Paper No. 317

The design and construction of Deg Super Passage on Marala Ravi Link.

By
S. M. NIAZ (1)
and
NAFIS AHMAD (2)

I—Introduction :

The Marala-Ravi Link is a channel connecting Marala Headworks on Chenab River to a point on River Ravi about 36 miles upstream of Shahdara Bridge. It has a capacity at head of 22000 cs. and is the biggest channel constructed so far in Punjab. It has been constructed to remove the chronic shortage of supplies occuring on the Sutlej Valley canals since the inception of Sutlej Valley Project. The means of conveying this water from Ravi to Sutlej have already been completed by the construction of Balloki-Sulemanki Link whose capacity at head is 15182 cs. Marala Ravi Link is therefore just the upper part of B. S. Link.

Surveys for the alignment of this link were finalized in June 1953, and its construction was started immediately after-wards. The link alignment is shown in the Index Plan attached, as figure (1). The link passes through Sialkot District which on the whole receives a very good rainfall. As such the link has to cross over several channels draining the surrounding country According to the approved project, 20 cross drainage works were to be provided over the link. Out of these, 18 are syphons under the link ; and two works are to be super passages crossing over the link.

The first big nallah, called the Aik crosses the link at R. D. 90,000 and another crosses the link at R. D. 220,000 and is called the Deg Nallah. Out of these two, the latter is much bigger than the former and hence the Deg Super Passage is the biggest cross drainage work over the link. This paper deals with the design and construction of the Deg Super-Passage.

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1 S. M. Niaz P.S.E.I. Director Central Designs (Links) Irrigation
2. Nafis Ahmad P.S.E, II. Assistant Design Engineer Irrigation.

II.—Preliminary Investigations :—

1. **Discharge of the Link :**—According to the approved L-Section at the time of design, the discharge of Marala Ravi Link at R.D. 220,000 was 19,000 cs. This discharge will of course vary in accordance with the variations in the discharge at head of the link.

2 Discharge of Deg Nallah:—The maximum flood discharge of Deg Nallah at the crossing site was an important factor to be determined before taking up its design. There is no discharge site on the Deg Nallah throughout its course. A railway bridge exists on the Deg Nallah about 8 miles upstream of the crossing site ; and about 8 miles downstream the Raya Branch of the Upper Chenab Canal crosses the nallah. At the later site the Deg Nallah crosses through 3 syphons under the Raya Branch. Figure 1 shows the lay out of Deg Nallah and the position of the Railway Bridge and the Raya Branch. It will be seen that the Deg Nallah splits up into three distinct streams after crossing the Marala Ravi Link alignment. There are three syphons on the Raya Branch which take care of the flood discharge of the nallah. Their capacities are given below :—

Syphon R.D.	106120	...	9500 cusecs.
Syphon R.D.	115800	...	2500 ,,
Syphon R D.	120025	...	1000 ,,
Total		...	———
			13000 cusecs.

It is therefore clear that the Deg Nallah passes a flood discharge of about 13000 cusecs at the crossing of Raya Branch.

From the record maintained at the railway bridge, it is known that the maximum flood discharge observed during 1953 was 26000 cusecs while the corresponding discharges meassured by the Bunds Circle near the Indo-Pakistan border was 37,550 cusecs. This shows that the Deg spills over its banks, right from the border onwards. From this data, it was estimated that the super-passage should be designed for a discharge of 17000 cusecs.

The catchment area of Deg Nallah is 256 sq. miles above the border line where the hill range finishes. From the border line to the site of crossing no tributary joins it From the various empirical formulae the flood discharges have been estimated but these do not give a very reliable estimate at the crossing site as the nallah does not behave as a proper river and spills over wherever its banks are shallow. The discharges calculated by some of these formulae are however given below.

Ryves Formula $Q = CM,^{\frac{2}{3}}$
Where C is 675 generally for catchments in upper reaches.

$Q = 675 \times (256)^{\frac{2}{3}}$ $= 27,216$ Cs.

Dickens formula $Q = CA,^{\frac{3}{4}}$ assuming C = 750 for Deg catchment

$Q = 750 \times (256)^{\frac{3}{4}}$ $= 48,000$ Cs.

M. E. S. Formula $Q = 2100\ A^{\frac{1}{2}}$
$= 2100 \times 256^{\frac{1}{2}}$
$= 33,600$ Cs

These formulae give varying discharges but it will be seen that these figures are not much different from the discharge already estimated near the border. It was therefore finally decided to design the work to pass 17000 cs. with liberal free board so that if a greater discharge flows down it may be able to pass without damaging the works.

It may be worth while to mention here that the maximum discharge observed at the syphon site, when it was under construction during the record rains of September 1954, was 18,730 cusecs

3. **Type of structure :**—On this crossing, as at any crossing of two streams, three types of structures were possible :—

(i) a level crossing
(ii) Syphoning Deg Nallah under the Link, and
(iii) Super Passage for Deg Nallah over the Link Each structure was examined in detail and it was ultimately decided to construct a super passage for Deg Nallah over Marala Ravi Link. A brief discussion given below would explain the merits and demerits of each type of crossing and would explain why a decision was made in favour of constructing a super passage.

Before the three forms of crossing are discussed it is considered necessary to give the relative levels of the two channels so that the discussion about the type of crossing can be easily followed. The bed levels and the full supply levels of the two streams are given below :—

Bed level of the Deg Nallah	...	R.L. 773.80
High flood level of the Deg Nallah (1950)	..	R.L. 785.89
Bed level of the Link.	...	R L. 767.49
Full supply level of the Link.	...	R.L. 780.99

(i) **Level crossing** :—It will be seen from the above mentioned levels that the high flood level in the Deg Nallah was about 5 ft. higher than the full supply level of the link In a level crossing, it would have been necessary to keep the pond level at R.L. 785·89 (the high flood level for Deg,) which was not possible as it would have meant a rise

of about 5 ft in the full supply level of the link. This rise would have further meant a very flat slope in the reach of the link upstream of the crossing. Moreover, it will be seen that the bed of the Deg Nallah is about 6 3 feet higher than that of the link. This difference also shows that the provision of level crossing is not possible. A level crossing would also have been more costly than the other two types of structures because of the large number of gates involved. Another disadvantage would have been the induction of a lot of silt, brought by the Deg Nallah into the link. Had this silt been excluded by means of a silt excluder the cost would have increased still further. Due to these reasons the possibility of this type of crossing was ruled out.

