		
Bottom of pile (RL feet)	394	389	379	395	391	381
Exit Gradient	0.189	0.168	0.140	0.185	0.168	0.139
Depth of pile (feet)	21	26	36	21	25	35

Scour depth is computed by Lacey' s formula shown below.

$$R = 1.35 * (q^2/f)^{1/3}$$

$$f = 1.76 * \mathrm{sqrt}(d)$$

where q: discharge per unit width (cumecs)

f: silt factor

d: diameter of sand particle in mm

Average diameter of sand is 0.2 mm, then the silt factor 'f' is 0.79. Discharge of the under-sluice and the weir are 29.7 cumecs and 23.2 cumecs, respectively. Computation shows that the length of downstream sheet pile is short compared to computed scour depth as shown in table B3.17.

Table B3.17 Lacey' s scour depth

	Under-sluice	Weir
Discharge (cumecs)	29.73	23.23
Silt factor	0.79	0.79
Scour depth (m)	14.0	11.9
Bottom of sheet pile (m)	114.3	117.5
Bottom of sheet pile RL.	375	385
Existing bottom of sheet pile	394	395

6) Uplift

Applying Khosla' s formula, percentage of head above the downstream water surface is computed as shown in table B3.18 for each point at the cutoff and sheet piles and table B3.19 shows pressure at each point. Unfortunately it cannot compare with actual pressure under the barrage as most of the pressure pipes are not working at present.

Table B3.18 Pressure at key points (in percentage)

	Upstream cutoff	Pile 1	Pile 2	Pile 3
Under sluice				
Upstream side	100.00	63.91	46.53	23.33
At bottom	87.13	60.38	41.89	18.35
Downstream side	84.03	58.21	39.26	0.00
Weir				
Upstream side	100.00	63.24	44.84	23.36
At bottom	87.00	59.93	39.94	18.53
Downstream side	83.81	57.98	37.04	0.00

Table B3.19 Pressure at key points

U/S/cutoff		Pile 1			Pile 2			Pile 3				
E1	D1	C1	E2	D2	C2	E3	D3	C3	E4	D4	C4	
Under sluice												
Per cent	100	87.1	84.0	63.9	60.4	58.2	46.5	41.9	39.3	23.3	18.4	0
Head (ft)	30	26.1	25.2	19.2	18.1	17.5	14.0	12.6	11.8	7.0	5.5	0
Water level (ft)	446	442.1	441.2	435.2	434.1	433.5	430	428.6	427.8	423.0	421.5	416
Weir												
Per cent	100	87.0	83.8	63.2	59.9	58.0	44.8	39.9	37.0	23.4	18.5	0
Head (ft)	30	26.1	25.1	19.0	18.0	17.4	13.5	12.0	11.1	7.0	5.6	0
Water level (ft)	446	442.1	441.1	435.0	434.0	433.4	429.5	428.0	427.1	423.0	421.6	416

(3) Present Status

Regarding to bearing capacity of the barrage, no major problem has occurred at the barrage since commencement of its operation.

Regarding to development of piping under the barrage, there are several signs indicating possible piping failure of the barrage. Inspection for the bay 42 of the barrage revealed the springs on the floor concrete which could create piping under the mass concrete. Facts around the Taunsa barrage, such as progressing piping on the left bank and failure of the silt ejector in

D.G.K. canal in 1996, indicate development of piping in a similar ground condition of the barrage. Addition to this, examination on the Bligh' s creep ratio and length of sheet piles against exit gradient and scouring depth suggests a possible undermining. It shows that barrage foundation might be critical condition if sucking out of sand grains from the foundation would not stop. Another possible cause of piping is development of scoured pits or depressions downstream on a river bed. If there exist scoured pits downstream, it may suggest a possibility of developing backward erosion from the end of the concrete floor.

Regarding to uplift, measuring pressure under the concrete floor is most important to check distribution of uplift to keep the barrage safely, but it is not possible at present since pressure pipes are mostly out of order.

B.3.5. Rehabilitation and Improvement Plan

B.3.5.1. Drains for leakage

Existence of the springs has been reported since beginning of the service of the barrage. During inspection of the barrage, springs were found on the floor in bay 42. Since the springs are originated leakage passed through the joints and cracks in the floor concrete, they would affect adversely to the safety of the barrage in view of development of piping and excess pressure acting to the skin concrete. Moreover, it is supposed that springs might be developed also in other bays. The springs, however, release excess uplift pressure acting to the skin concrete. Existing springs, on the other hand, make the barrage be safe in view of safety from uplift.

In 1963, it was proposed to place the pressure release pipes in the bay. The report says as following; "After fixing anchor bolts, ... pressure release pipes at the rate of two per bay in the weir and four per bay in the under-sluice are to be fixed in the center of the panels of skin concrete lying between trapezoidal blocks and cubicle blocks where the lifting of the skin concrete has been experienced in the previous years. About 90 nos. of such pipes are to be fixed in 1963. .... The pressure release pipe is 1 foot in length. The bottom 2 inches of the pipe will work as filter as this 2 inches piece has been perforated and surrounded by fine filter mesh. .... "

The above idea releasing excess pressure safely could be applied to treatment of the springs. Suitable drains with filter must be provided on the joints of the mass concrete for effective collecting and safe releasing of leaked water. Since leaked water is free from silt or clay, filter would not be clogged with fine material and would be permeable for long time. Filter must be placed on the joints of the mass concrete to catch sand particles traveling with water through the gaps. The filter will work for self-healing material filling gaps and voids. Sand particles carried with water caught by the filter will fill gaps, and since water will be able to pass the gaps filled with sand, sand particles will be carried more with water. Then they will fill the voids near the entrance under mass concrete.

Total discharge of leakage was observed not more than 0.2 liters/sec in bay 42 and calculated amount of leakage was 0.45 to 0.6 liters/sec from 2 cm diameter gap which can be existed. The drain must have drainage capacity of half litter per second (0.5 lit./sec) for one line of joint in the skin concrete.

