PAPER No. 401

DESIGN OF A TWO MILES LONG IRRIGATION TUNNEL FOR KHANPUR DAM PROJECT

By

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SYNOPSIS

This paper describes the design procedure adopted for the horse-shoe tunnel of Khanpur Irrigation System. The design of R.C.C. lining for this tunnel was done by the Elastic Centre Method which is the standard method for design of tunnels and sewers of any shape. The paper also describes the design of temporary steel ribs and circular steel ribs used as permanent lining for that portion of the tunnel which runs into soft ground.

GENERAL

1.1. Khanpur Dam is being constructed on Haro River near Khanpur village in Hazara District. Haro River rises in the Gali ranges and drains the western slopes of Murree hills and southern slopes of Nathiagali and then emerges in a broad valley near Khanpur. The catchment area of the river at this point is about 300 sq. miles and the average annual run-off is estimated at 3,25,000 acre feet. Khanpur Reservoir would provide a live storage of about 91,000 acre feet and is expected to yield 2,65,000 acre feet annually for irrigation and for municipal and commercial water supplies.

Khanpur Dam Project would irrigate C.C.A. of 73,000 acres located in the districts of Hazara, Campbellpur and Rawalpindi, in addition to water supplies of 66 m.g.d., for Wah Cantonement, Islamabad and Heavy Engineering Complexes at Taxila. The Khanpur Irrigation System is located in a terrain so difficult that this irrigation system is expected to be quite unique in Pakistan. More than 1000 canal structure would be constructed including 5 tunnels and numerous syphons, aqueducts, drainage crossings, road crossings, railway crossings, canal falls etc. Depending upon the terrain and the sub-soil strata, the canals would be lined canals (c.c. tiles), earthen canals, rocky channels, stone-pitched channels etc. In certain reaches water would be conveyed through R.C.C. Pipes.

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From the reservoir, an R.C.C. outlet conduit would deliver about 550 cusecs to the head reach of the main canal that would immediately bifurcate into a Left Bank Canal with a design discharge of 440 cusecs and a Right Bank Canal with a design discharge of 110 cusecs.

The Right Bank Canal is supposed to irrigate C.C.A. of about 23,000 acres situated across a mountainous terrain. The conveyance of water across the mountains would involve construction of an 8,500 ft. long tunnel, known as the Right Bank Tunnel.

DESIGN GUIDELINES

2.1. For Khanpur Dam Project, a number of tunnels have to be designed. It was, therefore, desirable to frame the design guidelines very carefully. Such guidelines ensure that everything connected with the design has been clearly defined and the designer is not losing sight of any important factor. More particularly, it is to be ensured that proper attention is being paid to the purpose of the structure, limitations of design, limitations of equipment, time limits, construction problems and economies etc. and all possible alternatives are considered. The Design Guidelines were framed in the form of a chart shown in Fig. 2. This sort of chart is quite useful where a number of engineers have to work on different designs of similar nature or they have to work on different parts of the design of the same structure.

DESIGN CRITERIA

3.1. Design Loads

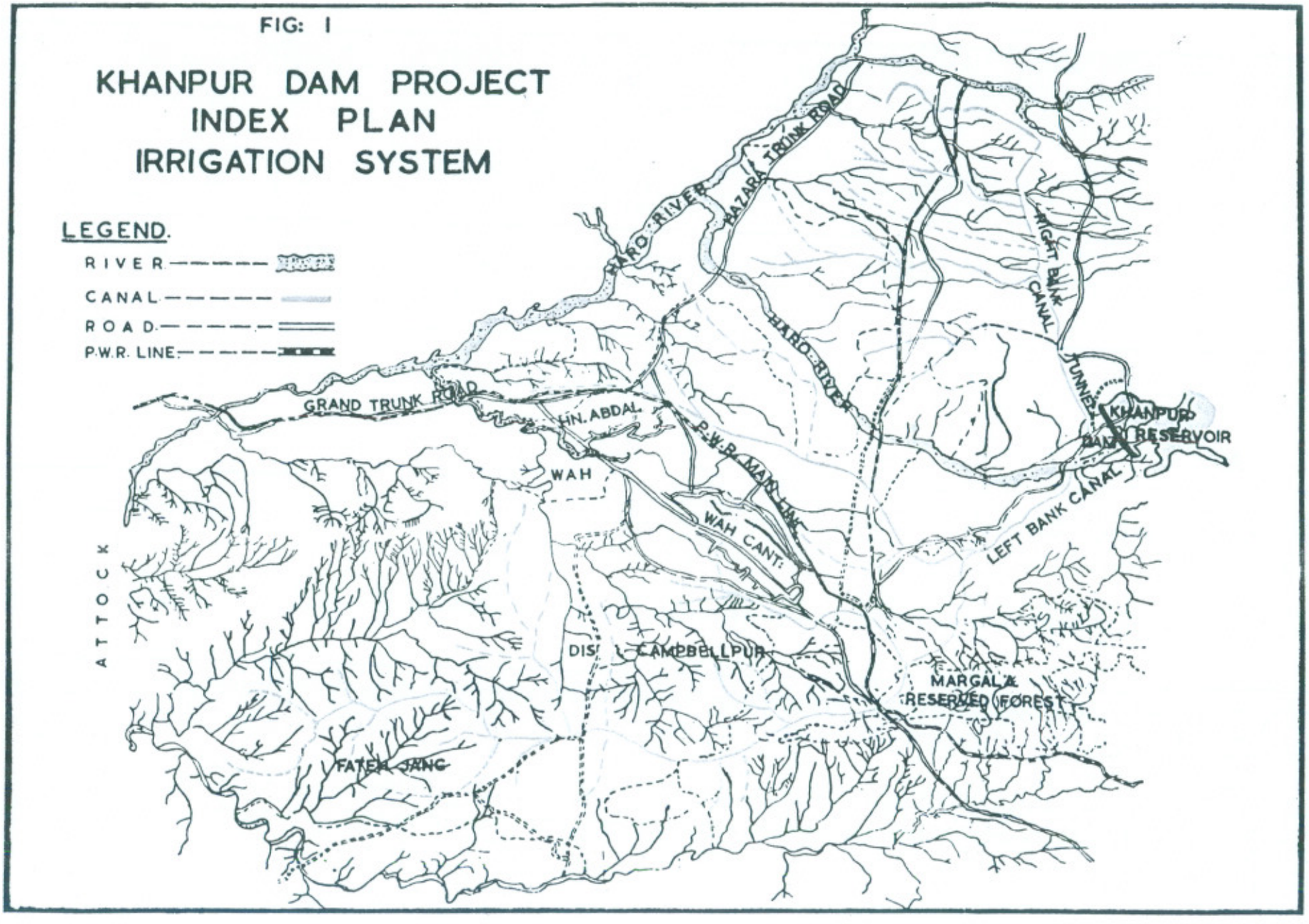
Estimation of design loads for lining and for temporary steel ribs was based on the table given in Appendix I (reproduced from Rock Tunnelling with Steel Supports by Proctor & White). This table gives recommended design loads for different geological conditions. The horizontal load was taken as 1/3 of the vertical load. The vertical and horizontal loads are assumed as uniformly distributed. It must be clearly understood that the assumed magnitude and pattern of design load is merely what may be called as 'adequate' loading and not the 'actual' loading. The 'adequate' loading is the loading which has been found as a 'safe' loading for a large number of structures, under the given conditions. Actual loading (vertical or horizontal) would be rarely uniformly distributed and may not even be symmetrical about the tunnel axis because of the rock stratification and its variations with the tunnel axis.

3.2. Foundation Reactions

On alluvial foundations, the reaction was assumed as uniformly distributed. On rock foundations, a triangular distribution of pressure was assumed