(ii) **Syphoning Deg Nallah under the link :**—Since the discharges of the Deg and link are about equal at the crossing site the cost of a syphon and super passage for Deg would not have been very much different. On detailed examination it was found that the syphoning of Deg Nallah under the link would be more costly on account of the following additional items of works :

(a) For syphoning the deg under the link the excavation of the pit would have been necessary upto R. L. 755.5, *i e*. 6.5 ft. deeper than the excavation required for the third alternative. As the foundation material is clay below subsoil water this additional excavation would have been considerably treacherous, difficult and expensive.

(b) Due to the excavation being 23 feet below subsoil water level. the cost of pumping, during the construction of the work, would have been much more than the cost of pumping necessary for the third alternative. In the latter case the deepest excavation was only 16 feet below the subsoil water level.

(c) The syphoning of Deg Nallah under the link would not have provided a very flexible crossing. Had a greater flood than 17000 Cs occured the Deg syphon would have caused a certain afflux on the upstream, which would have inundated the surrounding villages, while for a smaller flood the barrels would have silted up and would not be effectively available for passing the designed discharge occuring later on, unless silt cleared. With the provision of a super passage for the Deg, possibilities of the afflux are reduced and the flexibility of passing a greater discharge than the designed is there.

(iii) **Super passage for Deg Nallah over Marala Ravi Link .—** Having ruled out the above two alternatives the only way left was to cross the Deg Nallah over the Marala Ravi Link through the super passage. The main advantage of the super passages is that it can pass much greater discharge than for which it is designed without causing much afflux on the upstream side. In addition to these advantages this alternative has the following additional advantages -—

(a) Banks of Deg serve as bridge across the link for the road traffic. Canal road is connected by a D. R. bridge on Deg Nallah. In case Deg Nallah was syphoned, banks of Canal would have been continuous but a bridge would have been necessary across link to connect road traffic. Bed width of Nallah is kept 278 ft. at. at this site whereas the bed width of link is 335 ft. Syphoning the canal has thus resulted in a saving of 57 ft. hf bridge length.

(b) Since the discharge of Deg and the link are nearly the same as already mentioned and allowing a little flexibility on Deg Discharge, the number of barrels had remained the same even if Deg was syphoned. Length of barrels from inlet to exit in the case of link going through the syphon is 346.4 ft. whereas otherwise it would have been 422.5 ft. So there is a saving of 76.1 ft. length per barrel, or in other words about 81,000 cu/ft. of reinforced cement concrete. As the Deg bed is higher than the canal full supply level it cannot be a clear super passage above the link and the link had to be syphoned under the Deg. Figure 2 shows the general lay out of the crossing and the section through barrels.

3. **Geology** :—Several geological bores had been taken at the site of crossing. A typical bore is shown in fig. 3. The natural surface level at the site is at R. L. 784.0 while the subsoil water level is at R. L. 778.5. The deepest excavation for the barrels is at R. L. 762.0 where a layer of sitff clay exists. The layers below the foundation level consist of various types of clay and clay mixed with kankar. The foundation material was not very ideal because of the clay, and hence it was considered necessary to refer the problem of foundation investigation to the Research Institute.

4. **Foundation Investigations** :—The clay layer encountered at the foundation level of the structure extends upto an average depth of 80 feet below the foundation level. Several undisturbed samples were taken at the foundation level and unconfined compression tests were carried out for computing the safe bearing capacity of the soil. In addition, the subsoil sounding resistance tests were also carried out. On the basis of all these investigations it was recommended by the institute that the safe bearing capacity of the soil may be taken as 1150 lbs. per sq. ft.

III. DESIGN OF THE SUPER PASSAGE.

1. HYDRAULIC DESIGN.

(i) **Waterway for the Deg Nallah** :—As already decided above the flood discharge of the Deg Nallah was assumed as 17,000 cs. at the crossing site. Adopting Lacey's formula

$$Pw = 2.66 \times \sqrt{Q}$$

where Pw is wetted perimeter and Q is the flood discharge

$$Pw = 2.66 \sqrt{17,000} = 347 \text{ ft.}$$

In order to reduce the length of the syphon barrels for the link, it was decided to flume the Deg Nallah by 80 per cent. Taking into account the plinth levels of the surrounding villages. it was considered that this fluming will not cause appreciable afflux and damage to the surrounding villages.

Flumed water way = 347 × 80/100 = 278 ft. This was adopted as the bed width of Deg, with 1:1 side slopes. Actually this should have been the total wetted perimeter, but was adopted as the bed width only, to allow for a greater discharge to pass through.

The bed level of the Deg Nallah just u/s of the work is R. L. 773.80 and the highest flood level of 1950 is R. L. 785.89. The bed of Deg Nallah at the crossing was assumed to be R. L. 774.0. Assuming headless conditions of flow, the depth of water during high flood will be 11.89 ft. The discharge which can safely pass through the section provided for Deg, without further heading up of the water level works out to 22,800 cusecs, according to the formula $Q = CBH^{3/2}$ where C = 2.00 for headless conditions.

B = width of channel = 278 ft.

H = Depth of flow = 11.89 ft.

It, will therefore be seen that a bed width of 278 ft. is quite sufficient.

The waterway provided for the Deg on the Railway bridge about 8 miles u/s of this site is 8 spans of 57·6 ft. each or 461 ft. A lot of water spills between this bridge and the site of crossing and as such the waterway provided for the Nallah is considered to be quite sufficient.

(*ii*) **Waterway for the Link.** Assuming a velocity of 10 ft./sec. through the barrels, the water way required was 19,000/10 =1900 sft. The table given below gives the number of barrels of various sizes required for the link. Rectangular barrels, with chamfered corners were preferred over circular barrels, as the former are easy to construct.

Size	Area per barrel allowing for chamfers	No. of barrels required	Corresponding width of channel u/s
6′ × 6′	35·5 sft.	54	377 ft.
8′ × 8′	62·0 sft.	30	276·25 ft.
10′ × 10′	87·5 sft.	22	251·5 ft.