Grouting is also recommended for a bay where leakage is more than half litter per second (0.5 lit./sec) from one section of joint, that is a line of joint from a point of intersection to another point of intersection. Because this amount of leakage per unit section indicates that there might be formed a large gap which would be able to develop considerable piping that is harmful to the barrage safety.

The work for placing filter and drains could be performed during the skin concrete rehabilitation, since the skin concrete and blocks in the downstream floor will be repaired in the projected rehabilitation work.

B.3.5.2. Placing Geotextile Filter under the Bed Protection Concrete Blocks

In the study, it is known that the sheet piles in a downstream-most line have insufficient length for requirement in view of scour and exit gradient. Besides, river bed condition below the barrage, such as deformation of the downstream concrete blocks and launching stones, has confirmed only at designated points, though degradation of the downstream river bed tends to develop local scouring which accelerates piping in foundation ground.

The best measure to possible piping is extension of sheet piles, but it is not realistic solution since the work would be complicated and costly, addition to the problem of increasing uplift. In this study, use of geotextile filter is recommended in place of extension of sheet piles. Geotextile filter will be placed under permeable concrete blocks for bed protection avoiding increase of uplift and catching sand particles from seepage water.

Separating sand particles from water assures safety against progress of piping. Usually gravel filter is used for filter material, but bed material having particle size distribution of 0.15 mm at 15 % pass diameter (D_{15}) and 0.3 mm at 85 % pass diameter (D_{85}) requires filter material having size distribution of 1 mm at D_{15} and 2 mm at D_{85} with coefficient of uniformity (Cu =

D_{60}/D_{10}) between 1.5 and 8. Conforming to the openings in the concrete blocks about 5 cm, three different size filter layers are necessary as shown in Fig.4.1.

In construction works, filter materials need very careful size adjustment to agree with the required size distribution for each layer. On the other hand, factory made geotextile filter is easy to handle, moreover it has uniform quality to meet the requirement. Such condition of limited construction period, use of geotextile filter is recommended instead of sand and gravel filter because of reliability, durability and easy treatment.

Geotextile filter must be laid under the concrete blocks. In the area where concrete blocks exist, concrete blocks shall be removed then geotextile filter will be placed on remained filter material.

B.3.5.3. Rehabilitation on Left and Right Abutments

Left abutment of the barrage is critical condition at present. Analysis of horizontal ground water flow in a uniform pervious field shows that flow velocity along both ends of the structure will be much faster than other parts when an impervious structure is placed in the field. Then sand particles in this place move easily if they are not confined.

Depression in the parking area on the left bank is located in such place as mentioned in Sec. B3.4.2. There was a similar depression more than 10 years ago and they filled it with sand to the ground surface in a regular maintenance work. It shows piping is continuously progressing in this place.

Proposed plan on rehabilitation is reconstruction in the place. Most important thing is to reduce possible subsidence as small as possible and to stop sucking sand grains out. It is proposed that use of geotextile filter again at the back of concrete blocks on the slope.

The work includes removal of existing concrete blocks, replacing and compacting sand layers of foundation for concrete blocks, placing geotextile filter, placing retaining wall with buttresses instead of concrete blocks. The retaining wall will be equipped with drain pipes.

B.3.5.4. Improvement of Pressure Monitoring System

Existing pressure pipes are giving limited information about uplift, though pressure distribution under the mass concrete is the only indicator to monitor safety of the barrage.

Proposed monitoring system is automatic measuring with help of computer. Electric system has rather short life, however it can get data for any point instantly. Moreover, it is easy to equip additional function, such as warning apparatus. In the system, several important points will be able to check the pressure manually for safe.

Electric system has been improved very much in this decade, but an event such as striking by lightening damages the system.

There are two typical kinds of sensor, one is pore pressure meter and another is water pressure meter. Difference between two sensors is the former covered with porous media and the latter is without it, then the former is apt to be choked by fine material and assured life is five years at the maximum. The latter has more long life, but not more than 10 years.

There are a few kinds of transforming mechanism of sensors, such as strain gauge type, differential transformer type, vibrating wire type. They are almost same reliability in accuracy and durability.

Proposed plan is to set sensors for 13 piers or at every 5 pier in the barrage. Each pier is placed 12 sensors. At each pier, a remote scanner is set to collect the data from sensors. All the remote scanners will be connected and controlled by a controller which sends data to micro computer in the monitoring room.

B.4. Canal Structure

(1) Canal System of the Taunsa Irrigation Project

The canal systems of D.G. Khan and Muzaffargarh consist of main canal off-taking behind head gate and the water flow to the command areas. The main canals divert water to the respective canals. Branch canals which branch off from the main canals function as conveyance channels up to distributories and minors in the respective irrigation divisions. These channels distribute water to the watercourses. Small scale pumping stations have been constructed along the D.G. Khan main canal and Dajal branch canal at 1 to 2 km interval to irrigate the right side bank tracts of the both canal, where gravity irrigation is inapplicable because of high configuration of the irrigable areas.

T.P. link canal constitutes the last unit of eight link canals constructed under the Indus Basin Replacement Plan. This link canal, as identified by its name, transfers the Indus water available within 12,000 cusec at Taunsa barrage into the Chenab river about 180 km upstream of Panjnad headwork, to meet historic requirements of the Panjnad and Abbasia canals off-taking from Panjnad headwork.

Canal Length

(Unit: km)

Canal	Main Canal			Secondary Canal		
	Canal	Branch	Feeder	Disty	Minor	Total
D.G. Khan	11.8	119.4	12.6	1,317.2	133.9	1,774.9
Muzaffargarh	119.4	124.2	0	789.9	14.7	1,050.7
T.P. Link	61.2	0	0	0	0	61.2
Total	192.4	243.6	12.6	2,107.1	148.6	2,886.8

Type of the canals is totally earthen open canals from the head to the tail. Some canal side walls along roads and inside villages were revetted not for lining but for protection of the roads or related structure. Because alignment of the canals is on the pervious soil layer on the Indus river alluvial plain and seepage water from the earthen canals is tremendous, the seepage water causes not only lowering irrigation efficiency but also heightening the groundwater table to create waterlogging problems. The main features of the canals at the head are as follows:

Main Features of Canals at Head

Canal	Crest level of head gate (ft)	Width of bed (ft)	Water depth (ft)	Side slope	Canal slope	Design discharge (cusec)
D.G. Khan	El. 433.0	260	12.0	1 : 2.5	1 / 10,500	8,301 (14,200)*1
Muzaffargarh	El. 433.0	200	11.0	1 : 2.5	1 / 8,000	8,285
T.P. Link	El. 433.0	266	12.2	1 : 2.5	1 / 9,000	12,000 (14,000)*2

Remarks: *1 Provision in structure for future Dajal branch extension plan.
*2 Provision in structure for future increase in capacity.