FIG: 1

KHANPUR DAM PROJECT
INDEX PLAN
IRRIGATION SYSTEM



KHANPUR IRRIGATION TUNNELS
DESIGN GUIDELINES

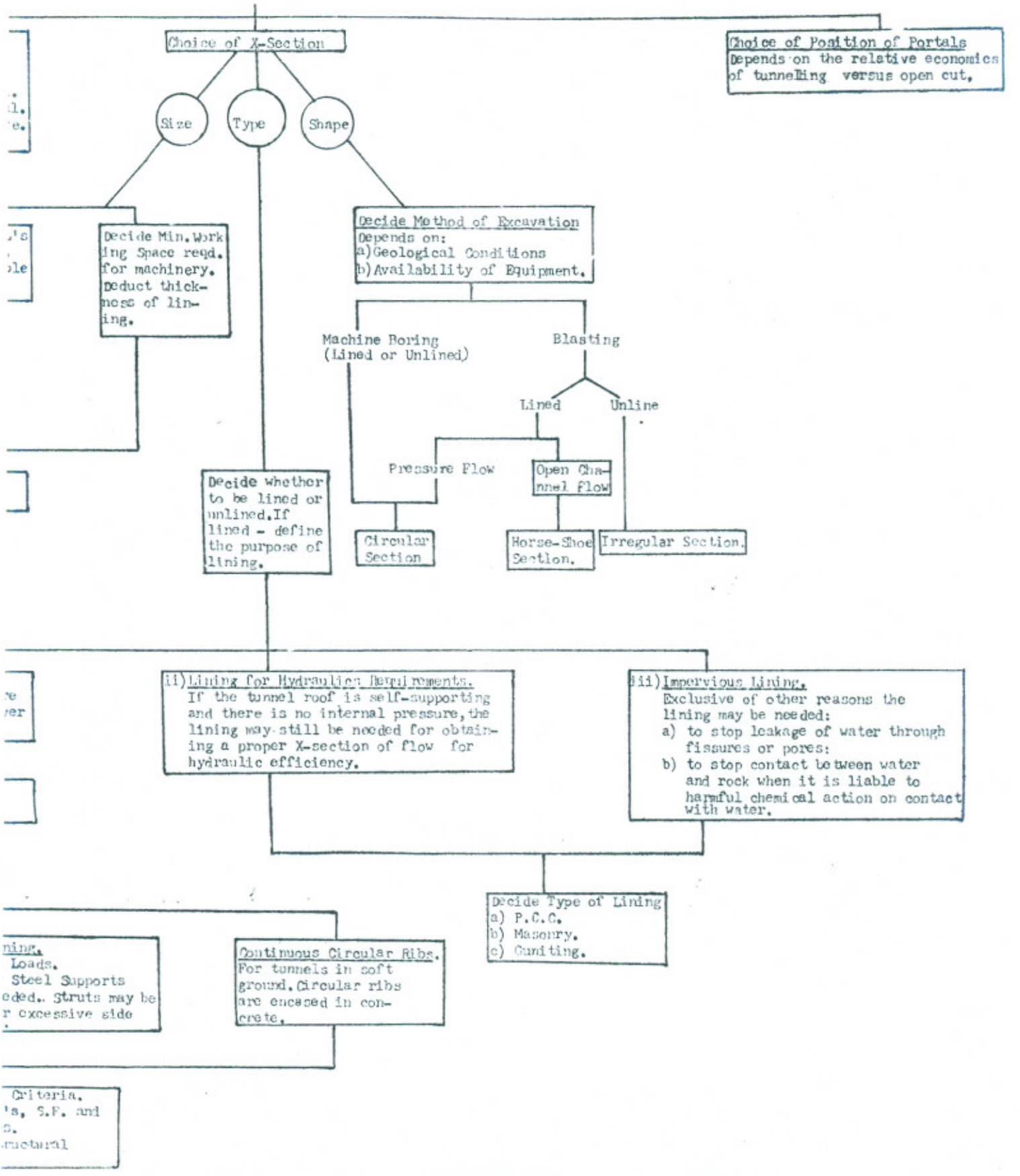


Fig. 2

KHANPUR IRRIGATION TUNNELS
DESIGN GUIDELINES

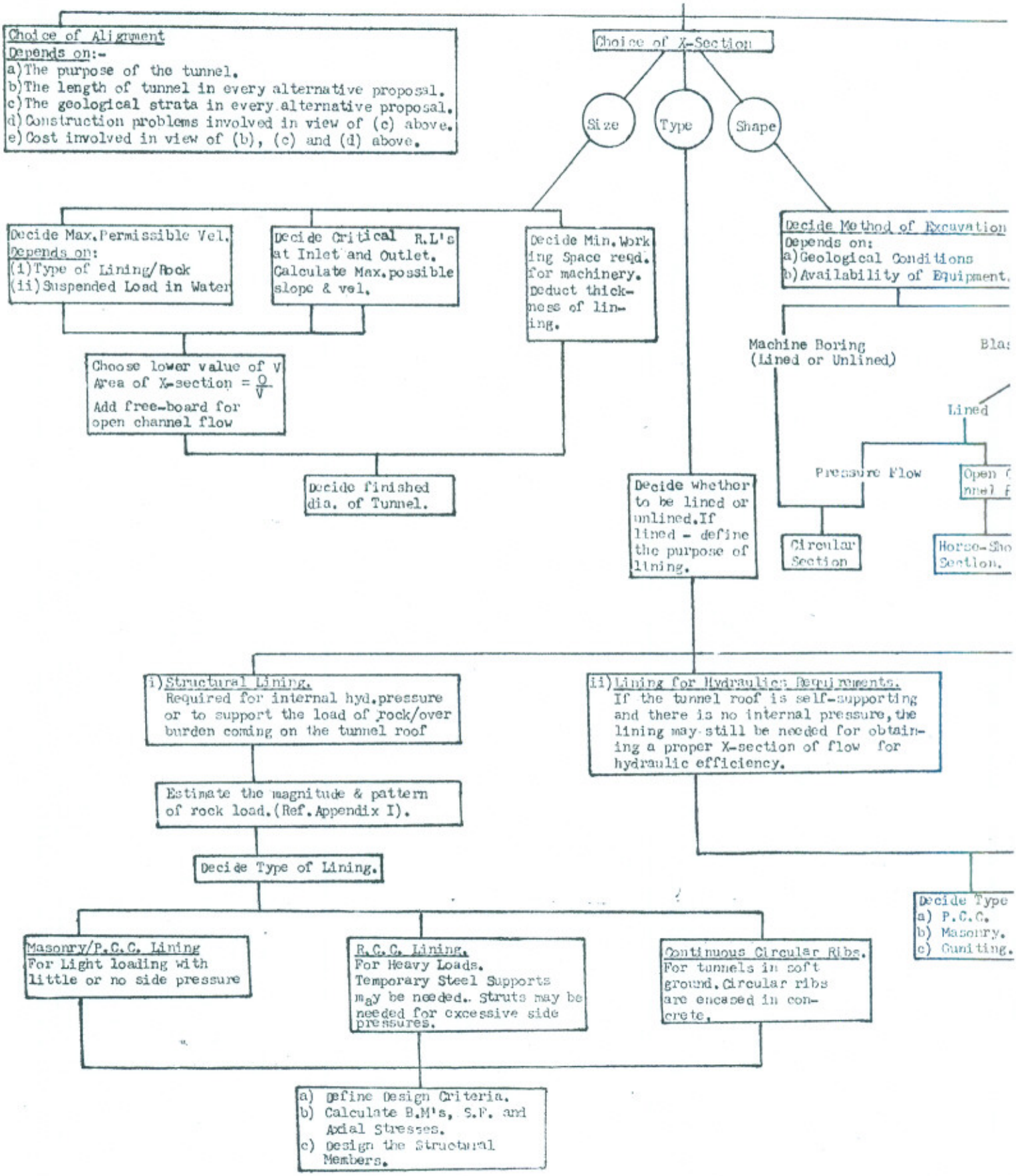


Fig. 2

as shown in Fig. 4. Again this pressure is considered as 'adequate' for design and not the actual distribution, especially when the actual loading on the tunnel is not even symmetrical.

3.3. Load Combinations

The lining was designed for the vertical load with zero horizontal pressure but it was checked for the condition of horizontal pressure equal to 1/3 of vertical pressure. The tunnel was designed for empty condition. This being a free flow tunnel, with depth of flow equal to 3.25 ft only, no special check was necessary for internal hydrostatic pressures.

3.4. Allowable Stresses

f_c = 750 psi

f_s = 15,000 psi for steel reinforcements.

f_s = 24,000 psi for steel supports (temporary load).

f_s = 18,000 psi for permanent circular steel ribs designed as part of lining.

GEOLOGICAL EXPLORATION

4.1. The geological information has been collected through surface geological studies, where bed rock is exposed. The surface geological features have been projected to the invert level of the tunnel. In addition 13 bore holes were drilled mainly to evaluate the depth of the overburden and to determine the rock conditions at the tunnel level. Seismic survey was also carried out to confirm the rock profiles.

As could be expected in a tunnel of this length, the geological conditions undergo great variations along the length of the tunnel. Geologically, the tunnel passes through the following types of the strata:—

1. Overburden

Mostly it consists of clayey silt with some angular limestone pieces. The thickness varies from a few feet to more than 160′.

2. Limestone-A

Thin to medium bedded, light to dark grey, hard, slightly weathered in depth.

3. Limestone-B

Mostly thin bedded, light grey, hard limestone with thin argillaceous bands. This limestone is not expected to be encountered at the tunnel level.

4. Limestone-C

Thin to medium bedded, yellow coloured, hard limestone with argillaceous partings.

5. Limestone-D

Massive hard limestone, dark brown colour.

6. Limestone-E

Alternating thin bedded and grey limestone.

7. Limestone-F

Complex of massive and medium bedded brownish grey limestone.

SALIENT FEATURES OF DESIGN

5.1. Tunnel Size

Referring to the chart (Fig. 2) it would be seen that the size of the tunnel depends on the X-sectional area of flow required for the Design Discharge subject to the minimum headroom required for the equipment.

If the water carries abrasive suspended load, the maximum velocity is usually restricted to $10 f_{ps}$. In the present case, the water would be silt-free and the maximum velocity is limited only by the available slope. Higher the velocity, the smaller would be the tunnel dia. for a given discharge. The critical velocities, however, have to be avoided by a safe margin. Once the flow area $= \frac{\text{Discharge}}{\text{Velocity}}$ is fixed, the tunnel dia. for a free flow tunnel is decided by choosing a reasonable free-board. Generally, the flow depth is about 0.8 times the diameter. A horse-shoe section is adopted for free flow tunnels excavated by blasting, whereas a circular section is more appropriate for pressure tunnels. A machine-bored tunnel is invariably circular in shape. In the case of Khanpur Right Bank Tunnel, one could adopt a very steep slope and the consequent high velocity could ensure the least area of flow. But the discharge in this tunnel is only 110 cusecs and one had to choose the minimum tunnel diameter from the practical considerations of the minimum headroom required for equipment during construction. After consultation with the Project staff, a 6 ft. dia. horse-shoe tunnel was adopted to achieve the required headroom of at least 8 ft. in excavated tunnel. As the steep bed slope would have resulted in a longer tunnel, a mild slope of 1 in 485 was adopted, the flow depth being about 3.25 ft. with a velocity of 6.4 fps. The free board is ample but a smaller tunnel would have created construction problems.

5.2. Choice of Lining

Keeping in view the engineering properties of the various geological formations, it was decided that structural lining would be needed for most of the tunnel length. For small portions where hard and intact rock is encountered, the structural lining would not be necessary, but to keep a uniform hydraulic section, some sort of lining would be provided. For the varying conditions of rock, lining has been designed for the following four types:—

Rock Condition	Rock Load in ft.	Remarks
1. Hard & intact	Zero	Light lining required only if spalling occurs.
2. Massive moderately jointed.	0.25 B	
3. Moderately blocky and seamy.	0.35 $(B+H_t)$	No side pressure.
4. Very blocky and seamy.	1.10 $(B+H_t)$	Little or no side pressure.

Note:—B and H_t are the outer width and height of the tunnel respectively

In addition, about 20% of the length of the tunnel passes through the strata consisting of clayey silts etc. For this portion, separate type of lining was designed. The detailed X-sections for various types of lining have been shown in Figures 10, 11, 12, 13, 14 and 15. Design calculations are given in Articles 6.1 to 7.1.

X-Section Type I

5.3. This X-section is shown in Fig. 10 and is recommended for the reach where hard and intact rock is encountered. It is assumed that the rock is self-supporting and no structural lining is required. In order to maintain a uniform hydraulic section, P.C.C. lining has been provided. The depth of flow is only 3.25 ft. and there is no need to carry the lining to the full height. The arrangement shown provides sufficient free-board, at minimum cost. The slope of 45° on the top part of lining provides a large X-sectional area of free-board, and saves some concrete as well.

X-Section Type II

5.4. This X-section is shown in Fig. 11 and it is recommended for massive moderately jointed rock. The rock load (in ft. of rock) is assumed equal to 0.25 B. This loading for a small tunnel was quite light and it was found that a masonry arch can easily take the loading. The construction of the

lower part of the X-section in P.C.C. was considered simpler than the masonry construction and this also ensured that the friction factor (Manning's co-efficient) in this reach would be the same as elsewhere and the water depth would remain uniform. The invert slab has been separated from the rest of the lining. Thus, the load on the arch is directly transmitted to the foundations through the walls and no B.M. is caused in the floor slab. The masonry arch has been back-packed to ensure somewhat uniform loading on the arch. Also, this has eliminated the risk of impact load on the arch due to falling of rock from the tunnel roof.

X-Section Type III

5.5. This X-section is shown in Fig. 12 and is recommended for moderately blocky and seamy rock. The design load in feet of rock has been taken as 0.35 $(B+H_t)$. The designed thickness of lining is 12'' and a margin of 6'' has been allowed for the pay line. The concrete cover is a minimum of $2\frac{1}{2}$'' clear. It would be noted that reinforcements on the inner face of the crown, being in tension and also being curved in a small radius, would have a resultant force in the inward direction and there would be a risk of concrete cover chipping off. To avoid this the reinforcement bars on the inner face have been tied with the outer reinforcement bars.

X-Section Type IV

5.6. This X-section is shown in Fig. 13 and has been designed for very blocky and seamy rock and would be used when no temporary supports are required. The lining thickness has been increased to 15'' because of higher design loads, i.e., 1.10 $(B+H_t)$. The other details are the same as for type III.

X-SectionType IV-A

5.7. This X-section is shown in Fig. 14 and would be used for very blocky and seamy rock in such reaches where temporary supports are required. The thickness of lining is 15'' measured from inside face of the lining to the outer face of the steel ribs. The design of steel supports is given under Article 6.6. A gap of 1/16'' at the crown is provided. This closes itself when the steel ribs are properly packed with wooden blocks. In case excessive side pressures are expected, a steel strut would be necessary at the floor level as shown in the section.

X-Section Type V

5.8. This X-section is shown in Fig. 15 and would be used for such tunnel reaches where soft ground consisting of clayey silt etc. is encountered. The design calculations of the section are given under Article 7.1. Earth

tunneling requires continuous supports, and full circle ribs are so perfectly adapted to meet the pressure conditions that any other arrangement is not worth consideration. Further, the full circle ribs serve as permanent reinforcements for the concrete lining. In fact the bending moment in the lining is negligible because the outer surface of the lining is cylindrical and the ribs are fully back-packed thus producing passive side pressures. The ribs are thus in pure thrust and are designed accordingly.