The bed width of the link u/s of the work is 335 ft Thus in the 1st case, the channel will have to expand by 12·5 per cent and in the second and third cases, it will have to contract by 17·5 per cent and 25.0 per cent respectively. Expansion and too much of contraction in the flow are not desirable, and as such the 8′ × 8′ barrel seems better from this point of view.

The loss of head in the 3 cases has been worked out roughly in the following table :

Size of barrel	Number of barrels	Actual vel. in barrel	Loss of head
6′ × 6′	54	9·3 fts.	2·45 ft.
8′ × 8′	30	10 2 fts.	2·14 ft.
10′ × 10′	22	9·88 fts.	1·62 ft.

Loss of head is an important factor in the design of syphons in canals but it is not very important here, as there is a 7 ft fall in the link about 3 miles d/s of this work, and a loss of head upto 7 ft could be provided here at the cost of extra excavation. A greater loss of head means a greater excavation in the reach between the syphon and the fall, because the water level upstream of the syphon is to be constant and only d/s water level is to change according to the loss of head. As the loss of head is minimum

for 10' × 10' barrels, this size would be most economical from this consideration. But for this size the excavation required would be deeper, and as such the next best 8' × 8' size was considered more suitable.

(*iii*) *Quantities of concrete.* The table below gives the quantity of reinforced concrete required for the construction of barrels of various sizes.

Barrel size	No. of barrels	Thickness of vertical wall	Total volume of R. C.
6 × 6	54	1·0 ft.	374,000 cft.
8 × 8	30	1·25 ft.	358,000 cft.
10 × 10	22	1·5 ft.	477,000 cft.

This shows that 8 × 8 barrels would be cheeper as compared to the others. These would also be cheaper than 10 × 10 in terms of local excavation, which will have to be about three feet lower in case of 10 × 10 barrels.

From the above analysis it is evident that the best alternative is to have 30 barrels of 8' × 8' and the same was finally adopted. The max. velocity through the barrels would be $19,000/30 \times 62 = 10{\cdot}22$ ft /sec. If the link runs with extra discharge due to the additional supply required for Bedian Dipalpur Link, then the velocity will increase to 11 ft. per sec. This will only prevail for about 2 months, when the link runs to its new full capacity of 22,000 cusecs at head.

Having decided the number and size of barrels the next step is to calculate the exact loss of head through the barrels.

(*iv*) *Loss of head.*—The loss of head through the syphon consists of the following losses :—

(a) Due to friction.

(b) Due to entrance.

(c) Due to exit.

(d) Due to bends.

Out of these, the major loss is due to friction.

These losses are worked out below :—

(a) **Due to Friction.**

This loss is calculated by Mannings' formula.

$$V = \frac{1{\cdot}486}{N} \quad R^{\frac{2}{3}} \quad S^{\frac{1}{2}} \qquad \text{——————(1)}$$

where V = Velocity in the barrel.

R = Hydraulic mean depth.

S = Hydraulic gradient.

N = Coefficient of roughness.

The loss of head depends directly on the coefficient of roughness. This should be chosen carefully. For concrete this value varies from 0·012 to 0·015, depending on the degree of smoothness of the finished surface. For this work, the value of N is assumed as 0·015.

From (1), $S^{\frac{1}{2}} = \dfrac{NV}{1{\cdot}486\ R^{\frac{2}{3}}}$

$$R = \frac{A}{P} = 2{\cdot}09$$

$$\therefore \quad S^{\frac{1}{2}} = \frac{{\cdot}015 \times 10{\cdot}22}{1{\cdot}486 \times (2{\cdot}09)^{\frac{2}{3}}} = 0{\cdot}0632$$

and $S = 0{\cdot}004$

Total length of barrel is

Horizontal portion	=	278 ft.
U/S inclined portion. Diff. in beds of 3·24 ft. at a slope of 1 in 12	=	38·88 ft.
D/s inclined portion (Diff. in beds of 1·74 ft. at a slope of 1 in 16)	=	27·84 ft.
Total	=	344·72 ft.

Loss due to friction = ·004 × 344·72 = 1·378 ft.

(b) **Due to Entrance.**

Velocity in the link = 4·0 fps.

Velocity in the barrels = 10·22 fps.

Difference of velocity head = $\Delta h = \frac{(10\ 22)^2 - 4^2}{2g} = 1{\cdot}34$ ft.

Loss of head at entrance = $0{\cdot}1\ \Delta h = 0{\cdot}134$ ft.

(c) **Due to Exit.**

Loss of head at exit = $0{\cdot}2\ \Delta h = 0{\cdot}268$ ft.

(d) **Due to Bends.**

Loss of head due to bends is given by the formula

$$H_b = C \times \frac{v^2}{2g}$$

where C is a constant depending on the angle of bend. In this case the u/s angle is 4°, 36′ and the d/s angle is 3°, 36′.

Assuming C = 0 1 for these angles, loss of head at each bend $= 0{\cdot}1 \times \frac{(10\ 22)^2}{2g} = 0{\cdot}163$ ft

For both bends, loss of head = 2 × 0·163 = 0·326 ft. Therefore total loss of head is as below :—

Due to friction	=	1·378	
Due te inlet	=	0 134	
Due to exit	=	0·268	
Due to bends	=	0 326	
Total		2 106	say 2·11 ft.

The actual loss of head porvided for in the work is only 1·5 ft. This has one distinct advantage and one minor disadvantage These are discussed below :—

The disadvantage is that the provision of a lesser head loss would mean heading up to this extent on the u/s side of the syphon. In this case the extra heading up will be 0 61 ft. which is not much Moreover this will occur only when the link is running with 19000 cs. which will be only for about 2 months.

This heading up will flatten the water surface slope to 1 in 10,285 as compared to the original slope of 1 in 10,000, thereby reducing the mean velocity to 3·86 fps This is slightly lower than the actual velocity of 4 fps. and is not expected to induce any silting in the link.

The advantage is that by providing a lesser loss of head, the bed of the link has been kept higher by 0.5 ft. on the d/s side. This has resulted in a saving of 25·5 lacs cft. of wet e/w, meaning a saving of about Rs. 1·28 lacs without any material disadvantage.