Taunsa barrage has serious problem on the bed load inflow into the canals, especially at the head of D.G. Khan canal. Accumulated sediment plugs almost two third of the canal flow section due to various reasons including shortage of the budget on O&M including the dredging of the sedimentation. The problems of excessive inflow of the bed load through the head gate will be described in ANNEX C.

Silt ejector is one of the effective devices to exclude the bed load in the main canal. Although temporary repairing works have been occasionally conducted, drastic measure of rehabilitation is essential to recover the original function from the deterioration and damage of the structure.

Adjustable proportional type regulators with standardized sluice gates were provided at the key location of the main canals to distribute water to link, branch and major distributory. The gates and related structures are considerably deteriorated and some troubles at the operation. Heavy scouring at the direct downstream of the regulator and the canal dikes are exposed to collapse. In some cases, diversion water from parent canals to minor distributories are diverted by a conduit with gate under the dike without a regulator. Moghas which have no gate are provided and in-controllable distribution water flow into watercourses depending on the parent water level.

Another sedimentation problem is caused from wash load in the canals beside bed load at the canal head. Sufficient canal velocity flow wash load down to the field without sedimentation, but low canal velocity due to smaller discharge causes another sedimentation problem of wash load, especially at the upstream of regulator. Furthermore, the low canal velocity disturbs the canal function of proportional distribution system. Accordingly, full water discharge is favorable from the view point of operation, and the practice will produce over irrigation and accelerate waterlogging problems in the CCA.

Escape canals are provided to secure canal system from accidents. In the case of D.G. Khan canal, hill torrent from the Suleman mountains sometimes attacks right side canal dike in summer, and the escape has the role to cope with the outbroken flood. Escape has also been deteriorated, and some of the gates installed the escape were heavily damaged and out of order.

(2) Issuse of D.G.Khan Canal

Siltation is a serious issue of D.G.Khan Canal. Heavy silt deposition is appeared on the D.G.Khan Canal bed by direct silt entry from right pocket of Taunsa Barrage, as described in the ANNEX C.6.

During annual closure term of the barrage, the JICA Study Team surveyed on cross sectional and longitudinal sounding about 2,300 meter length of D.G.Khan Canal from its feeding entry point. Significant dunes has developed on the surface of channel bed with an average scale of 20 meters and more width. Alternative bars can also be recognozed as shown in Fig.B4.1.

In Fig.B4.2 and Fig.B.4.3, present status of the silt deposition of the channel is presented. According to the survey result, around 2 meters channel bed lift can be identified. About 250,000 cu.m of silt deposite is left on the channel until RD.7500.

(3) Deteriorations on the Canal System

1) General Features of Canal Structure

Off-taking canals of Taunsa Barrage are earth channel having trapezoidal section. These were designed regime canal applying Lacey's Regime Theory, which indicate velocity not siltation and not scoring. However, it is not always realized regime flow depend upon hydraulic condition which is different from design assumption. Furthermore, as time passing after construction, canal condition have been changed from original conditions.

3) Problems of Canal Structures

Canal structures are deteriorated particularly of the gates, and water-logging and salinity which is caused by leakage are distinct everywhere in the irrigated area. Dajal Branch Canal have heavy leakage because they pass on the edge of hill trent which is composed of semi-pervious silt. In addition to these, hill torrent floods damage for the canal system is prominent.

Farmers are compelled to inundate their farm land in an emergency not so as to spill canal water over when delay of operation. Escape facilities are essential for the protection of canal system. It is shortage of the escape function, and even existing escape is in inadequate situation for function. As siltation is transported along canal as suspended loads, it has deposited at deficient canal section in view of hydraulics. At the canal portion of transition downstream of canal crossing structures, such deposition of suspended load is found.

During Kharif irrigation term, over-full water were observed in main canals and link canals as almost flooding. It is caused delay of taking action for indents. Prompt anticipation of actual irrigation supply needs are strongly required in consideration with long transporting time of canal to reach end.

Present condition of major canal structures of D.G.Khan Canal, Muzaffargarh Canal and T.P.Link Canal were invsetigated during the field survey. The results are described in ANNEX D

Table B1.1.1 Present Condition of Hydraulic Structure of Taunsa Barrage (1 of 4)