The ribs are encased in concrete and the inner face of lining has nominal reinforcements.

General Points

5.9. The six types of tunnel X-sections have been supplied to the Project Director, along with other construction details. The working drawings showing the various types of X-sections carry a note to the effect that the Project Geologist would make a detailed logging of the excavated tunnel and would categorise the rock properly for every reach. Suitable type of lining would then be chosen for every reach of the tunnel, in accordance with the rock conditions.

The position of portals was decided by considering the relative economies of open cut versus tunneling. It was found that for depths of 30 ft. or less, the open cut was cheaper than the tunneling. So the tunnel portals were positioned where natural surface is about 30 ft. above the invert level.

The tunnel is proposed to be excavated simultaneously from the up-stream and the downstream sides as well as from a central vertical shaft. Thus, four sets of equipment would be needed. The tunnel is expected to be completed in three years and is estimated to cost Rs. 70 lakh.

DESIGN CALCULATIONS

6.1. The structural design of the lining has been done by the Elastic Centre Method. Design calculations are in a tabulated form (Table I). To make the calculations understandable to the reader, a brief description of the Elastic Centre Method would be necessary.

Elastic Centre Method of Analysis

6.2. Analysis of a statically indeterminate structure is based on the following fundamental steps:—

(*i*) Remove enough reactions to transform the structure from indeterminate to a determinate one.

(*ii*) In the determinate structure compute certain angle changes and displacements including those due to forces substituted for the reactions originally removed.

(*iii*) Set computed angle changes and displacements equal to known values, which givesone equation for each of the redundant reactions.

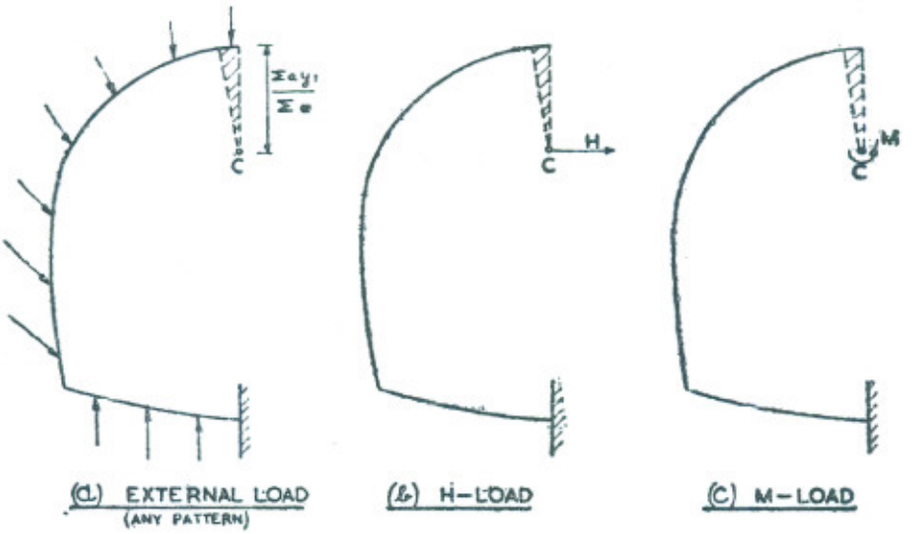


Fig. 3

(*iv*) Find out redundants and compute final moments, shears and thrusts.

It is beyond the scope of this paper to prove the theoretical equations involved in the design of statically indeterminate structures. The inquisitive reader is advised to study the references given in the Bibliography. It would be noted that we need one equation for each of the redundant reactions. If the structural member is divided into small segments of length ds, the area under the $M/_{EI}$ curve for the segment would be ds. $M/_{EI}$ = m.a. where $a = ds/_{EI}$. From the Principles of Moment Area, the following relations can be derived:—

Angle Change	$= \Delta \phi$	$= \Sigma$	Ma.....(1)
Displacement in X-Direction	$= \Delta x$	$= \Sigma$	May..... (2)
Displacement in Y-Direction.	$= \Delta y$	$= \Sigma$	Max..... (3)

(Ref: Analysis of Arches, Rigid Frames & Sewer Sections—Portland Cement Association).

Now let us consider Fig. 3 showing the half ring of a tunnel section, cut free at the crown and fixed at invert where shear is zero, An infinitely stiff bracket is extended from the crown to the Elastic Centre 'C' which is the centre of gravity of the 'a' quantities. The relative values of $a = ds/d^3$ may be used in calculations. Using C as the origin of co-ordinate X and Y, the problem is to determine displacements and rotations at C and then to calculate values of M

and H by putting the known conditions *viz.* the sum of X-displacements at C is zero and the sum of rotations at C is zero.

Now using equation (1), the rotation at C due to external load plus (H – load) plus (M – load) is equal to

$$\begin{aligned}\triangle\phi &= \Sigma ma + \Sigma H.y.a + \Sigma Ma \\ &= \Sigma ma + H\Sigma ay + M\Sigma a \\ &= \Sigma ma + M\Sigma a \qquad (\because \Sigma ay = 0 \text{ as C is the } c.g. \text{ of 'a' quantities})\end{aligned}$$

Putting $\triangle\phi = 0$, because the sum of rotation at C must be zero, in a closed ring:—

$$M = \frac{-\Sigma ma}{\Sigma a} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

Using equation (2), the sum of horizontal displacements at C due to external load plus H – load plus M-load is equal to

$$\begin{aligned}\triangle x &= \Sigma may + \Sigma H.y.\ ay + \Sigma May \\ &= \Sigma may + H\Sigma ay^2 + M\Sigma ay \\ &= \Sigma may + H\Sigma ay^2 \qquad (\because \Sigma ay = 0)\end{aligned}$$

Putting $\triangle x = 0$ at C

$$H = \frac{-\Sigma may}{\Sigma ay^2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(5)$$

Now equations (4) and (5) would be used for calculation of M and H.

6.3. Design Procedure for R.C.C. Lining

As already described, the vertical and horizontal loads are fixed in accordance with the geological conditions. Here we take the example of very blocky and seamy rock. Assuming 15″ thickness of lining:—

$$\begin{aligned}\text{Vertical load} &= 1.1\,(B + H_t) \times \gamma \\ &= 1.1 \times 8.5 + 8.5)\ \text{x0 165} \\ &= 3.18 \text{ (kips per sq. ft.)} \\ \text{Horizontal pressure} &= 1/3 \times 3.18 \\ &= 1.06 \text{ kip/ft}^2\end{aligned}$$

Half of the tunnel section is drawn to scale as shown in Fig. 4 and is divided into small segments and centre line is drawn through the mid points of the lining thickness. The foundation reaction in this case has been assumed as having a

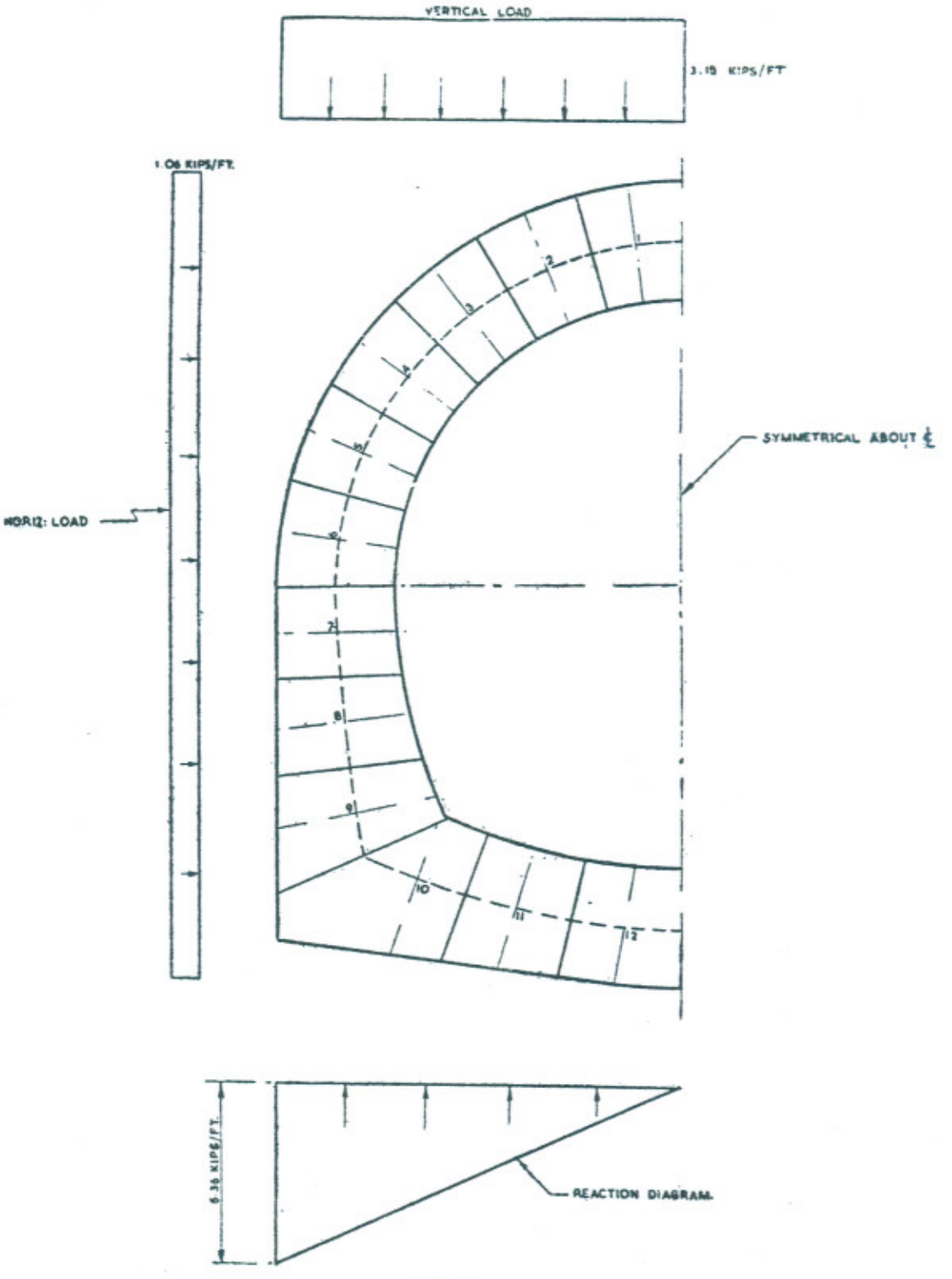

VERTICAL LOAD
3.15 KIPS/FT
1.06 KIPS/FT
HORIZ. LOAD
SYMMETRICAL ABOUT ℄
6.30 KIPS/FT
REACTION DIAGRAM

Fig. 4

TABLE I

Given					Derived				Vertical Load			Horizontal Load			Canti: Mom:	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
No.	ds	d	x	y_1	$a-ds/_{d^3}$	ay_1	y	ay^2	v	ΣV	x_n-x_{n-1}	h	Σh	y_n-Y_{n-1}	VER:	HOR:
1.	0.95	1.25	—0.475	—0.033	0.487	—0.016	+3.437	5.750	3.608	3.608	—0.915	0.243	0.243	—0.242	—0.413	—0.00735
2.	0.95	1.25	—1.390	—0.275	0.497	—0.134	+3.195	4.970	3.518	7.126	—0.814	0.442	0.685	—0.500	—3.713	—0.06595
3.	0.95	1.25	—2.204	—0.775	0.487	—0.377	+2.695	3.540	2.948	10.074	—0.671	0.685	1.370	—0.645	—9.507	—0.40845
4.	0.95	1.25	—2.875	—1.420	0.487	—0.687	+2.050	2.050	2.338	12.412	—0.475	0.927	2.297	—0.816	—16.257	—1.29340
5.	0.95	1.25	—3.350	—2.236	0.487	—1.088	+1.234	0.742	1.578	13.990	—0.250	1.080	3.377	—0.914	—22.157	—3.16440
6.	0.95	1.25	—3.600	—3.150	0.487	—1.534	+0.320	0.049	0.688	14.678	—	1.190	4.567	—0.950	—25.650	—6.24400
7.	0.97	1.28	—3.600	—4.100	0.463	—1.900	—0.630	0.184	0.185	14.863	+0.060	1.034	5.601	—0.960	—25.650	—10.5890
8.	0.97	1.42	—3.540	—5.060	0.339	—1.720	—1.590	0.856	0.207	15.070	+0.130	1.080	6.681	—0.960	—24.758	—15.9590
9.	0.97	1.71	—3.410	—6.020	0.194	—1.170	—2.550	1.260	0.249	15.319	+0.590	1.300	7.981	—0.690	—22.806	—22.2690
10.	1.17	1.65	—2.820	—6.710	0.261	—1.754	—3.240	2.740	—1.271	14.048	+1.110	0.772	8.753	—0.350	—13.766	—27.7840
11.	1.17	1.31	—1.710	—7.060	0.521	—3.680	—3.590	6.720	—3.980	10.068	+1.120	0.176	8.929	—0.170	+ 1.784	—30.8440
12.	1.17	1.25	—0.590	—7.230	0.600	—4.338	—3.760	8.590	—10.068	—	—	0.099	9.028	—	—13.024	—32.3590
SUMMATIONS					5.3	—18.398		37.451								