2. Structural Design.

(*i*) *General arrangement of barrels :* Before proceeding with the detailed structural design of the barrels it was necessary to decide on the general arrangement of the syphon barrels. As already mentioned in the previous chapter, the link is to pass through 30 barrels of 8′×8′. Assuming a barrel thickness of 1.25 ft. the total length of the work across the barrels works out to 278.75 feet. Similarly the length of the barrels is 344.72 feet including upstream and downstream sloping portions, with these dimensions, it became necessary to provide joints both construction as well as expansion in both directions. The details of these joints would be discussed later on but it is necessary to give a rough arrangement here so that the structural design may be clear. Provision of expansion joints across the barrels was considered difficult and expensive, and it was therefore decided to split up the structure into three distinctly separate blocks of 10 barrels each. These blocks are separated from one another by a space of 2·5 ft. as shown in fig:(2). This space is filled up with loose earth and covered on top by means of a slab so that these blocks can expand and contract according to the temperature variations. Similarly along the barrel length, the expansion joints were arranged at the junction of the sloping and horizontal portions and at two more places in the horizontal portion. The design of the upstream and the downstream sloping portions is different from that of the horizontal portion and each would be discussed separately as under :—

(*ii*) *Design of the horizontal portion :* The horizontal length of the barrels (278 ft.) was separated from the sloping end portions by means of expansion joints as they behave quite independent of each other because of the different loadings The horizontal portion of the barrels extends under the bed of the Deg Nallah, while the sloping portions support the banks of the Nallah.

The maximum stresses in the structure will develop because of the following two extreme conditions of loading :—

(a) When the external pressure is maximum, and there is no internal pressure. This case would occur when the Deg Nallah is in the highest flood and the canal is absolutely dry.

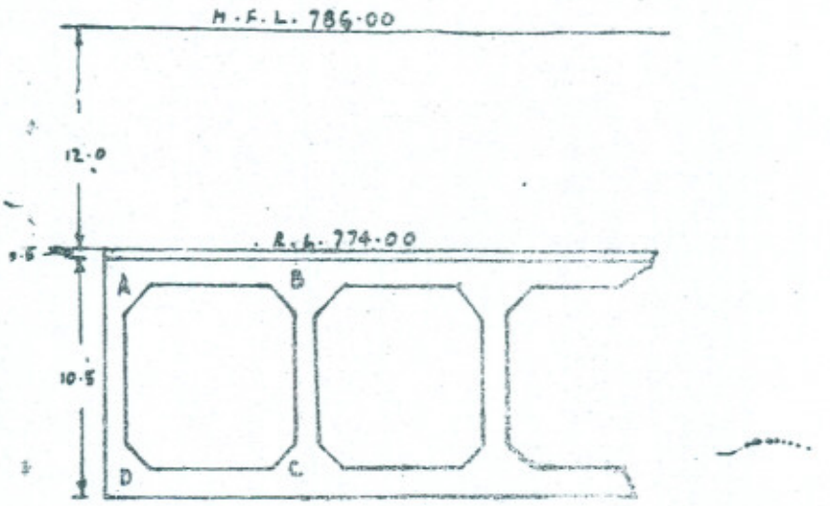
(b) When the internal pressure is maximum and there is no external pressure. This would occur when the canal is running full and the nallah is completely dry. As an extreme case it was assumed that there is no silt in the nallah. The design was worked out from the above two conditions and it was also checked later on for the following cases :—

(c) When the external and internal pressures are absent and the work is subjected to own load only.

(d) For the hoop tension.

The following text gives the detailed design of the horizontal portion :—

Case (i) Deg running full, Barrel empty.



Bending moments due to the loading on the top slab. and the side slab would be first determined, as if these were behaving in dependent of each other. These moments would then be adjusted by the method of moment distribution.

Assume thickness of every member as 1·25 feet.

Top slab.

H.F.L. of Deg :	=	785·89
Proposed top of wearing cost on barrels	=	774.00
Maximum water depth.	=	11·89 ft.
say	=	12 foot.
Water load	=	12×62.5=750 lbs/sq. foot.
Weight of 0·5 foot thick concrete wearing coat.	=	0·5×140=70
Wt. of top slab.	=	1.25×150=187-5 lbs./sq. ft.
Total u.d. load	=	1007.5 lbs. sq./ft.
	=	0.45 tons/sq. ft.

Thus the top slab is loaded at the rate of 0·45 tons/sq. ft.

Bottom slab.

Weight of 2 vertical walls and chamfers = (8 × 2 × 1.25 + 2) 150

= 3,300 lbs.

Reaction on base slab due to the walls. = $\frac{3,300}{10.5}$ = 314 lbs/sq. ft.

= 0.14 tons/sq. ft,

∴ Total reaction on the base slab from below. = 0.45 + 0.14

= 0.59 Tons/sq. ft.

Side slab.

Water pressure at A. → = 62.5 × 12.5

= 782 lbs.

= 0.35 tons

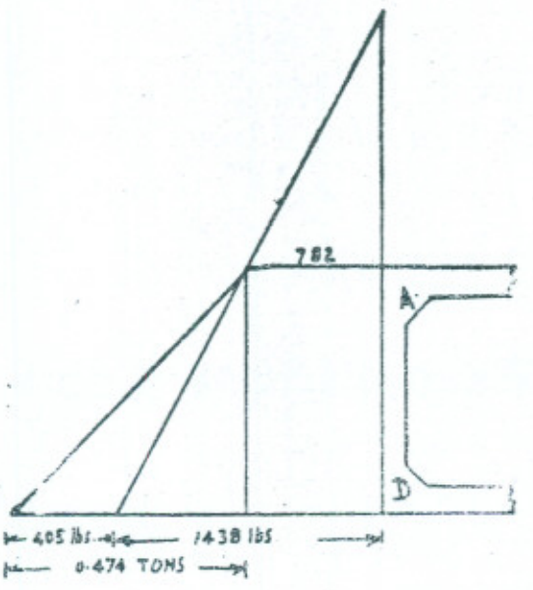
Water pressure at D → = 62.5 × 23

= 1438 lbs.