Bay No.	U/S Loose Stone and C.C.Block	U/S C.C.Floor	U/S Glacis	D/S Glacis	D/S C.C.Floor	Friction Blocks	D/S Loose Stone and C.C.Blocks	Remarks
Under-sluice Bay No.1 Pier No.1	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	Water spouting at the downstream of D/S L.S. apron had been reported.
Under-sluice Bay No.2 Pier No.2	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Under-sluice Bay No.3 Pier No.3	Loose stone apron was considerably swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was slightly swelled.	
Under-sluice Bay No.4 Pier No.4	Loose stone apron was considerably swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Block (1 no.) was badly dameged.	Loose stone apron was slightly waved.	
Under-sluice Bay No.5 Pier No.5	Loose stone apron was considerably swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (6 nos.) were badly dameged.	Loose stone apron was slightly swelled.	
Under-sluice Bay No.6 Pier No.6	Loose stone apron was considerably swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (5 nos.) were badly dameged.	No damages were inspected.	
Under-sluice Bay No.7 Pier No.7	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (2 nos.) were badly dameged.	No damages were inspected.	
Lock Gate Bay No.8 Guide Bund	No damages were inspected.	-	-	-	-	-	No damages were inspected.	D/S skin concrete rolled down was repaired during 1961-62.
Fish Ladder Pier No.8	No damages were inspected.	-	-	-	-	-	No damages were inspected.	Wall in U/S entry was slightly collapsed.
Weir Bay No.9 Pier No.9	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (5 nos.) were badly dameged.	No damages were inspected.	
Weir Bay No.10 Pier No.10	Loose stone apron was considerably hollowed.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (5 nos.) were badly dameged, one of	Loose stone apron was considerably swelled.	
Weir Bay No.11 Pier No.11	Loose stone apron was heavily swelled.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was severely scoured as steel bar was partly	No significant damages were inspected.	Blocks (7 nos.) were badly dameged.	Loose stone apron was slightly swelled.	
Weir Bay No.12 Pier No.12	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (11 nos.) were badly dameged.	Loose stone apron was considerably swelled.	
Weir Bay No.13 Pier No.13	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (11 nos.) were badly dameged.	No damages were inspected.	
Weir Bay No.14 Pier No.14	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (9 nos.) were badly dameged.	Loose stone apron was slightly waved.	
Weir Bay No.15 Pier No.15	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (10 nos.) were badly dameged.	Loose stone apron was slightly swelled.	
Weir Bay No.16 Pier No.16	Loose stone apron was slightly hollowed.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (7 nos.) were badly dameged.	Loose stone apron was considerably swelled.	
Weir Bay No.17 Pier No.17	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (9 nos.) were badly dameged.	Loose stone apron was considerably hollowed.	

Table B1.1.1 Present Condition of Hydraulic Structure of Taunsa Barrage (2 of 4)

	U/S Loose Stone and C.C.Block	U/S C.C.Floor	U/S Glacis	D/S Glacis	D/S C.C.Floor	Friction Blocks	D/S C.C.Blocks and Loose Stone	Remarks
Pier No.17								
Weir Bay No.18	Loose stone apron was slightly hollowed.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was siverely scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (8 nos.) were badly dameged.	Loose stone apron was slightly hollowed.	
Pier No.18								
Weir Bay No.19	Loose stone apron was considerably hollowed.	No significant damages were inspected.	No significant damages were inspected.	Skin concreto was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (11 nos.) were badly dameged.	No damages were inspected.	
Pier No.19								
Weir Bay No.20	Loose stone apron was slightly hollowed.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (10 nos.) were badly dameged.	Loose stone apron was heavily swelled.	
Pier No.20								
Weir Bay No.21	Loose stone apron was heavily hollowed.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Block (1 no.) was missing.	Loose stone apron was considerably waved.	D/S skin concrete rolled down was repaired during 1961-62.
Pier No.21								
Weir Bay No.22	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (7 nos.) were badly dameged.	Loose stone apron was considerably swelled.	
Pier No.22								
Weir Bay No.23	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably swelled.	D/S skin concrete rolled down was repaired during 1961-62.
Pier No.23								
Weir Bay No.24	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	D/S skin concrete rolled down was repaired during 1961-62.
Pier No.24								
Weir Bay No.25	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.25								
Weir Bay No.26	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was siverely scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.26								
Weir Bay No.27	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.27								
Weir Bay No.28	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Block (1 no.) was missing.	No damages were inspected.	
Pier No.28								
Weir Bay No.29	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	Skin concreto was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.29								
Weir Bay No.30	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.30								
Weir Bay No.31	Loose stone apron was considerably hollowed.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably swelled.	
Pier No.31								
Weir Bay No.32	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was heavily swelled.	
Pier No.32								
Weir Bay No.33	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was siverely scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably swelled.	
Pier No.33								
Weir Bay No.34	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Blocks (3 nos.) were missing.	Loose stone apron was slightly swelled.	
Pier No.34								

Table B1.1.1 Present Condition of Hydraulic Structure of Taunsa Barrage (3 of 4)

	U/S Loose Stone and C.C.Block	U/S C.C.Floor	U/S Glacis	D/S Glacis	D/S C.C.Floor	Friction Blocks	D/S C.C.Blocks and Loose Stone	Remarks
Pier No.34								
Weir Bay No.35	Loose stone apron was slightly hollowed.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was slightly hollowed.	
Pier No.35								
Weir Bay No.36	Loose stone apron was slightly hollowed.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	Block (1 no.) was missing.	Loose stone apron was slightly hollowed.	
Pier No.36								
Weir Bay No.37	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably hollowed.	
Pier No.37								
Weir Bay No.38	Loose stone apron was considerably hollowed.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably swelled.	
Pier No.38								
Weir Bay No.39	Loose stone apron was heavily hollowed.	No significant damages were inspected.	No significant damages were inspected.	Skin concrete was scoured as steel bar was partly unwrapped.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.39								
Weir Bay No.40	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably hollowed.	
Pier No.40								
Weir Bay No.41	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (7 nos.) were missing.	Loose stone apron was slightly hollowed.	D/S skin concrete damaged was repaired during 1987-88.
Pier No.41								
Weir Bay No.42	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (4 nos.) were missing.	No damages were inspected.	D/S skin concrete damaged was repaired during 1987-88.
Pier No.42								
Weir Bay No.43	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was slightly hollowed.	
Pier No.43								
Weir Bay No.44	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably hollowed.	
Pier No.44								
Weir Bay No.45	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.45								
Weir Bay No.46	Loose stone apron was considerably swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.46								
Weir Bay No.47	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.47								
Weir Bay No.48	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (4 nos.) were missing.	No damages were inspected.	
Pier No.48								
Weir Bay No.49	Loose stone apron was considerably swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was slightly hollowed.	D/S skin concrete rolled down was repaired during 1961-62.
Pier No.49								
Weir Bay No.50	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (9 nos.) were badly dameged.	No damages were inspected.	D/S skin concrete rolled down was repaired during 1961-62.
Pier No.50								
Weir Bay No.51	Loose stone apron was considerably swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (8 nos.) were badly dameged.	Loose stone apron was slightly swelled.	
Pier No.51								
Weir Bay No.52	Loose stone apron was slightly swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably swelled.	D/S skin concrete rolled down was repaired during 1990-91.
Pier No.52								