TABLE I

	Vertical Load			Horizontal Load			Canti: Mom:					Final Mom:			Thrust	Shear
9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
y^2	v	ΣV	$x_n - x_{n-1}$	h	Σh	$y_n - Y_{n-1}$	VER:	HOR:	m	ma	may	m+M	H.Y	M_{Final}	T	S
.750	3.608	3.608	—0.915	0.243	0.243	—0.242	—0.413	—0.00735	—0.42035	—0.204	—0.70	+22.3096	—12.00	+10.3096	+3.4	1.14
.970	3.518	7.126	—0.814	0.442	0.685	—0.500	—3.713	—0.06595	—3.77890	—1.840	—5.88	+18.9511	—11.14	+ 7.8111	+4.7	3.80
.540	2.948	10.074	—0.671	0.685	1.370	—0.645	—9.507	—0.40845	—9.91540	—4.830	—13.00	+12.8146	—9.40	+ 3.4146	+7.1	5.50
.050	2.338	12.412	—0.475	0.927	2.297	—0.816	—16.257	—1.29340	—17.5504	—8.540	—17.50	+ 5.1794	—7.15	—1.9706	+9.8	5.70
.742	1.578	13.990	—0.250	1.080	3.377	—0.914	—22.157	—3.16440	—25.3214	—12.350	—15.25	—2.5914	—4.31	—6.9014	+12.4	4.70
.049	0.688	14.678	—	1.190	4.567	—0.950	—25.650	—6.24400	—31.8940	—15.550	—4.97	—9.1640	—1.12	—10.2840	+14.2	2.60
.184	0.185	14.863	+0.060	1.034	5.601	—0.960	—25.650	—10.5890	—36.2390	—16.800	+10.60	—13.5090	+2.20	—11.3090	+14.8	1.60
.856	0.207	15.070	+0.130	1.080	6.681	—0.960	—24.758	—15.9590	—40.7170	—13.800	+21.95	—17.9870	+5.55	—12.4370	+15.2	0.80
.260	0.249	15.319	+0.590	1.300	7.981	—0.690	—22.806	—22.2690	—45.0750	—8.740	+22.34	—22.4502	+8.90	—13.5502	+15.8	0.90
740	—1.271	14.048	+1.110	0.772	8.753	—0.350	—13.766	—27.7840	—41.5500	—10.880	+35.24	—18.8200	+11.31	—7.5100	+10.5	11.60
.720	—3.980	10.068	+1.120	0.176	8.929	—0.170	+ 1.784	—30.8440	—29.0600	—15.200	+54.50	—6.3300	+12.51	+6.1800	+9.0	9.70
590	—10.068	—	—	0.099	9.028	—	—13.024	—32.3590	—19.3350	—11.600	+43.60	+3.3950	+13.11	+16.5050	+6.4	4.10
451										—120.334	+130.93					

$$M = -\frac{\Sigma ma}{\Sigma a} = \frac{120.334}{5.3} = +22.73$$

$$H = -\frac{\Sigma may}{\Sigma ay^2} = -\frac{130.93}{37.451} = -3.49$$

triangular distribution as shown. Step by step calculations have been entered in Table I.

Col. 1 shows the serial No of segments 1 to 12.

Col. 2 gives the length of each segment measured along the centre line.

Col. 3 gives the total depth of concrete section in each segment at mid-point.

Col. 4 & 5 give x and y_1 co-ordinates of the mid-points of the segments with reference to a temporary origin at the crown.

Col. 6 gives values of a $= \frac{ds}{d^3}$ which are used as relative values instead of actual values of $\frac{ds}{EI}$

Col. 7 gives values of $a.y_1$

At this point the summation of values of columns 6 and 7 is carried out to determine the vertical distance between the crown and the elastic centre C.

This is equal to $\frac{\Sigma\, ay1}{\Sigma a}$

Col. 8 gives corrected values of y co-ordinates with reference to the origin at C.

Col. 9 gives values of ay^2 and the summation.

Col. 10 gives values of v, the vertical load on each segment. This is calculated by multiplying the vertical load intensity with the horizontal projection of the extrodes of each segment.

Col. 11 gives cumulative values of v.

Col. 12 gives values like $x_2 - x_1$, $x_3 - x_2$ etc.

Col. 13 gives values of h, the horizontal load on each segment. This is the product of horizontal load intensity times the vertical projection of the extrodes of each segment.

Col. 14 gives cumulative values of h.

Col. 15 gives values of $y_2 - y_1$, $y_3 - y_2$ etc.

6.4. Cantilever Moments

Let us consider the cantilever shown in Fig. 5. The B.M's at points A, B, C & D are as follows:—

$$M_A = 0$$

$$M_B = M_A + W_1\ l_1$$

$$M_C = M_A + W_1 (l_1+l_2)+W_2\ l_2$$
$$= M_B + (W_1 + W_2)l_2$$

$$M_D = M_A + W_1(l_1 + l_2 + l_3) + W_2(l_2 + l_3) + W_3 l_3$$
$$= M_C + (W_1 + W_2 + W_3)l_3$$

In general:

$$M_n = M_{n-1} + \left(\sum_{1}^{n-1} W\right) l_{n-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

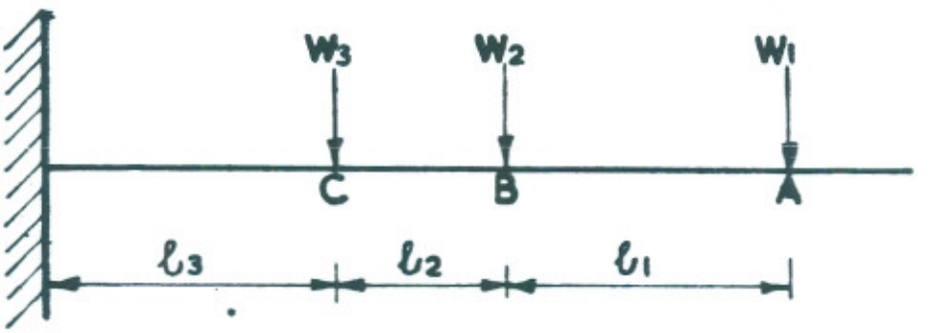


Fig. 5

This relationship would be used for calculating cantilever moments in the various segments of the tunnel section due to vertical and horizontal loads.

Col. 16. In segment No. 1, the value of moment should be zero but this leads us to a small error, because the load of the segment is not actually concentrated at the centre point of the segment.

To remedy this, the moment is taken as:—

$1/2\ v \times 1/4\ (x_2 - x_1)$

$$= \frac{3.608}{2} \times \frac{-0.915}{4}$$
$$= -0.413$$

For segment No. 2 cantilever moment is

$$-0.413 + 3.698 \times (-0.915) = -3.713$$

For segment No. 3 cantilever moment is

$$-3.713 = 7.126 \times (-0.814) = -9.507$$

Col. 17. Catnilever moments are:—

In Seg. No. 1 $1/2\ h. \left\{\frac{y_2 - y_1}{4}\right\}$

$$= \frac{0.243}{2} \times \frac{-0.242}{4}$$

$$= -0.00735$$

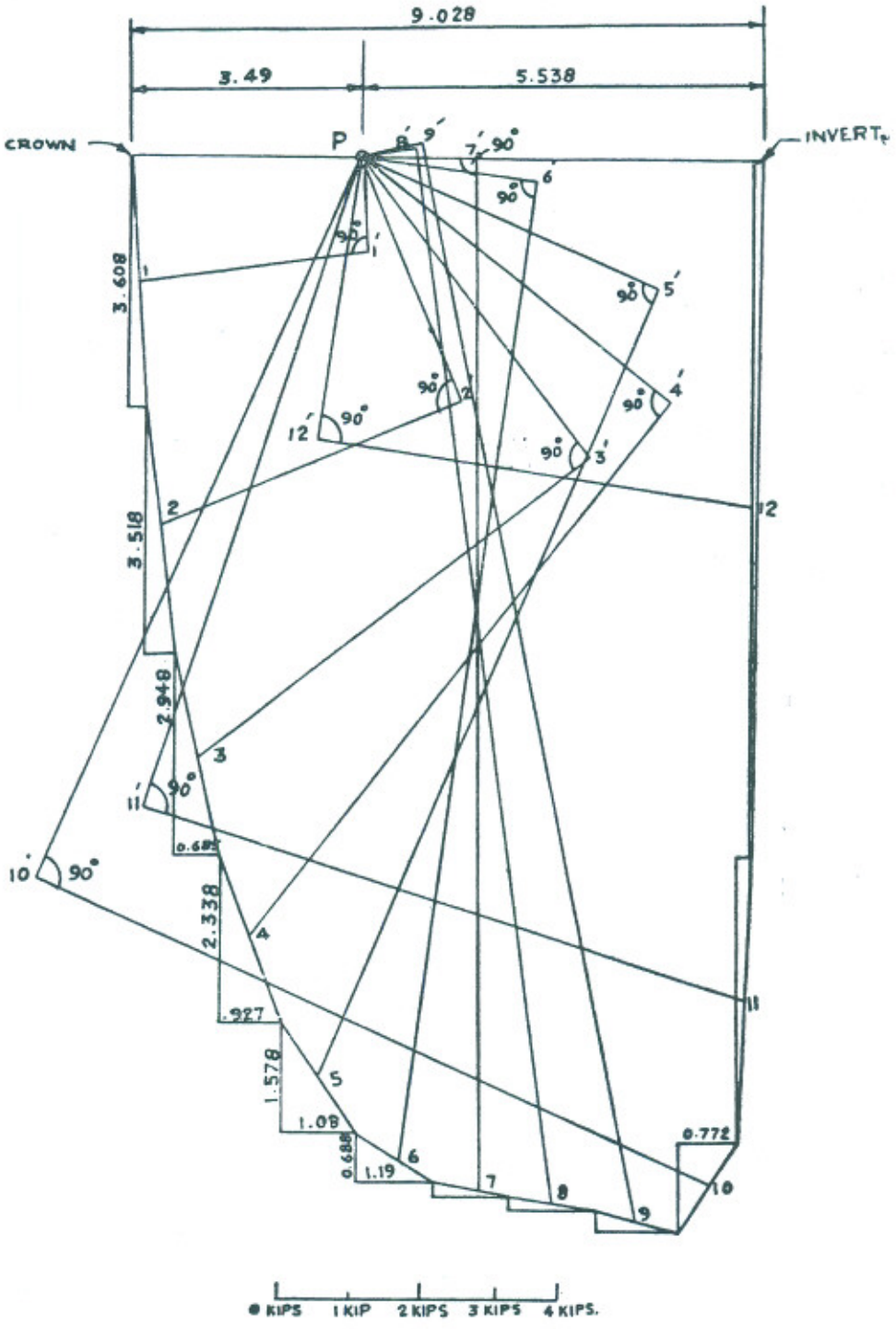

9.028
3.49
5.538
CROWN
INVERT
P
3.608
3.518
2.948
2.338
.927
1.578
1.08
0.688
1.19
0.772
90°
0 KIPS
1 KIP
2 KIPS
3 KIPS
4 KIPS.