Assuming angle of repose for wet clay as 15°, Earth pressure = $62.5 \times 11 \times \frac{1 - \sin 15°}{1 + \sin 15°}$

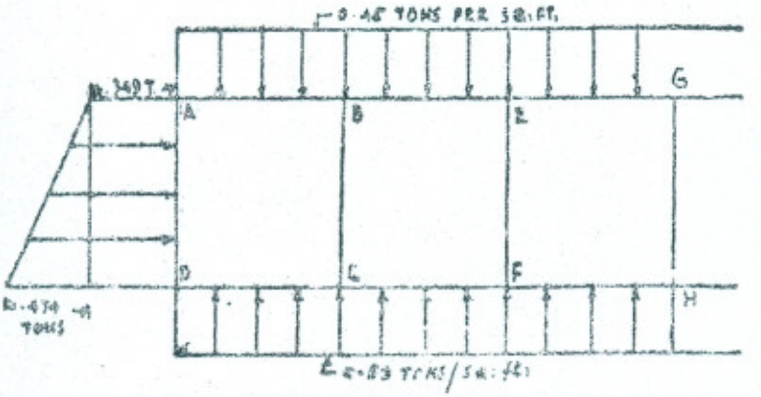
= 405 lbs.

The pressure diagram for the side slab is sketched below :—



Bending moments.

The usual convention is that the clockwise moments are called positive moments, while the anticlockwise are termed as negative moments, but in this case considering each barrel, moments giving tension on outer face are considered negative moments end those giving tension on inner face are positive.



The loading on the set of barrels is sketched above. Effective span of each member $= 8 + \frac{1.25}{2} + \frac{1.25}{2} = 9{\cdot}25$ ft.

Top slab.

Assume members AB, BE, EG, as fixed at ends, and loaded with 0.45 tons/sq : ft :

Then B.M. at ends $= \frac{-wl^2}{12}$

$$= -00.45 \times \frac{(9.25)^2}{12} = 3{\cdot}21 \text{ tons/ft}$$

Bottom slab.

Assume members DC, CF, FH loaded with 0 59 tons/sq , ft : and fixed at ends

then B. M. at ends $= \frac{wl^2}{12} = \frac{-0.59 \times (9.25)^2}{12}$

$= -4.21$ tons/ft.

Side slab.

Assume side AD as a fixed beam loaded with a uniformally distributed load of 0.349 tons/sq. ft. and triangularly distributed load of 0.474 tons at D.

B. M. due to u.d.l. at A & D $= -\frac{wl^2}{12} = \frac{-\cdot 349 \times (9\cdot 25)^2}{12}$

$= -2\cdot 49$ tons ft.

B. M. at A due to triangular load $= -\frac{wl}{15} = \frac{-\cdot 474 \times 9.25 \times 9\cdot 25}{2 \times 15} = -1\cdot 35$ tons ft.

B. M. at to A due to triangular load $= -\frac{wl}{10} = \frac{-0\cdot 474 \times (9.25)^2}{2 \times 10}$

$= 2\cdot 03$ tons feet.

$\therefore$ Total B. M. at A $= -(2\cdot 49 + 1\cdot 35) = -3\cdot 84$ tons feet.

Total B. M. at D $= -(2\cdot 49 + 2\cdot 03) = -4\cdot 52$ tons feet.

The actual bending moment at each joint is calculated in figure (4) by the moment distribution method. Since the length and thickness of each member is the same, the stiffness factor $\frac{I}{L}$ is the same for each member, and thus the moments distributed are equal.

Maximum negative moment at each corner for case (*i*) is given in the table below :—

TABLE

Corner		*Maximum neg : moment.*
A.	=	3.57 Tons ft.
B.	=	3.15 ,,
C.	=	4.17 ,,
D.	=	4.45 ,,
E.	=	3.23 ,,
F.	=	4.22 ,,
G.	=	3.21 ,,
H.	=	4.21 ,,

Having found the moments at each joint, the next step is to find the maximum positive moments in each member. This is calculated as below :—

Top slab :—Considering one span of top slab as simply supported with a u. d, load of 0.45 tons per sq : ft.

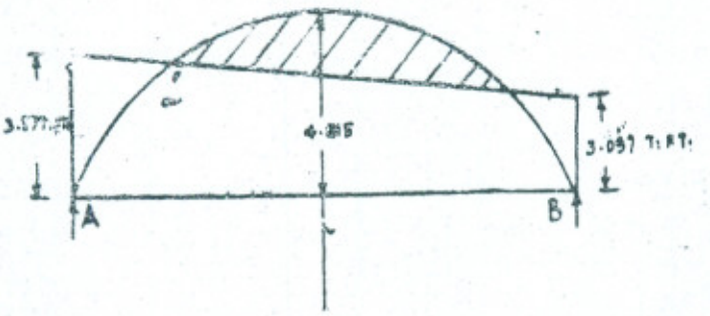
Maximum positive B.M. $= \frac{wl^2}{8}$

$= \frac{0.45 \times (9\cdot 25)^2}{8}$

$= +4\cdot 815$ Tons ft.

The end moments are −3.57 Tons ft. at A and −3.097 Tons ft. at B.

Positive Moments :—These moments are illustrated in figure below.



From this figure.

Max : + ive Moment in span A B $= 4.815 - \frac{(3.57 + 3.097)}{2}$

$= 1.482$ Tons ft.

Similarly the maximum positive moments in other spans were calculated and are given in the table below :—

Span.	*Max : + ive moment*
A B	1.482 Tons ft.
B E	1.628 ,,
E G	1.601 ,,
G I	1.605 ,,
A D	2.331 ,,
D C	2.021 ,,
C F	2.125 ,,
F H	2.103 ,,
H J	2.105 ,,

Case (*ii*) **No water in Deg and Link running full :—**

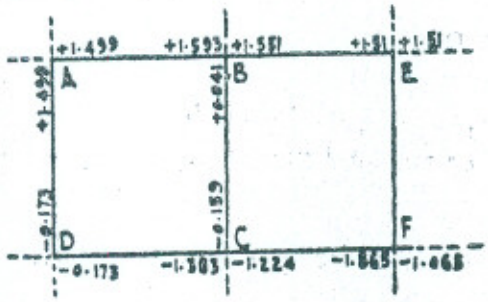
With these conditions the loads on various members are given below :—

Top slab .—An upward pressure of 0.212 Tons/Sq : ft.