Table B1.1.1 Present Condition of Hydraulic Structure of Taunsa Barrage (4 of 4)

	U/S Loose Stone and C.C.Block	U/S C.C.Floor	U/S Glacis	D/S Glacis	D/S C.C.Floor	Friction Blocks	D/S C.C.Blocks and Loose Stone	Remarks
Pier No.52								
Weir Bay No.53	Loose stone apron was considerably swelled.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (13 nos.) were badly dameged.	Loose stone apron was slightly swelled.	
Pier No.53								
Weir Bay No.54	Loose stone apron was heavily hollowed.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (12 nos.) were badly dameged.	No damages were inspected.	
Pier No.54								
Weir Bay No.55	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (10 nos.) were badly dameged.	No damages were inspected.	
Pier No.55								
Weir Bay No.56	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (8 nos.) were badly dameged.	No damages were inspected.	D/S skin concrete rolled down was repaired in 1961.
Pier No.56								
Weir Bay No.57	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably swelled.	
Pier No.57								
Weir Bay No.58	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.58								
Weir Bay No.59	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.59								
Weir Bay No.60	Loose stone apron was heavily hollowed.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably swelled.	
Pier No.60								
Weir Bay No.61	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.61								
Fish Ladder	No damages were inspected.	-	-	-	-	-	No damages were inspected.	
Guide Bund								
Under-sluice Bay No.62	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (13 nos.) were badly dameged.	No damages were inspected.	
Pier No.62								
Under-sluice Bay No.63	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	Loose stone apron was considerably hollowed.	
Pier No.63								
Under-sluice Bay No.64	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	No drawbacks were found.	No damages were inspected.	
Pier No.64								
Under-sluice Bay No.65	No damages were inspected.	No significant damages were inspected.	No significant damages were inspected.	It was considerably eroded.	No significant damages were inspected.	Blocks (2 nos.) were missing.	No damages were inspected.	D/S skin concrete rolled down was repaired in 1961.

Refered sources: "Proving Books of Taunsa Barrge","Annual Headworks Reports" in several years

Table B2.1.1 Histogram of Annual Highest Floods at Taunsa Barrage

Year	Date	U/S Water Level(RL.)	D/S Water Level(RL.)	U/S Dis-charge (Cs.)	D/S Dis-charge (Cs.)	Remarks
1958	21-Jul	441.80	441.60	760,784	760,784	V.H Flood
1959	10-Jul	440.50	437.30	518,000	518,000	H.Flood
1960	21-Jul	444.00	434.30	515,687	509,309	H Flood
1961	23-Jul	443.00	433.20	476,374	476,374	M.Flood
1962	26-Jul	444.00	431.88	340,113	330,613	L.Flood
1963	18-Jul	444.00	431.45	368,631	354,813	L.Flood
1964	15-Jul	443.80	432.60	504,340	502,340	H.Flood
1965	15-Aug	443.80	432.80	438,257	438,257	M.Flood
1966	7-Jul	446.50	434.48	516,113	516,113	H.Flood
1967	31-Jul	447.00	432.70	471,136	457,055	M.Flood
1968	18-Jul	446.00	432.05	446,609	431,179	M Flood
1969	30-Jul	446.00	431.70	471,105	464,705	M.Flood
1970	8-Jul	446.00	430.00	384,614	368,304	L.Flood
1971	13-Aug	446.70	430.35	406,691	392,892	M.Flood
1972	2-Jul	445.25	430.50	399,450	371,212	L.Flood
1973	23-Jul	446.00	432.70	570,087	567,623	H.Flood
1974	31-Aug	446.25	430.50	376,941	368,941	L.Flood
1975	26-Aug	446.00	432.55	524,495	524,495	H.Flood
1976	7-Aug	445.50	433.60	677,105	675,233	V.H.Flood
1977	21-Jul	445.50	433.60	459,741	443,349	M.Flood
1978	14-Jul	446.00	431.25	508,922	508,422	H.Flood
1979	3-Aug	446.50	430.50	398,069	388,069	L.Flood
1980	13-Aug	448.80	431.85	435,133	425,235	M.Flood
1981	17-Aug	446.20	430.90	402,391	392,091	M.Flood
1982	20-Aug	447.00	432.30	391,899	375,499	M.Flood
1983	10-Aug	447.50	434.10	504,189	502,189	H.Flood
1984	20-Aug	446.80	434.50	512,194	509,694	H.Flood
1985	4-Aug	446.50	433.75	318,680	306,680	L.Flood
1986	10-Aug	447.50	435.20	512,769	505,069	H.Flood
1987	26-Aug	446.50	432.60	329,204	313,204	L.Flood
1988	21-Jul	446.50	435.40	563,416	560,916	H.Flood
1989	5-Aug	447.00	434.40	560,630	558,630	H Flood
1990	2-Aug	446.90	434.30	517,652	502,152	H.Flood
1991	18-Jul	445.00	432.00	434,147	422,947	M.Flood
1992	14-Sep	446.00	434.70	655,879	654,579	V.H.Flood
1993	28-Jul	446.50	432.50	385,302	381,302	M.Flood
1994	19-Jul	444.00	436.70	574,602	574,602	H.Flood
1995	30-Jul	446.00	434.70	611,937	611,937	H.Flood
1996	20-Aug	448.00	432.30	521,708	518,208	H.Flood

Return Period	Probable Value(Cs.)
2	466,283
3	511,460
5	561,777
10	625,002
20	685,650
30	720,538
40	745,136
50	764,151
60	779,655
100	822,977
150	857,280
200	881,589