Fig. 6

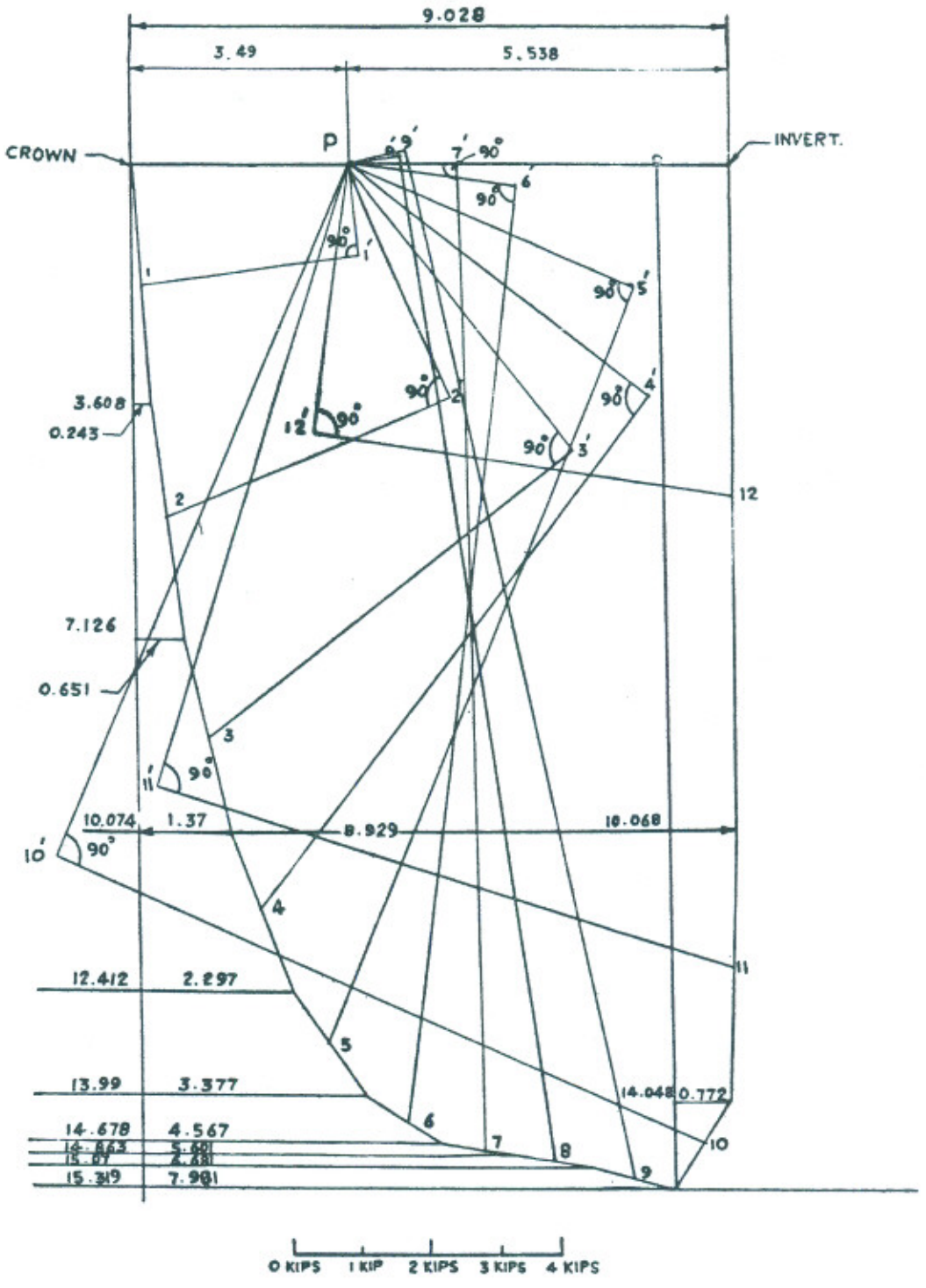

9.028
3.49
5.538
CROWN
INVERT.
P
3.608
0.243
7.126
0.651
10.074
1.37
8.929
10.068
12.412
2.297
13.99
3.377
14.678
4.567
14.048
0.772
0 KIPS
1 KIP
2 KIPS
3 KIPS
4 KIPS

Fig. 7

In Seg. No. 2 $-.00735 + 0.243 \times(-0.242)$

$= -0.06595$

In Seg. No. 3 $-0.0659 + 0.685 \times(-0.5)$

$= -0.4084$

Col. 18 gives values of Col. 16 plus Col. 17.

Col. 19 gives values of *ma* and its summation.

Col. 20 gives values of *may* and its summation.

At this stage using equations (4) and (5) values of M and H are calculated as

$$M = \frac{-\Sigma ma}{\Sigma a}$$

and $$H = \frac{-\Sigma May}{\Sigma ay^2}$$

Col. 21 gives values of $(m + M)$

Col. 22 gives values of $H.y$.

Col. 23 gives values of col. 21 plus col. 22. This is the final B.M. and is considered positive when tension is produced on the inside face.

Col. 24 and 25 give values of Thrust and Shear which are derived graphically as described below:—

6.5. Analysis for Thrust and Shear

Values of thrust and shear in various segments have been determined graphically as in Fig. 6. The forces recorded in columns 10 and 13 of Table I are drawn to scale starting downwards from the crown and horizontally from the lowest points of the *v* forces in each segment for Segment Nos. 1 to 9. Now the *V* forces are drawn vertically from the lowest point of force polygon upward for Segment Nos. 10 to 12. The *h* forces are drawn horizontally from the end points of *v* forces. This force polygon is now used to determine axial thrust and shear at the mid-points of the segments. For example, line 1—1′ is drawn parallel to the tangent to centre line at mid-point of segment No. 1. Lines 1—1′ 2—2′, 3—3′ etc. represent axial thrusts to scale and shear lines 1′—P, 2′—P, 3′—P etc. are drawn normal to the thrust lines. For greater accuracy, we may draw the force polygon by using Σv and Σh values instead of *V* and *h* values. This is shown in Fig. 7.

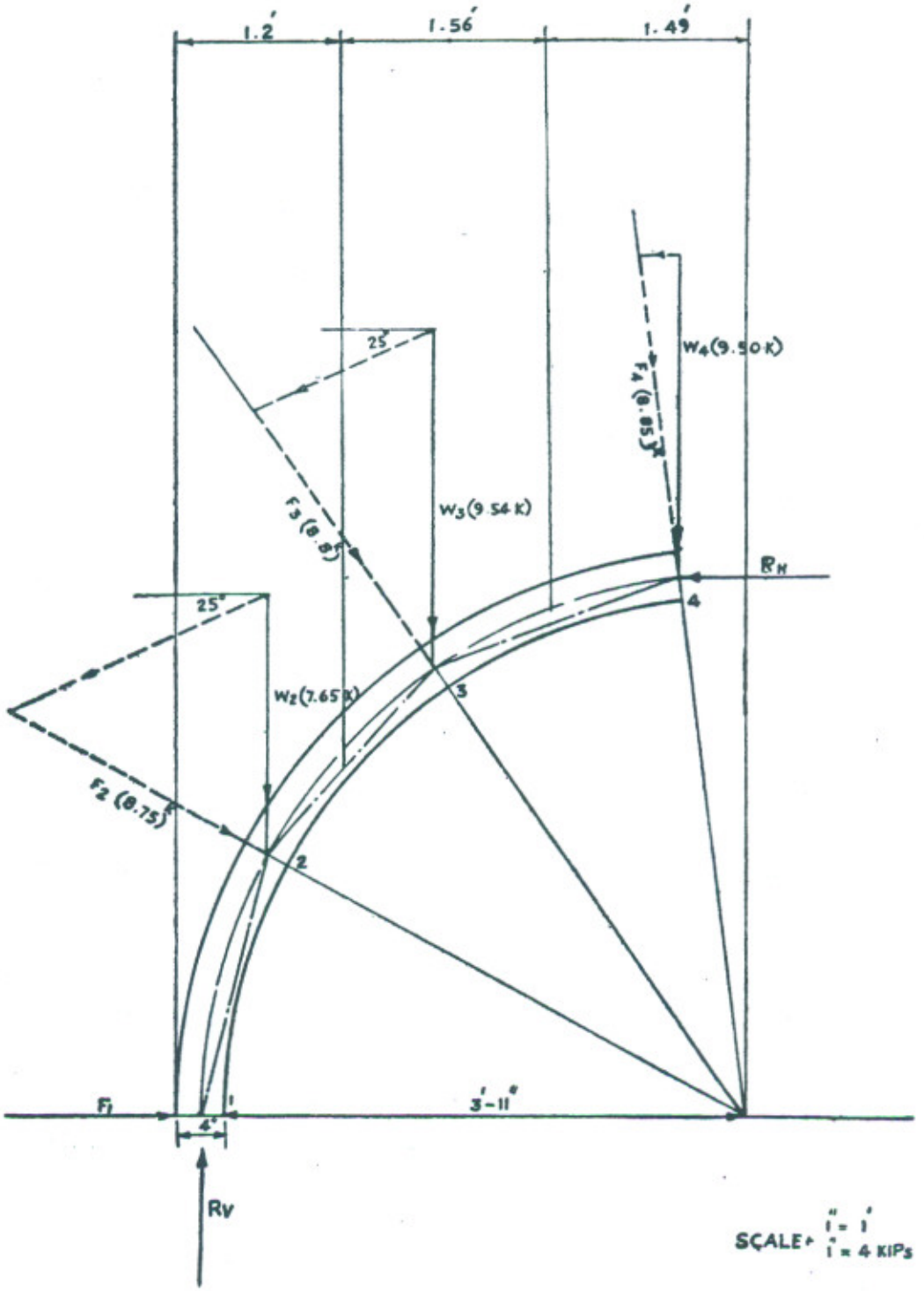
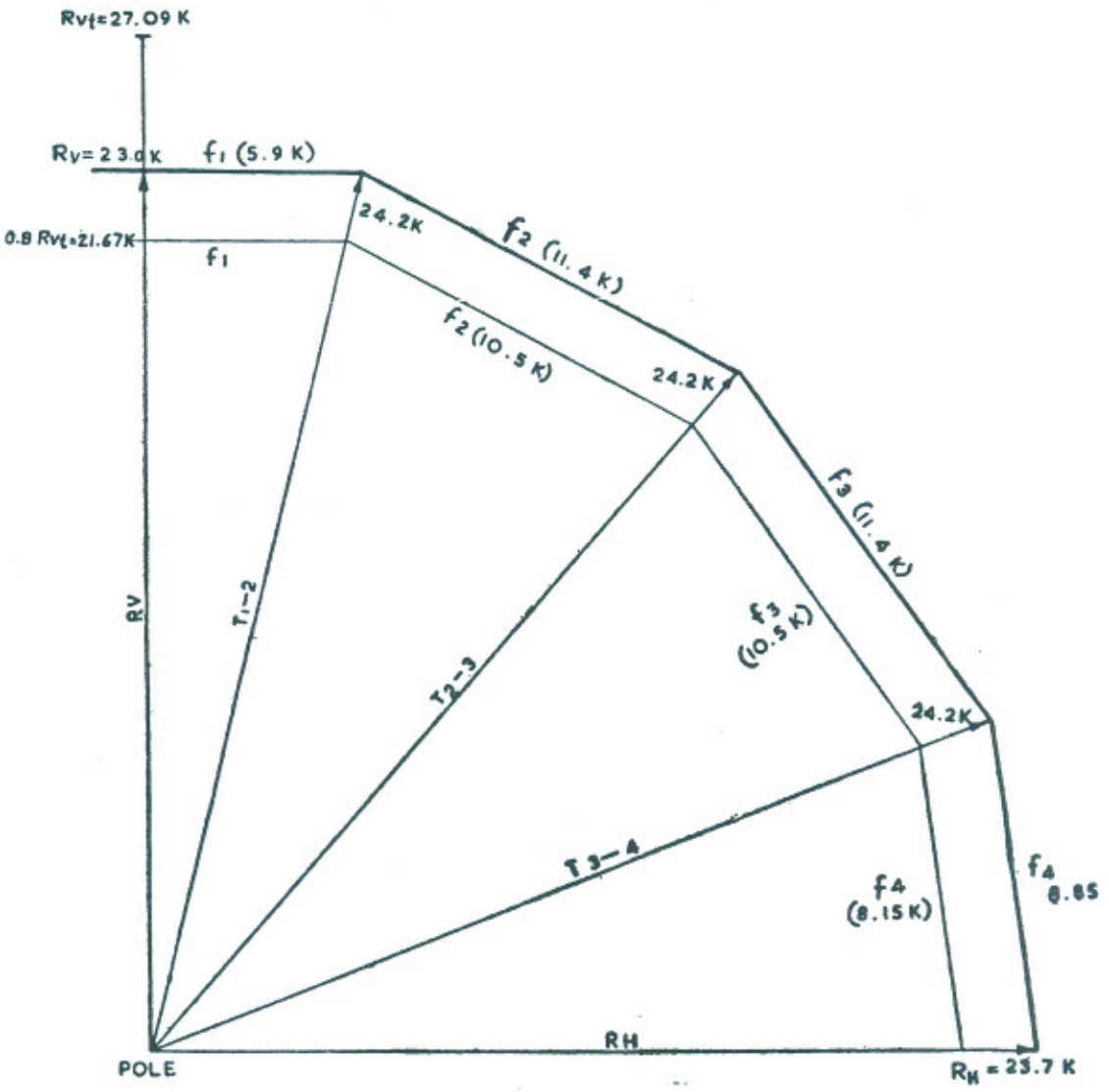