Bottom slab—An upward reaction of 0.149 Tons/Sq. ft.

End slab AD :—The net pressure is from inside, at A = 0.19 tons/sq. ft. and at D = 0.07 Tons/sq ft :

The moment at each corner, due to this set of loading was calculated by moment distribution and the final result is given in the following figure :—



It will be seen that at A and B, the moment is now positive, while at C and D it is negative, but of a lesser magnitude. Similarly for the vertical member BC, the moment is of opposite sign at the bottom.

Case (*iii*) **No external or internal pressure** :—

In this condition, the work is subjected to its own load, and uplift pressure due to the subsoil water, assumed at R.L. 774·4.

Assuming 7 ft. depth of anchorage wall, 1 ft. thick, Total weight of the barrels, wearing coat, blinding layer and walls of one block of 10 barrels = 67,435 lbs.
Weight of the displaced volume of water. = 68,178 lbs.
uplift. = 68,178 — 67,435
= 743 lbs.

This means that the whole structure is subjected to this uplift force and it has to be tied down. For this purpose anchorage walls are provided at ends.

The skin friction developed on 7′ deep anchorage walls is calculated below :—

Skin friction to resist uplift is $-\mu$ R Where R is the horizontal earth pressure and $\mu = \tan\theta$
and θ is the angle of friction between the particles of wet clay taken as 15°.

Average E/press. for 7′ depth $= \frac{1/2 \times 62.5 \times 7 \times 1 - \sin 15°}{1 + \sin 15°} = 128$ lbs.

∴ Skin friction per sq : ft. = 128 × ton 15° = 35 lbs/sq : ft.
Depth of anchorage required = 743/35 × 4 = 5.3 ft. So 7 ft. deep anchorage wall is safe and is provided.

Case (*iv*) **For Hoop Tension**

(a) Average water pressure inside the barrel. = 8.75 lbs/sq. ft
Dimension along the diagonal of the barrel. $= \sqrt{64 + 64}$
= 11.3 ft.

∴ Total pressure on the ring of 11.3 ft. diameter = 875 × 11.3
= 9900 lbs.

Pressure on each corner = 9900/2
= 4950 lbs.

(b) Total direct tension in each intermediate vertical member. = .212×8×2240
= 3800 lbs.

(iii) Reinforcement of horizontal portion of Barrel.

After ascertaining the moments resulting from various forces on this structure the next step is to calculate the reinforcement required to resist the moments. The Unit stress in steel and concrete has been assumed as 14,000 lbs/ sq. inch and 650 lbs/sq : inch respectively.

The reinforcement provided in the horizontal portion of the barrels is shown in Fig. 5 and 5*a* 5/8" ϕ bars at 6" centres have been provided in addition to take up the hoop stress.

(iv) Design of the sloping portions :—

The sloping portions of barrels are subjected to the weight of earth forming banks of the Deg Nallah and any superimposed load on the bank. The superimposed load has been assumed as I. R. C., class 'A' loading The moments due to these loads were calculated exactly in the same manner as previously, and proper reinforcement provided. These portions are subjected to heavier loads and the boxing of foundations by 7 ft. deep anchorage walls was found sufficient.

(v) Design of the Breast walls :—

These walls retain the banks of the Deg Nallah on the outer sides. The U/S breast wall is 15 ft. above the barrels, while the D/S wall is 16·5 ft. high.

These walls have been designed essentially as cantilever retaining walls, subjected to the earth pressure from inside.

Assuming a hydraulic gradient of 1 : 6 from the H. F. L. of Deg, the wet earth pressure and the water pressure on the walls has also been taken into account. These walls are kept 15 inches thick throughout their height, and are in continuation of the top slab of the barrels.

(vi) Design of the Bridge :—

A bridge had to be provided over the Deg Nallah for the inspection road. This has been accomodated on the right, just downstream of the last barrel, as shown in Fig. 2. The bridge has 9 spans of 31 ft. and is designed to carry I.R.C class A loading. The roadway is 14 ft. wide with 0.75 ft curbs on each side. The piers under the bridge are 3.1 ft. wide,

and the decking consists of 3 tee-beams spaced 4.0 ft. apart with 6 inches top slab. The piers are based on built up wells, as these had to be accomodated on excavated ground.

(*vii*) *Joints*

Two types of joints have been provided in this structure, the construction joints and the flexible expansion joints. Construction joints are provided for allowing the concrete to take the initial contraction and to mark the section where the day's work should end, The expansion joints are provided to allow for expansion and contraction of the structure according to the temperature variations.

In this structure such joints have been provided in both directions. Across the barrels, the expansion joints have been provided by dividing 30 barrels into 3 blocks of 10 barrels each, with an earth cushion of 2.5 ft. in between each block The construction joints in this direction have also been provided and are shown in fig : 6.

Along the length of barrels, two expansion joints have been provided at the junction of the sloping portions with the horizontal portion of the barrel. The remaining horizontal portion is 270 ft. long. This has been divided into 3 portions 90 ft. long by providing two more expansion joints in this length. The details of these joints are shown in fig. 6, 6a and 6b.

Three construction joints have also been provided in the horizontal portion midway between the expansion joints. The water tightness at these joints has been ensured by embedding G. I. sheet strips in the construction joints and copper sheets in the expansion joints.

(*viii*) *Training works.*—As Deg Nallah has considerable spills, it was essential to provide proper training works on the upstream to ensure against the outflanking of the work. A right marginal bund, left marginal bund and a spur, have been provided as shown in Fig. 7, to train the flow to pass through the guide banks.

The guide banks are the gradually expanding type, with 1 : 5 splay. Length of these is kept according to Spring's empirical rule. On the upstream side, guide banks extend upto 450 ft. from the upstream end of flared out walls. The ratio $\frac{L}{W}$ for this is 1·29 where L is length of guide bank and W is width of waterway. Straight length of downstream guide banks is 70 ft. Top width of guide banks is 25 ft in straight portion and at the upstream nose is increased to 30 ft. Inner and outer faces have a slope of 2:1. Inner face in the straight reach is brick pitched and the nose portion is armoured with stone. Usual stone apron is given around the noses and the straight portions.