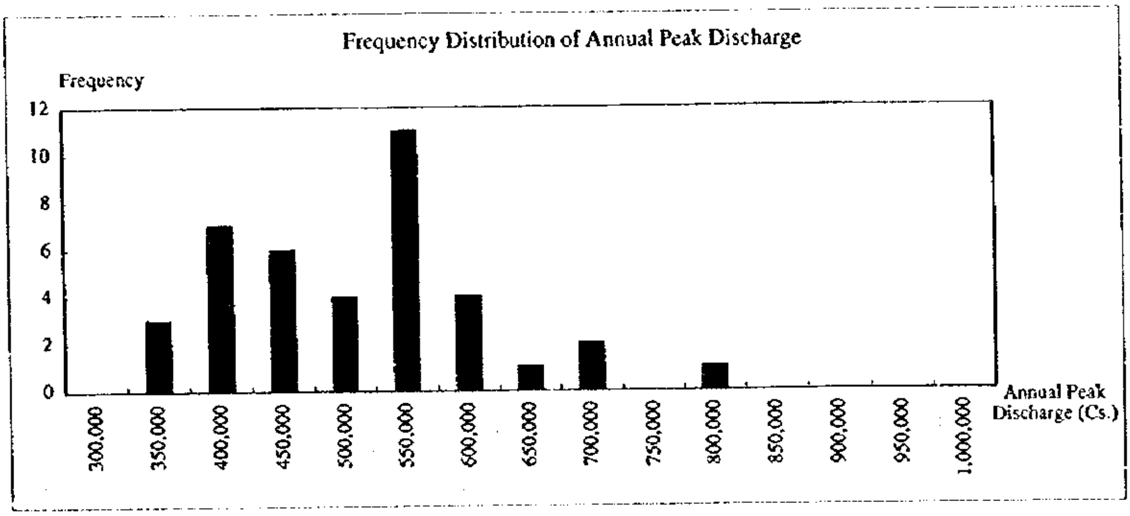

Table B2.2.1 Summary of Gauge Dischrge Relationship at Taunsa Barrage

Discharge in Cs	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975
30,000	421.90	-	418.00	419.00	420.40	419.00	419.60	419.90	420.00	420.20	421.10	419.80	420.30	420.00	420.25	420.20	420.20	420.90
40,000	423.70	420.30	419.50	420.60	421.70	420.80	420.80	422.00	421.40	421.60	422.20	421.10	421.50	421.80	421.15	421.30	421.50	422.10
50,000	424.00	421.60	420.30	421.70	422.70	421.60	421.18	423.00	422.40	422.80	423.10	422.10	422.40	422.40	422.30	422.10	422.50	422.90
80,000	427.80	424.30	423.30	424.00	424.90	423.70	423.90	424.80	424.90	424.80	425.10	424.10	424.30	424.30	424.30	424.10	424.50	424.90
100,000	429.10	425.30	424.50	425.10	425.90	424.70	424.80	425.70	425.60	425.70	426.00	425.10	425.20	425.30	425.70	425.10	425.60	425.80
200,000	433.20	429.50	428.30	425.40	429.00	428.00	427.90	428.60	428.80	428.90	428.80	428.10	427.90	427.20	428.40	429.20	428.50	428.50
300,000	435.60	431.60	430.50	430.40	430.90	429.80	429.70	430.20	430.70	430.70	430.50	429.70	429.60	429.00	429.90	430.30	430.40	430.20
400,000	437.20	433.30	432.10	431.80	431.20	431.10	431.00	431.30	432.00	432.00	431.60	431.00	430.70	430.40	431.00	430.80	431.70	431.30
500,000	438.60	434.70	433.30	432.90	432.20	432.10	432.00	432.20	433.00	433.00	432.50	432.00	431.60	431.30	432.00	432.30	432.70	432.20
600,000	439.60	435.70	434.30	433.70	433.00	433.00	432.80	433.00	433.80	433.80	433.30	432.80	432.30	432.00	432.70	433.10	433.50	432.90
700,000	440.60	436.60	435.10	434.60	434.70	433.70	433.40	433.60	434.70	434.70	433.90	433.40	432.90	432.60	433.40	433.80	434.34	433.60
800,000	441.30	437.30	435.90	435.20	435.30	434.30	434.00	434.10	435.20	435.20	434.40	434.00	433.50	433.20	434.00	434.50	434.90	434.10
900,000	442.00	438.00	436.50	435.70	435.90	434.80	434.90	434.60	435.70	435.70	435.90	434.50	433.90	433.70	434.40	434.90	435.30	434.60
1,000,000	442.80	438.60	437.00	436.30	436.30	435.30	435.00	435.00	436.20	436.20	435.40	435.00	434.30	434.00	435.19	435.40	435.80	435.00

Discharge in Cs	1976	1977	1978	1979	1980	1981 to 1988	1989	1990	1991	1992	1993	1994	1995	1996	1997	Designed Minimum Retrogressed level
30,000	-	421.00	-	420.00	424.30	-	422.20	421.00	421.00	-	420.00	420.30	-	-	420.10	427.15
40,000	-	422.30	421.15	420.40	425.50	-	423.40	422.40	422.30	-	422.00	421.60	421.00	420.70	422.20	428.35
50,000	423.60	423.40	421.98	421.70	426.30	-	424.30	423.40	423.30	-	423.00	422.50	422.00	421.70	423.10	429.33
80,000	424.80	424.50	423.70	-	428.00	-	426.30	425.60	425.30	-	425.00	424.50	424.10	423.80	424.85	431.33
100,000	425.80	426.30	424.50	424.75	428.90	-	427.30	426.70	426.30	-	425.90	425.40	425.10	424.80	425.65	432.15
200,000	428.35	428.70	427.20	427.30	431.40	-	430.10	430.00	429.30	-	428.90	428.20	428.20	428.00	428.30	435.00
300,000	430.30	429.40	428.70	428.10	432.90	-	431.90	431.70	431.10	-	430.50	429.80	430.20	429.80	429.80	436.70
400,000	431.50	430.30	429.30	429.40	434.00	-	433.10	433.00	432.30	-	431.70	431.00	431.40	431.10	430.90	438.00
500,000	432.30	431.60	431.00	431.00	434.80	-	434.10	434.00	433.30	-	432.80	432.00	432.30	431.90	431.70	439.00
600,000	433.15	432.20	431.85	431.50	435.50	-	434.90	434.90	434.10	-	433.50	432.70	433.20	433.00	432.50	439.70
700,000	433.70	432.70	432.33	432.00	436.00	-	435.50	435.60	434.70	-	434.20	433.40	433.90	433.60	433.00	440.30
800,000	434.30	433.20	432.80	432.55	436.60	-	436.00	436.20	435.30	-	434.80	434.00	434.50	434.30	433.55	440.90
900,000	434.70	433.70	433.30	433.05	437.00	-	436.60	436.70	435.80	-	435.30	434.60	435.10	434.90	434.00	441.40
1,000,000	435.20	434.10	433.68	433.55	437.40	-	437.00	437.30	436.30	-	435.70	435.00	435.60	435.30	434.00	441.80