1.2'
1.56'
1.49'
25°
25°
W4(9.50K)
F4 (0.85)
W3(9.54K)
F3 (0.8)
RH
4
W2(7.65K)
3
F2 (0.75)
2
F1
1
3'-11"
4"
RV
SCALE: 1" = 1'
1" = 4 KIPS

Fig. 8



RV t = 27.09 KPS = $(W_2+W_3+W_4)$
RV = 0.8 X 27.09 (ASSUMED)
= 21.67 KPI
RV = 23.0 KPS (ACTUAL)
MAX THRUST = 24.2 KPS

Fig. 9

Having thus determined the B.M. Thrust and Shear at each of the 12 points, we can now easily determine the required reinforcements, by using standard methods for design of members with combined bending and axial load. The working stresses of 15000 psi and 750 psi were used for steel and concrete respectively. The designed cross section is shown in Fig. 13.

6.6. Design of Steel Supports

Steel supports have been designed for very blocky and seamy rock. The design consists of the following steps:—

1. Assume rib section and rib spacing.
2. Draw the design concrete line and the rib.
   Also draw the neutral axis of the rib. (Fig. 8).
3. Assume the maximum block spacing. One blocking point should be at the junction of arch and the leg, another not more than 6 inches from the crown. As the rib must be designed for the worst conditions, the 2nd and 3rd blocking points should be at maximum spacing from the crown. Similarly the first blocking point above spring line must be at max. spacing from spring line.
4. With the assumed rock load intensity applied on the horizontal projected area of the rib, calculate loads at each blocking points, W_2, W_3, W_4 etc.

   Rock Load = 3.18 K/ft²
   Rib Spacing = 2 ft.
   ∴ Load per ft of rib = 2×3.18
   = 6.36 kips.

   $W_2 = 6.36 \times 1.2 = 7.65$ kips.
   $W_3 = 6.36 \times 1.56 = 9.94$ kips.
   $W_4 = 6.36 \times 1.49 = 9.50$ kips.
   $R_{vt} = 7.65 + 9.94 + 9.50$
   $= 27.09$ kips.

5. Draw W-forces and their radial and tangential components F and Ft. If the tangent is inclined more than 25° with horizontal, a 25° component is drawn. Draw the chords connecting the neutral axis blocking points (Fig. 8).
6. From a pole, we draw vertical ray Rv and thrust lines parallel to chords (Fig. 9). Also we draw horizontal ray Rh. Actual reaction Rv is less than Rvt and as a first trial it is taken as 0.8 Rvt
   $= 0.8 \times 27.09$
   $= 21.67$ kips.

Starting from this point, f_1, f_2 etc. are drawn parallel to F_1, F_2 etc. (Fig. 8 & 9). Here a check is made whether any of the F forces are greater than f forces and if so, the rays are extended accordingly. From the extended rays, a new forces polygon is drawn for the force $f = F$ farthest from the pole. Now actual value of Rv =23.0 kips. The longest ray in the polygon gives the max. thrust = 24.2 kips.

6.7. Design of Arch Section

For I-Section 4" × $2\frac{5}{8}$", selected by trial:

A = 2.76 in²
I = 6.7 in⁴
r = 1,56 in
Z = 3.3 in³
T = 24.2 kips.
C = Chord length.
= 23.4 ins (for 24" block spacing)
h = rise of arc between blocking points

$$= R - \sqrt{R^2 - \left(\frac{C}{2}\right)^2}$$

= 1.396 in

R = Radius of neutral axis of rib = 51.99 ins.

Max. B.M. in the arch is given by equation (7).

$$M_{max} = 0.86\, h\, T \quad \ldots\ldots\ldots\

6.8. Design of Leg as Column

Reduced allowable stress is given by equation (8)

$$f_{red} = \frac{f_{all}}{18000} \times \left[17000 - 0.485 \left(\frac{l}{r}\right)^2 \right] \quad \ldots\ldots(8)$$

$$= \frac{24000}{18000} \left[17000 - 0.485 \left(\frac{54}{1.56}\right)^2 \right]$$

$= 21900$ psi.

M_1 = B.M. at spring line.

$$= 0.67 \; hT \quad \ldots\ldots(9)$$

$$= 0.67 \times 1.396 \times 24.2 \times 1000$$

$= 22{,}600$ in lbs.

d_1 = deflection of leg caused by M_1

$$= 0.0642 \; \frac{M_1}{EI} l^2 \quad \ldots\ldots(10)$$

$$= 0.0642 \times \frac{22{,}600 \times 54^2}{30 \times 10^6 \times 6.7}$$

$= 0.0211$ inch.

M_c = B.M. at point of max. deflection.

$$= \frac{(0.578\, M_1)^2}{0.578\, M_1 - d_1 R_y} \quad \ldots\ldots(11)$$

$$= \frac{(0.578 \times 22600)^2}{0.578 \times 22600 - .0211 \times 23000}$$

$= 13{,}600$ in lbs.

f_1 = Max. stress at spring line.

$$= \frac{T}{A} + \frac{M_1}{Z}$$

$$= \frac{24{,}200}{2.76} + \frac{22{,}600}{3.3}$$

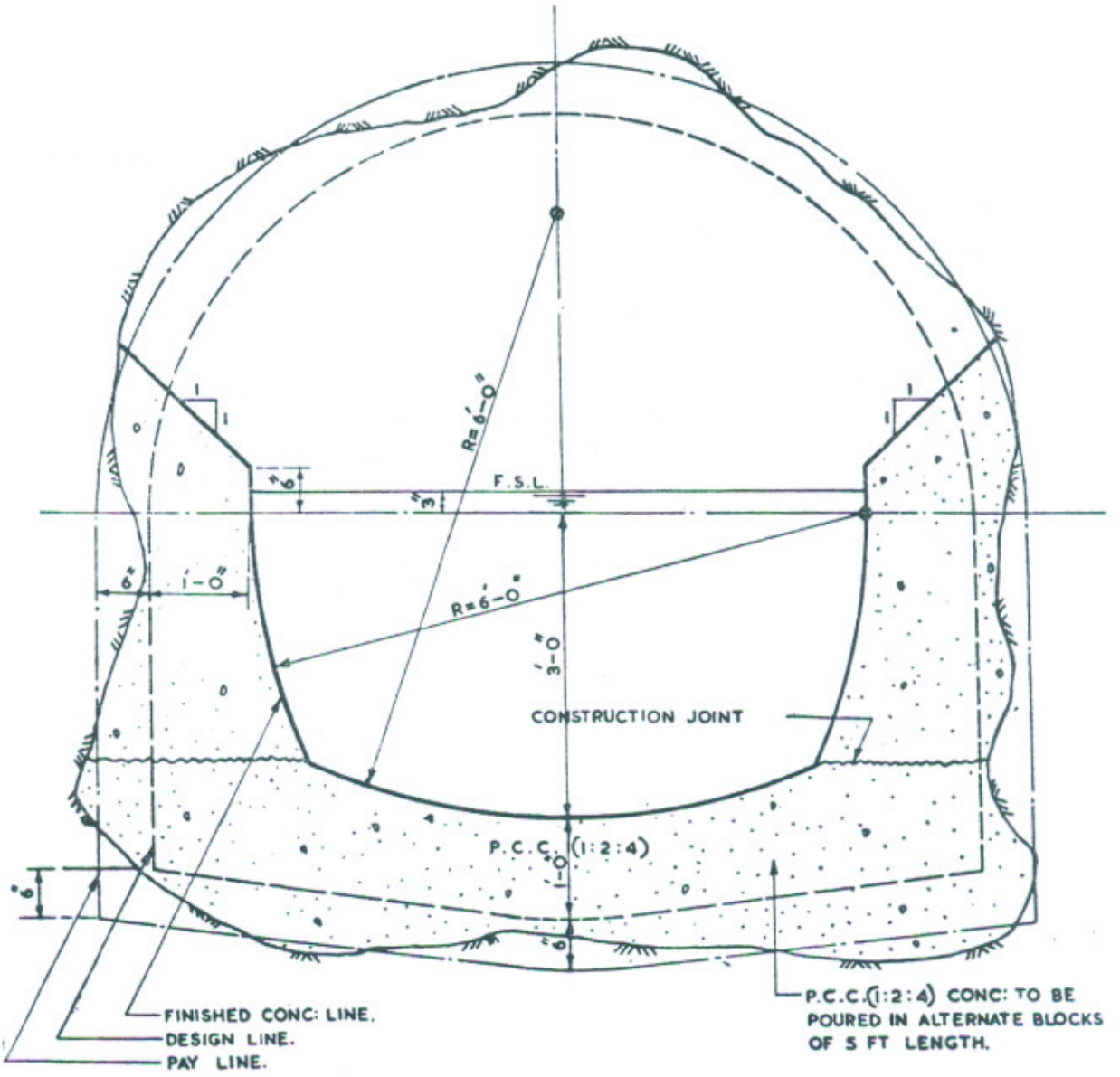
$= 15{,}600$ psi.