(*ix*) *Pressure pipes.* To record the uplift pressures, 6No. 1½ inch dia. pipes are provided on the inner side of anchorage walls, under the bottom

slab of barrels and 10 pipes on the outer side. Strainers are protected with proper filters around them so that they do not get choked with clay and function properly. This shall provide a useful information about the behaviours of uplift pressures under the foundations of syphons.

IV.—Construction Materials and quantities.

Before undertaking such a big and important work, it is necessary to arrange the proper quantities of materials of construction of the required quality. The sand, and the coarse aggregate required for R C. work was to be selected carefully in order to have the concrete of the proper quality. The construction material used are briefly described below.

1. Cement.

Cement was obtained from Wah and Dandot factories. This was transported in Railway wagons upto Pasrur, and then by trucks from Pasrur to the site, a distance of about 8 miles. Total quatity of Cement used in the work amounted to 6600 tons, which is about $\frac{1}{3}$rd of the amount used in the construction of Rasul Hydel Project. Throughout the construction of this work, a regular supply of cement was maintained and the work was not allowed to suffer on this account. The Cement obtained for this work conformed to the British standard specifications.

2. Fine Aggregate.

Locally available sand was very fine and had a modulus of fineness of 0·8, which is very low and renders it unsuitable for R. C. work. This sand was however used in Cement masonry and plain concrete of wearing coats. etc. Sand of proper quality had therefore to be arranged for R. C work. Marala sand had a fineness modulus of 2·01 and was considered quite suitable, but had to be brought over from Marala, about 45 miles away, in trucks, making it very costly.

Local nallahs in upper part of Sialkot District were searched for good deposits of sand, and Turram Nallah, about 26 miles from the site, was found to have large deposits of resonably good quality of sand, with a fineness Modulus of 1·87. It may be mentioned here, that the ideal sand should have a fineness modolus of 2.2 to 3.6 Fig. 8 shows the gradation of this sand, along with the British Specifications for ideal sand. The design of the conerete mix for Turram sand was done by the Research Institute, and the concreting in the early stages was done by using this sand. Later on as this work progressed, it was noticed that the fineness modulus of Turram sand was falling, and it became necessary to procure sand from Harro Nallah near Campbellpur. The fineness modulus of Harro sand was 3.81. and its gradation is also shown in Fig. 8. Its modulus was rather high and moreover it was quite costly, and in order to economise, 50% of Turram sand was mixed with 50% of Harro sand. This mixture gave a suitable fineness modulus, and a concrete mix was designed for this sand by the Research Institute. The various concrete mixes are discussed under item 4.

Total quantity of sand used in the work is given below :—

Local sand	3,57,000	cft.
Turram sand	3,50,000	cft.
Harro sand	1,02,000	cft.

3. Coarse Aggregate:

The coarse aggregate was obtained from Pakki Shah Mardan Quarry under the control of Kalabagh Division of Irrigation Branch. This aggregate varied in size from 1" to $\frac{1}{4}$", and had a fineness modulus of 7·37. The gradation of this shingle is also shown in figure 8, Some quantity of coarse aggregate was also obtained from Taxila later on. Total quantity of coarse aggregate employed on this superpassage amounts to 398,000 cft.

4. Concrete :

In the earlier stages of construction, Turram sand was employed for concreting of the anchorage walls and syphon barrels. The concrete mix as designed by the Irrigation Research Institute, for a 28-day strength of 3000 lbs/sq : inch. is given below :—

Cement	1	part by volume
Turram sand	1.5	,, ,,
Coarse aggregate	3.42	,, ,,

with this mix water cement ratio recommended was 0.76

In actual practice, this mix normally gave a 7-day strength of 2282 lbs/sq: inch and a 28-day strength of 3500 lbs/sq. inch. Such tests were carried out by the Research Institute regularly during construction.

With the introduction of Campbellpur sand, the ratio of materials in the concrete had to be revised. The new mix recommended by the Institute was as follows:-

Cement	1	part by volume
Campbellpur sand	1.8	parts by volume
Coarse aggregate	3.36	parts by volume
Water cement ratio	0.76	

This mix normally gave a 7-day strength of 3246 lbs/sq inch. and a 28-day strength of 4000 lbs/sq: inch.

As the Campbellpur sand was costly, a 50-50 mixture of this and the Turram sand was used in concreting of the syphon barrels.

The revised concrete mix for the mixed sand is given below :—

Cement	1	part by volume.
Turram sand 50%	2	parts by volume.
Campbellpur sand 50%		
Coarse aggregate	3·8	parts by volume.
Water cement ratio.	0·76	

This normally gave a 7-day strength of 2280 lbs/sq: inch, and a 28-day strength of 3300 lbs/sq. inch.

It may be stated here that the first mix utilized more cement than the other two, and as such this much saving in the amount of Cement partially balances the extra cost incurred in transporting sand from Campbelpur.

Total quantity of concrete of various types employed on the construction of this project is given below :-

Type		Quantity.
Reinforced cement concrete	1 : 1.5 : 3.42	4,42,500 cft.
Reinforced cement concrete	1 : 1.8 : 3.36	
Reinforced cement concrete	1 : 2.0 : 3.80	
Plain cement concrete	1 : 2 : 4	2,53,500 cft.
Plain cement concrete	1 : 3 : 6	
Plain cement concrete	1 : 4 : 8	
Total concrete ...		6,96,000 cft.

5. Steel

About 1950 tons of Mild Steel reinforcing bars were used in this structure. This quantity consisted of various sizes as specified by the design. The Steel used conformed to the British standard specification, and was mostly imported.

6 Bricks.

Total No. of bricks used in this work was 24 lacs. To get a prompt supply of bricks for this work, a brick kiln was arranged at a distance of 2 miles from the site of work, where suitable clay for the purpose was available.

Total cement masonry for the work was 233,000 cft, out of which 26.500 cft was reinforced brick work.