Table B2.2.2 Summary of Gauge Dischrge Relationship at Taunsa Barrage

Discharge in Cs	Minimum Gauge in 1954	Minimum Gauge in 1958 - 1968	Actual Retrogression from 1954 to 1968	Designed Minimum Rated Tail Water Levels	Designed Minimum Retrogressed Tail Water Levels	Designed Magnitude of Retrogression Provided	Actual Tail Water Level from Min. Envelope for 1995	Actual Tail Water Level from Min. Envelope for 1996	Actual Tail Water Level from Min. Envelope for 1997	Actual Retrogression at Present
30,000	426.50	418.00	8.50	-	-	-	-	-	420.10	-
40,000	427.70	419.50	8.20	432.00	428.35	3.65	421.00	420.70	422.20	9.80
50,000	428.50	420.80	7.70	-	-	-	422.00	421.70	423.10	-
80,000	430.40	423.30	7.10	434.40	431.30	3.10	424.10	423.80	424.85	9.55
100,000	431.30	424.50	6.80	435.15	432.15	3.00	423.10	424.80	425.65	9.50
200,000	434.10	427.90	6.20	437.50	435.00	2.50	426.20	428.00	428.30	9.20
300,000	435.70	429.70	6.00	438.90	436.70	2.20	430.20	429.80	429.80	9.10
400,000	436.70	431.00	5.70	439.90	438.00	1.90	431.40	431.10	430.90	9.00
500,000	437.80	432.00	5.80	440.70	439.00	1.70	432.30	431.90	431.70	9.00
600,000	437.80	432.80	5.00	441.30	439.70	1.60	433.20	433.00	432.50	8.80
700,000	436.30	433.40	2.90	441.80	440.30	1.50	433.90	433.60	433.00	8.80
800,000	438.70	434.00	4.70	442.30	440.90	1.40	434.50	434.30	433.55	8.75
900,000	439.00	434.60	4.40	442.70	441.40	1.30	435.10	434.90	434.00	8.70
1,000,000	439.30	435.00	4.30	443.00	441.80	1.20	435.60	435.30	434.40	8.60

Table B2.3.1 Results of Hydraulic Jump Analysis

Hydraulic Jump Analysis and Prformance of Stilling Basin for Designed Retrogressed Tail Water Levels

Discharge (Cfs)	q (Cfs/ft)	Pond Level (RL.ft)	Tail Water Levels as per Minimum Envelope Curve 1997	U/S TEL (ft)	D/S TEL (ft)	HL ((5)-(6)) (ft)	Ef2 (ft)	Fr1	Level at which jump will form (ft)	Initial Depth (y1) (ft)	Sequent Depth (y2) (ft)	Tail Water Levels Required for Stable Jump (RL.ft)	Difference in Tail Water Levels ((13)-(4)) (ft)	Stilling Basin Length Required (ft)	Stilling Basin Length Required (ft)	Stilling Basin Length Provideed (ft)	Difference ofStilling Basin Length (ft)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15'	15	16	17
100,000	31.45	446.00	432.15	446.05	432.21	13.84	8.329	7.079	423.880	0.850	8.095	424.095	-8.056	36.22	*37.48*	79.00	42.78
300,000	94.34	446.00	436.70	446.43	437.02	9.40	14.232	4.048	422.791	2.565	13.470	429.470	-7.230	54.52	*64.04*	79.00	24.48
500,000	157.23	446.00	439.00	447.19	439.73	7.46	18.427	3.210	421.299	4.209	17.116	433.116	-5.884	64.53	*82.92*	79.00	14.47
700,000																	
600,000 Weir	188.68	446.00	440.30	447.71	441.24	6.47	20.125	2.914	421.112	5.069	18.511	434.511	-5.789	67.21	*90.56*	79.00	11.79
64,000 L.U-S	152.38	446.00	440.30	446.82	440.86	5.95	17.580	2.976	423.284	4.335	16.206	431.206	-9.094	59.36	*79.11*	88.00	28.64
36,000 R.U-S	150.00	446.00	440.30	446.79	440.85	5.95	17.417	2.987	423.429	4.279	16.062	431.062	-9.238	58.91	*78.38*	88.00	29.09
1,000,000																	
800,000 Weir	251.57	446.00	441.80	449.04	443.28	5.76	23.530	2.627	419.747	6.581	21.378	437.378	-4.422	73.99	*105.88*	79.00	5.01
134,000 L.U-S	319.05	446.00	441.80	449.59	444.00	5.58	27.026	2.476	416.976	8.020	24.361	439.361	-2.439	81.70	*121.62*	88.00	6.30
66,000 R.U-S	275.00	446.00	441.80	448.66	443.44	5.23	24.561	2.502	418.875	7.215	22.170	437.170	-4.630	74.78	*110.52*	88.00	13.22

Item 15 is calculated by 5.0 x ((12) - (11))

Item 15' is calculated by 4.5 x (8)

Hydraulic Jump Analysis and Prformance of Stilling Basin for 1997 Tail Water Levels