f_c = Max. stress at the point of max. deflection.

$$= \frac{R_y}{A} + \frac{M_c}{Z}$$

$$= \frac{23{,}000}{2.76} + \frac{13{,}600}{3.3}$$

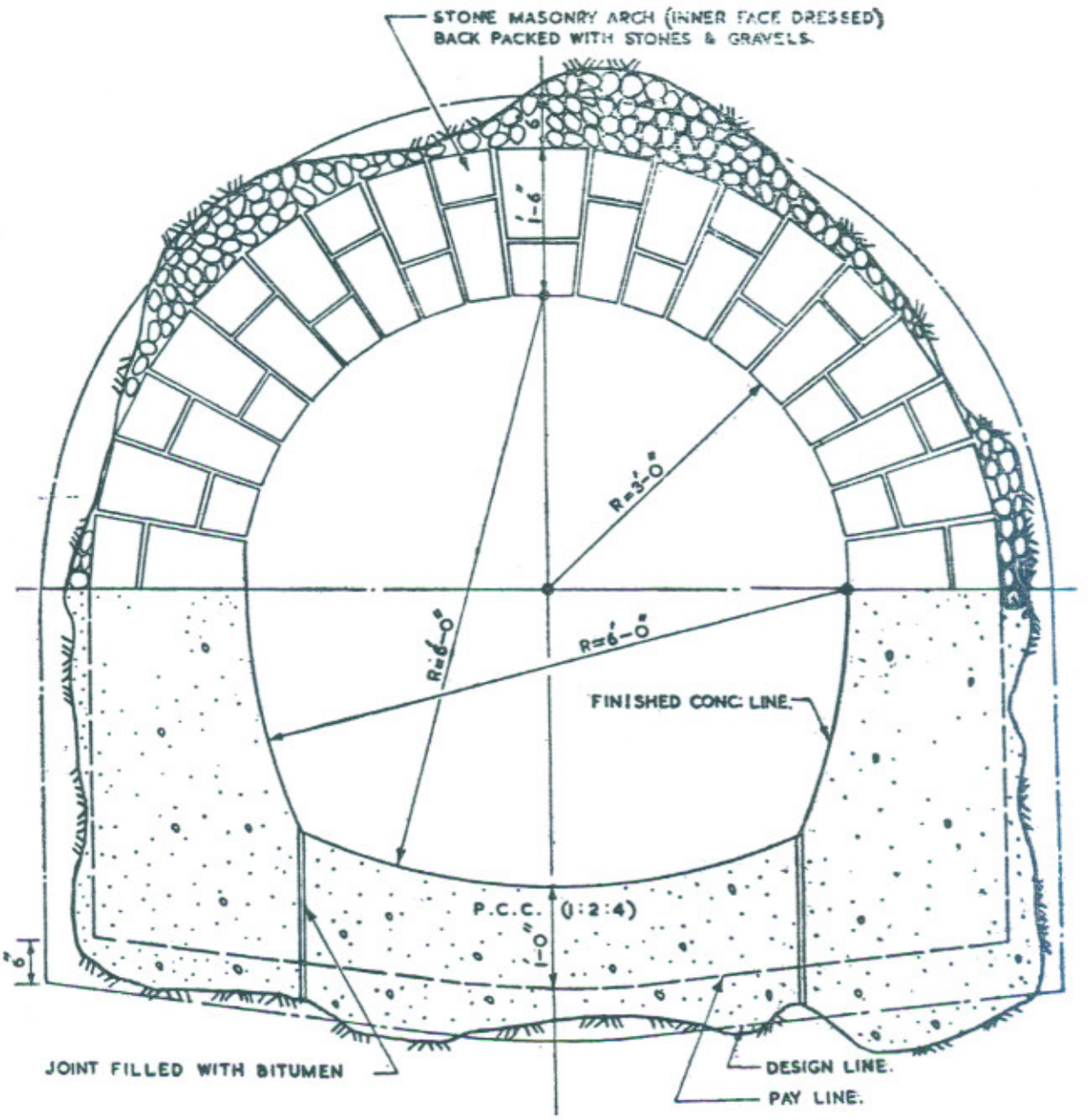
$= 12{,}500$ psi.



X-SECTION LINING TYPE I.

(FOR HARD AND INTACT ROCK)

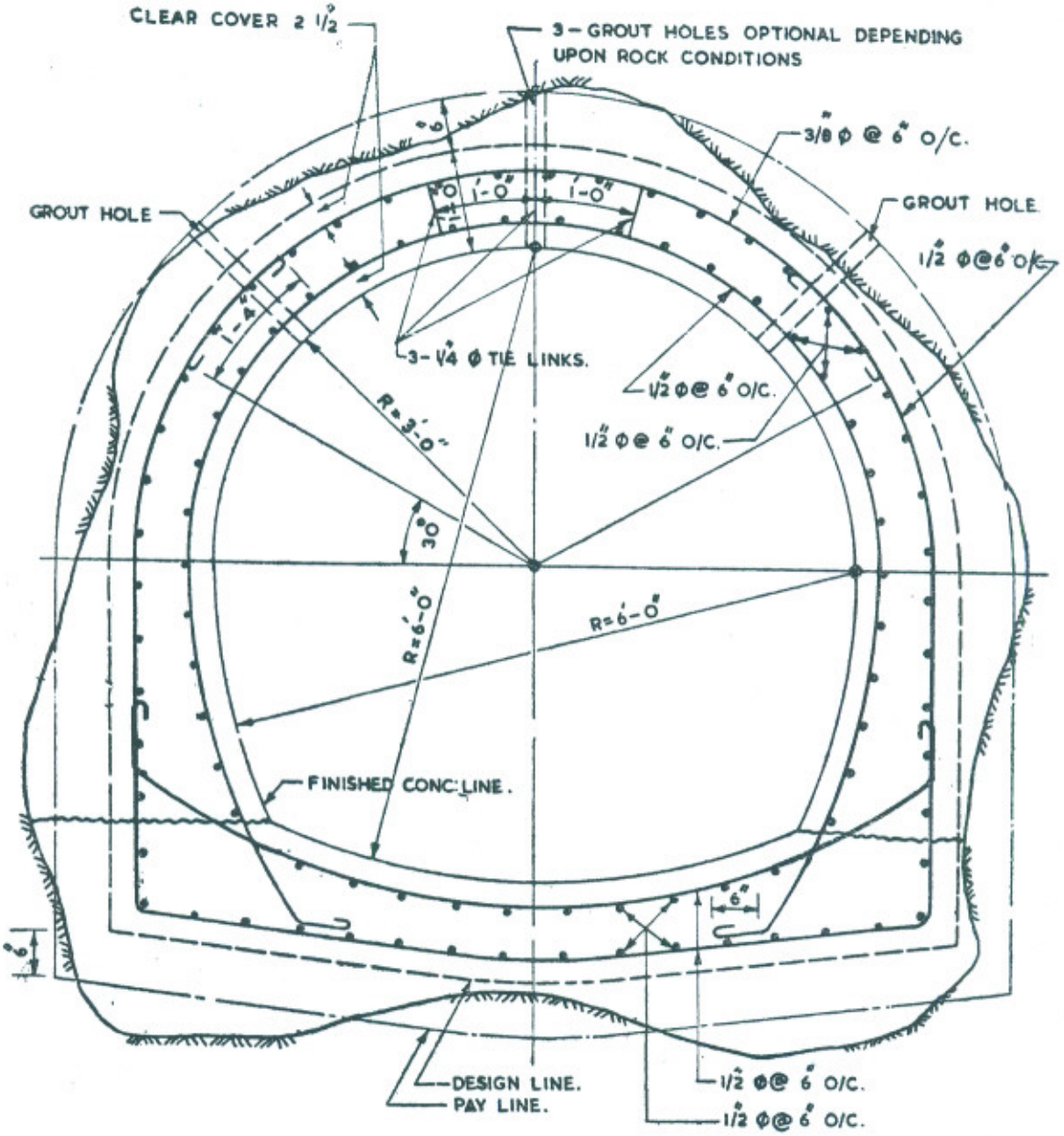
Fig. 10



X-SECTION LINING TYPE II.

(FOR MASSIVE MODERATELY JOINTED ROCK)

Fig. 11

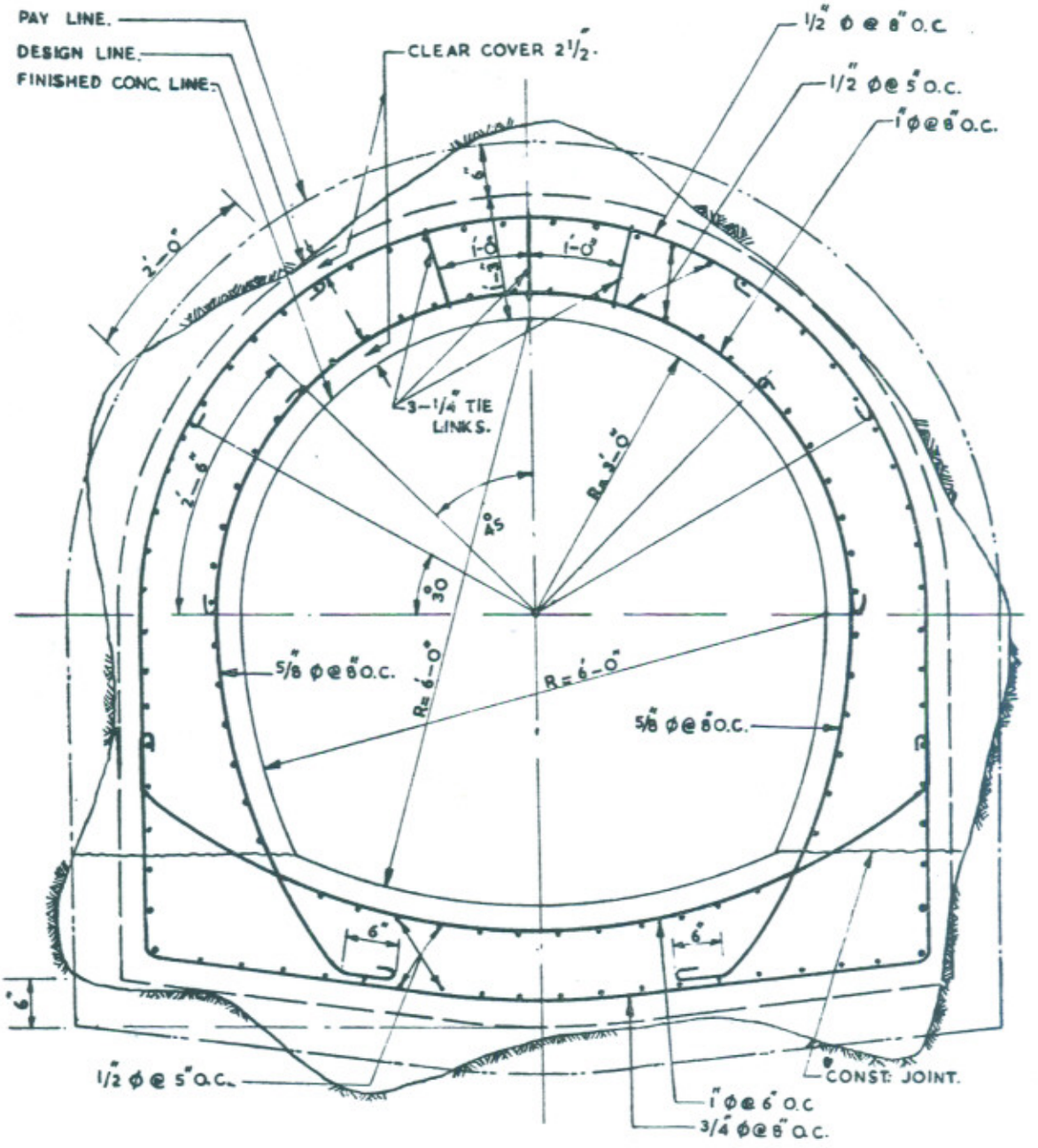


X-SECTION LINING TYPE III.