7. Stone.

Total quantity of stone used in the work was 182,000 cft. This stone was obtained from Banganwala quarries. About 275,000 cft. of stone ballast was also arranged, out of which 50,700 cft. was employed in the 6" layer below the blinding layer, and the rest for concreting, the wearing coat over the syphon barrels.

8 Timber.

28,000 cft of Deodar wood in the form of sleepers and log was obtained for this work. This quantity was consumed in the construction of form work for the syphon barrels, and for other odd uses.

9. G. I. and copper sheets.

This was another important item, and was arranged with some difficulty, because of the shortage of this material in the country. About 60 tons of G. I. sheet and 14 tons of copper sheet were used in the construction. The copper sheets were exclusively used at the expansion joints, while the G I. sheets were used in the construction joints and for lining the wooden forms for the syphon barrels.

10. Miscellaneous stores.

In addition to the above-mentioned quantities of construction materials, a lot of other miscellaneous stores were used up in the construction. These include about 7 tons of Plastic Bitumen No. 4 for the joints, 10 tons of nails of various sizes and 5 tons of binding wire. A lot of disel oils of various types, and greases were also used.

11. Water.

Arrangement for the supply of clean water were very essential, both for use in the work and for drinking. Two tubewells were sunk on each side of the work for this purpose. Three storage tanks were also constructed, which supplied the water by means of pipe lines to various points of consumption. All shingle and sand was washed before use at the washing bins.

12. Arrangement of Material dumps:

Materials, to be used during construction, were located in bins on the two sides of barrels along Deg Nallah so that their use is convenient during construction. Sump wells were also located, three on each side, at such a distance that those do not interfere with the work. Fig (9) show the correct location of bins, godowns, sump wells, out fall drains and inspection hut to suit the site conditions.

V.—CONSTRUCTION.

1. Selection of the site

Fig 10 shows the final alignment of the link. The original alignment in the vicinity of Deg Nallah is shown as ABC, and crossed the Nallah at point B at an angle of 65°. This site was not considered suitable for the construction of the super-passage, on account of the following reasons :—

(a) Being a skew crossing.

and (b) A diversion channel would have been necessary, in order to construct the work in the existing Deg Nallah.

Survey of the Deg Nallah about a mile upstream and downstream was then carried out, and it was found that about a mile upstream from B, a left handed loop in the course of Deg existed. This provided an ideal site for the construction of superpassage. It was therefore decided to site the crossing on the right of this loop, and re-align the link in this locality so as to have right angled crossing for the link.

After construction of the work, the Deg would be diverted on to the work by means of leading cuts.

2. Excavation.

The average natural surface level at the site was R. L. 784·00, while the pit was to be excavated to R. L. 762·00. The bottom of the anchor walls was at R. L. 756·00, and the subsoil water level was at R. L. 778.5. The excavation was started on 5-11-53 by mannual labour, and the excavated material was transported away from the pit by donkeys. As the subsoil water level was only 5.5 ft. below clay, it was difficult for the donkeys to operate below the subsoil water level. After removing the top 6 ft. of over burden, curbs were laid for the installation of six open sump wells. These wells were sunk by 10-12-53 and centrifugal pumps were installed on them for dewatering the area. The pumping arrangements having been made, the excavation area was kept dry by means of a system of open drains. The excavation of the pit, then went on smoothly, and was completed by 10-1-54.

3. Anchorage Walls and Blinding Layer for Barrels.

After completing the general excavation, local excavation was carried out for the anchorage walls, This was achieved with some difficulty, as the material being clay, it supported itself fairly vertically as shown in fig 11 and very little shuttering had to employed for this excavation. Moreover the quantity of subsoil water pumped out remained low. The reinforcement for these walls was then put in place, and the first basket of concrete

was poured in on 7-2-54. One anchorage wall was first concreted, and then the foundation for laying of blinding layer was prepared The extra digging done for the trenches of anchorage walls was filled in with sand. The top layer of foundation was quite slushy and it was not possible to lay the blinding layer directly on the slush Six nch layer of dry ballast was first laid on the slush and compacted at site It was then overlaid by 6" layer of 1 : 3 : 6 plain concrete, which was to serve as the blinding layer for the barrels.

4 Shuttering for the barrels.

The shuttering was made from deodar wood sleepers, and was invariably covered on top with G. I. sheets. This covering gave a smooth finish to the concrete, and also prolonged the life of the shuttering.

The shuttering was made in three parts. One part was for the bottom haunches, while the second was for the vertical walls, and the third for the top slab and its haunches. These parts were fixed in place by means of vertical posts, which were in turn braced properly. The joints in the shuttering were covered by means of strips of G. I, sheets, so as to make these water tight.

5. Concreting of the barrels

As soon as the blinding layer was completed the reinforcement for the base slab and the vertical walls for the barrels was laid down and the base slab concrete was poured in. The bottom haunches were also poured in at the same time. The construction joints in the base slab across the barrels were kept at the end of every 2 barrels For the base slab only alternate portions were concreted at the same time. The adjacent bays were concreted after the surrounding portions had taken the initial set

After concreting the base slab of one block, the shuttering for the vertical walls was erected, and concreting was carried out. These walls were allowed to set for 7 days and then the top slab shuttering was fixed in place and concreting done in portions as for the base slab Care was also taken not to pour the adjacent block of top slab, unless the previous portions had taken the initial set

The horizontal portion was tackled first and the sloping portions were laid later.

The placing of concrete in the barrels was done by human labour. The mixing of concrete was done by concrete mixers and wriggling by pneumatic wrigglers.

6. Breast walls.

The brest walls were quite high. Then shuttering was difficult, as the wall was quite thin and very long. The shuttering was made of timber at the junction of breast wall and barrel top, and higher up it was made of

masonry in two or three stages. In order to give artificial stone effect to the wall, wooden V shaped "chaptis" were fixed to the shuttering.

7. Making up banks of Deg.

After completing the breast wall, earth was transported for making the banks of Deg. The earth was deposited in 6″ layers and was rolled by means of a tractor and sheeps foot rollers to get the desired compaction. Later the inner side of this bank was pitched with bricks to make a proper bank for the Deg.

8. General.

An artistic reinforced concrete parapet was also provided on the breast walls, inner banks of Deg and the bridge. Fig. 12 shows a view of the completed work. The work was completed on 17-3-55.