Discharge (Cfs)	q (Cfs/ft)	Pond Level (RL.ft)	Tail Water Levels as per Minimum Envelope Curve 1997	U/S TEL (ft)	D/S TEL (ft)	HL ((5)-(6)) (ft)	Ef2 (ft)	Fr1	Level at which jump will form (ft)	Initial Depth (y1) (ft)	Sequent Depth (y2) (ft)	Tail Water Levels Required for Stable Jump (RL.ft)	Difference in Tail Water Levels ((13)-(4)) (ft)	Stilling Basin Length Required (ft)	Stilling Basin Length Required (ft)	Stilling Basin Length Provideed (ft)	Difference ofStilling Basin Length (ft)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15'	15	16	17
100,000	31.45	446.00	425.65	446.05	425.81	20.23	8.987	8.797	416.828	0.735	8.788	424.788	-0.862	40.27	*44.93*	79.00	38.74
300,000	94.34	446.00	429.80	446.43	430.53	15.90	15.520	5.185	415.006	2.175	14.897	430.897	1.097	63.61	*77.60*	79.00	15.39
500,000	157.23	446.00	431.70	447.19	433.26	13.93	20.908	4.596	412.350	3.313	19.943	435.943	4.243	83.15	*104.54*	79.00	-4.15
700,000																	
600,000 Weir	188.68	446.00	433.00	447.71	434.91	12.79	22.067	3.795	412.847	4.251	20.787	436.787	3.787	82.68	*110.34*	79.00	-3.68
64,000 L.U-S	152.38	446.00	433.00	446.82	434.11	12.70	19.535	4.023	414.579	3.546	18.478	433.478	0.478	74.66	*97.67*	88.00	13.34
36,000 R.U-S	150.00	446.00	433.00	446.79	434.08	12.71	19.365	4.043	414.714	3.497	18.324	433.324	0.324	74.13	*96.82*	88.00	13.87
1,000,000																	
800,000 Weir	251.57	446.00	434.40	449.04	437.30	11.73	25.686	3.387	411.618	5.556	23.976	439.976	5.576	92.10	*115.59*	79.00	-13.10(-36.59)
134,000 L.U-S	319.05	446.00	434.40	449.59	438.60	10.98	29.210	3.108	409.393	6.892	27.048	442.048	7.648	100.78	*131.44*	88.00	-12.78(-43.44)
66,000 R.U-S	275.00	446.00	434.40	448.66	437.52	11.14	26.851	3.244	410.671	6.067	24.966	439.966	5.566	94.49	*120.83*	88.00	-6.49(-32.83)

Item 15 is calculated by 5.0 x ((12) - (11))

Item 15' is calculated by 4.5 x (8)

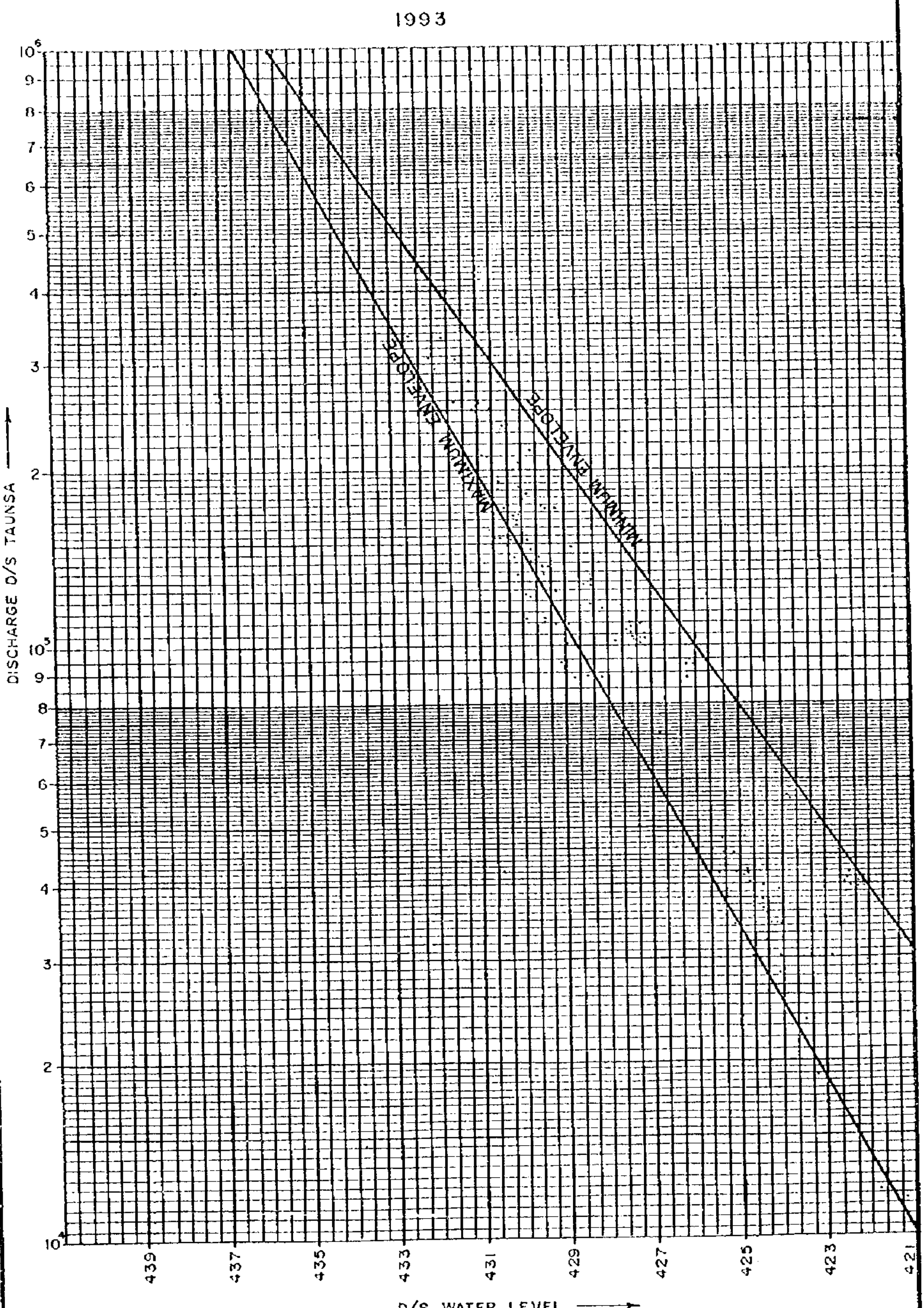

Fig.B2.2.1 Envelope / Lines for the Discharges Observed on D/S in1993