ALSO ALTERNATIVE FOR LINING TYPE II.

(FOR MODERATELY BLOCKY & SEAMY ROCK.)

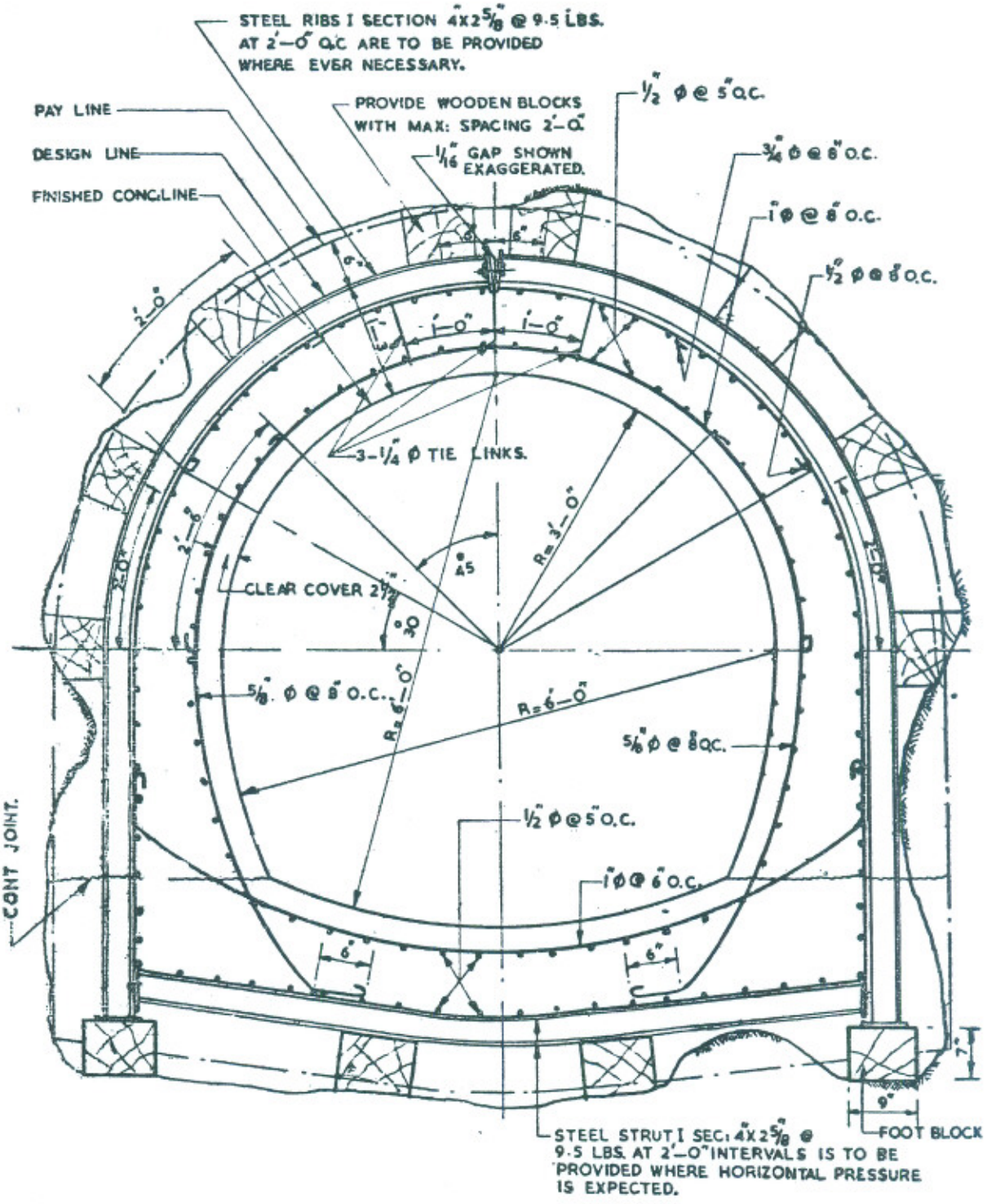
Fig. 12



X-SECTION "LINING TYPE IV"

(FOR VERY BLOCKY & SEAMY ROCK NOT REQUIRING TEMPORARY SUPPORT)

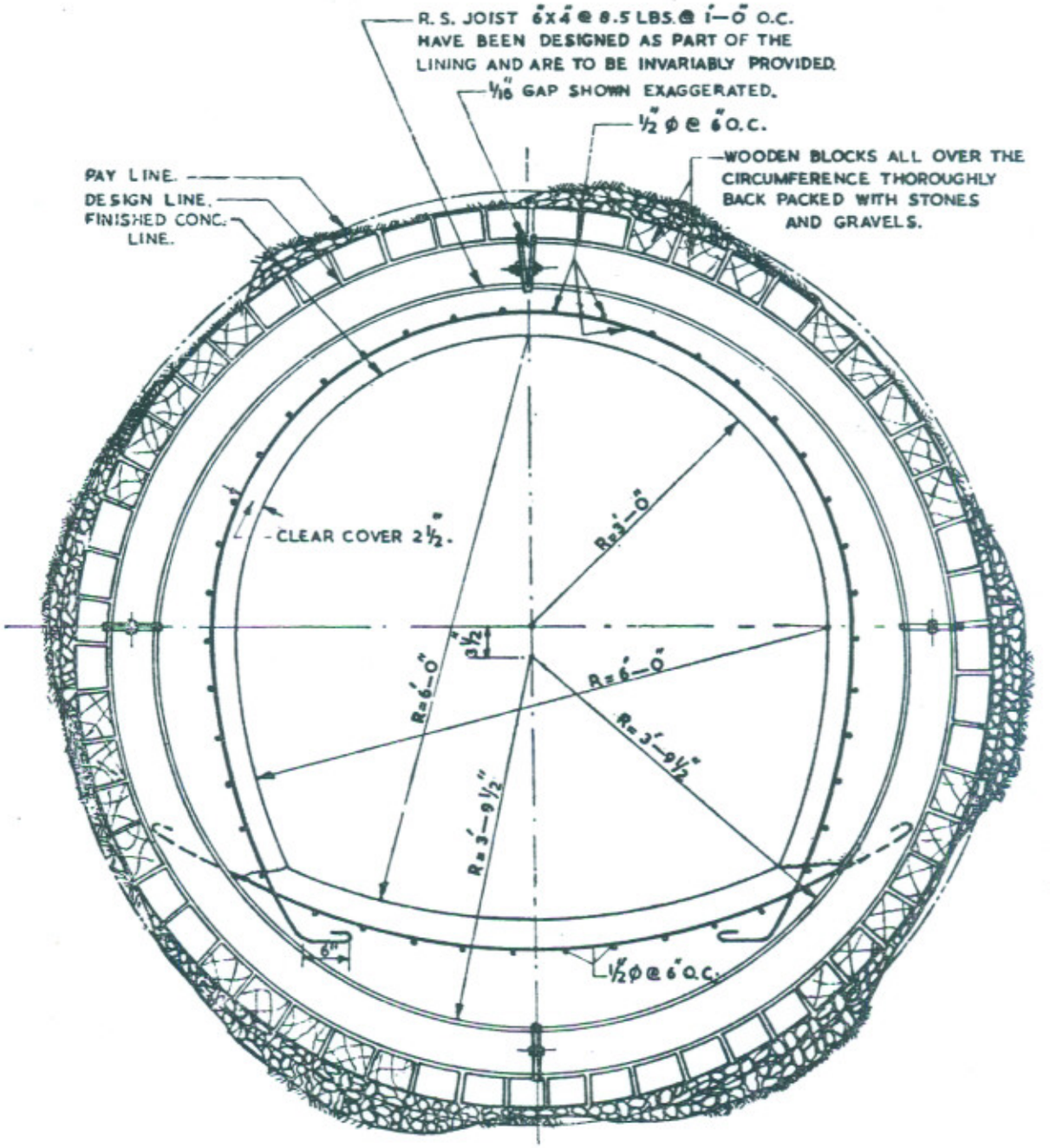
Fig. 13



X-SECTION "LINING TYPE IVA"

(FOR VERY BLOCKY & SEAMY ROCK REQUIRING TEMPORARY SUPPORTS)

FIG: 14



X—SECTION "LINING TYPE V"
(FOR SOFT GROUND OR SQUEEZING ROCK OF MODERATE DEPTH).

Fig. 15

EARTH TUNNELING

7.1. About 20% of the length of the Right Bank Tunnel passes through overburden consisting of clayey slits. Earth tunneling requires continuous supports, and full circle ribs are so perfectly adapted to meet the pressure conditions that any other arrangement is not worth consideration. Further, the full circle ribs serve as permanent reinforcements for the concrete lining. In fact the bending moment in the lining is negligible because the outer surface of the lining is cylindrical and the ribs are fully back-packed thus producing passive side pressures. The ribs are thus in pure thrust and are designed accordingly.

Vertical load intensity = 11.32 kips/ft²
Assuming outer dia. of 8 ft. the load
per linear ft. of tunnel = 8×11.34
= 90.72 kips.

Try R.S.J. 6″×4″ @ 8½ lbs.

$$\text{Comp. stress} = \frac{90.72}{2 \times A} = \frac{90.72}{2 \times 2.5}$$
$$= 18 \text{ kips/in}^2$$

This is quite safe.

The designed cross-section is shown in Fig. 15. The ribs are encased in concrete and the inner face of lining has nominal reinforcements.

Acknowledgements

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APPENDIX I

Reference Table for Expected Loads on Tunnels

Rock load H_p in feet of rock on roof of support in tunnel with width B(ft) and height H_t (ft) at depth of more than 1.5 $(B + H_t)$[1].

Rock Condition	Rock Load H_p in feet.	Remarks
1. Hard and intact	Zero	Light lining, required only if spalling or popping occurs.
2. Hard stratified or schistose[2]	0 to 0.5B	Light support.
3. Massive moderately jointed.	0 to 0.25B	Load may change erratically from point to point.
4. Moderately blocky and seamy.	0.25B to 0.35 $(B+H_t)$	No side pressure.
5. Very blocky and seamy	(0.35 to 1.10) $(B+H_t)$	Little or no side pressure.
6. Completely crushed but chemically intact.	1.10 $(B+H_t)$	Considerable side pressure. Softening effect of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs.
7. Squeezing rock, moderate depth.	(1.10 to 2.10) $(B+H_t)$	Heavy side pressure, invert struts required. Circular ribs are recommended.
8. Sequeezing rock, great depth	(2.10 to 4.50) $(B+H_t)$	
9. Swelling rock	Upto 250 ft. irrespective of value of $(B+H_t)$.	Circular ribs required. In extreme cases use yielding support.

1. The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 can be reduced by fifty per cent.

2. Some of the most common rock formations contain layers of shale. In an unweathered state, real shales are no worse than other stratified rocks. However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so-called shales may behave in the tunnel like squeezing or even swelling rock.

If a rock formation consists of a sequence of horizontal layers of sandstone or limestone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the so-called shale and rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.