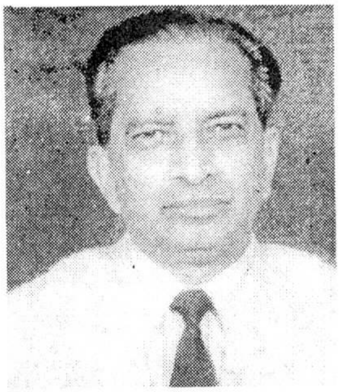
DIFFICULT FOUNDATIONS OF JOGIGHOPA BRIDGE - SOME DESIGN ASPECTS

R. L. Malik, IRSE
General Manager (Constn)
N. F. Railway, Maligaon
Guwahati-781011 (Assam)



Born in 1941, he graduated in Civil Engineering in 1962 and joined Indian Railway Service of Engineers in 1963. He has been associated with survey and construction of new lines, doublings, gauge conversion, re-girdering/rebuilding of bridges etc. on various Zonal Railways. In his present assignment, he was closely involved in construction of Rail-cum-Road bridge at Jogighopa where Pier No. 17 & 18 had special foundation problems.

Lalloo Singh, IRSE
Chief Engineer
N. C. Railway,
Allahabad (U.P.)

Born in 1956, Shri Singh is a graduate in Civil Engineering and joined India Railway Service of Engineers in 1982. For the last 15 years he has been closely associated with construction of bridges over mighty Brahmaputra at Tezpur and Jogighopa under N. F. Railway Construction Organisation.

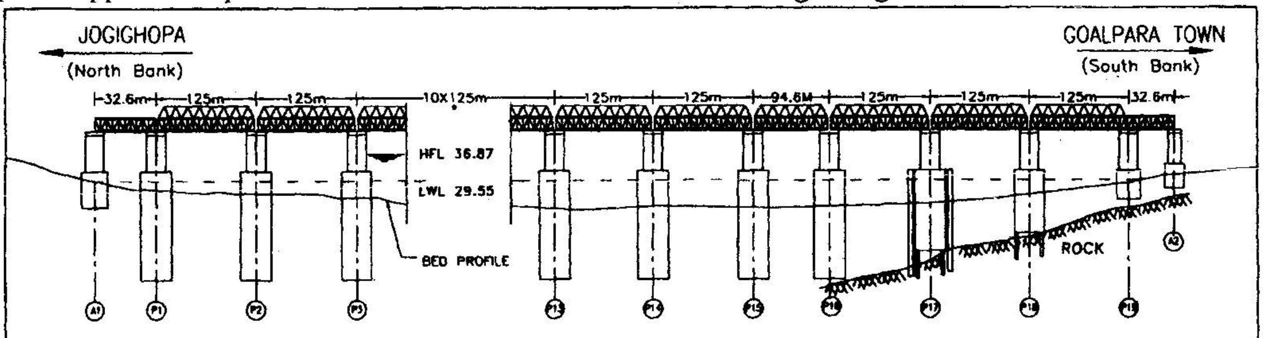
Abstract

The construction of the third bridge across the mighty river Brahmaputra was completed recently by the Indian Railways. The 2.30 Kms-long rail-cum-road bridge is situated at Jogighopa, 150 kilometres West of Guwahati, in the Bongaigoan district of the State of Assam. The work on the bridge commenced in December 1987 and it was opened to traffic on 15th April 1998. The bridge connects the existing Jogighopa Railway Station on the north bank to the Kamakhya Railway station near Guwahati by a 142 Kilometres long railway line traversing on the southern bank of Brahmaputra. The bridge also provides the road connection between the National Highway–31(B) on the north bank and the NH-37 originating from Pancharatna on the south bank of the river.

The site selected for the bridge was considered suitable in spite of problematic foundation conditions near south end of the bridge. The problem was caused by the presence of sloping rock at a depth of 45m below LWL, which is within the zone of anticipated maximum scour conditions. Two foundations, namely P-17 and P-18, had to be founded on undulating bedrock. Because of the difficult and typical foundation conditions encountered at these two locations, a special foundation system comprising of large diameter well and the anchor piles through the well steining was evolved. This paper briefly discusses the main design features for these two difficult foundations.

INTRODUCTION :

Jogighopa Bridge has a span configuration of 17 spans of 125 m each, one span of 94.60m and 2 shore spans of 32.60m each, all between the centres of the piers. The superstructure of the bridge consists of 18.50m high and 11.50m wide double Warren-type open-web steel girders. The roadway is on upper deck and the railway line is on the lower deck. All the foundations are well foundations. Abutments are supported on twin circular wells of 6.0m diameter having common well cap. 17 foundation wells are of double D type with plan dimensions of 11m x 17m. For separation of the road and rail traffic immediately beyond the main bridge, two road viaducts have been constructed. The north viaduct is 422m long and has 7 spans of 18m-RCC girders and 10 spans of 30m-PSC girders. The south viaduct is 660m long and has 7 spans of 18m-RCC girders and 18 spans of 30m-PSC girders. The substructure consists of hollow circular piers supported on pile foundations. The salient features of the bridge are given in the table below.



RAIL CUM ROAD BRIDGE AT JOGIGHOPA

Fig: 1

SALIENT FEATURES AT A GLANCE

Length of Main Bridge	:	2.284 Kms
Span configuration	:	1x32.6m+14x125m+1x94.60m+3x125m+1x32.60m.
Length of Road Viaducts	:	1.082 Kms
Span Configuration (i) North Viaduct	:	7 x 17.86m (RCC) + 10 x 29.73m (PSC)
(ii) South Viaduct	:	7 x 17.86m (RCC) + 18 x 29.73m (PSC)
Guide Bunds : (i) North Bank	:	850 m long.
(ii) South Bank	:	478 m long.

General Configuration :

Road on upper deck, Railway Line on lower deck.
Main spans – steel girders.
Double 'D' Type well foundations (11m x 17m).
Circular well foundations for P-17 and P-18 (18m dia).
Navigational Clearance 12.50 M above HFL.

Foundation Design Parameters :

Design discharge	:	90,400 cumecs.
Design discharge intensity	:	90 Cumecs/m max.
Max. velocity of flow	:	5.00 m/sec
Design HFL	:	RL (+) 36.87 m.
Seismic design HFL	:	RL (+) 35.87 m.
Low water level	:	RL (+) 29.55 m
Scour level : Maximum	:	RL (-) 16.95 m
Seismic	:	RL(-) 8.79 m
Design founding level	:	RL(-) 36.75 m
Buoyancy – well resting on sand	:	100%
Well resting on rock	:	75%
Maximum tilt	:	1 in 80
Maximum shift	:	D/40

FOUNDATION PROBLEM :

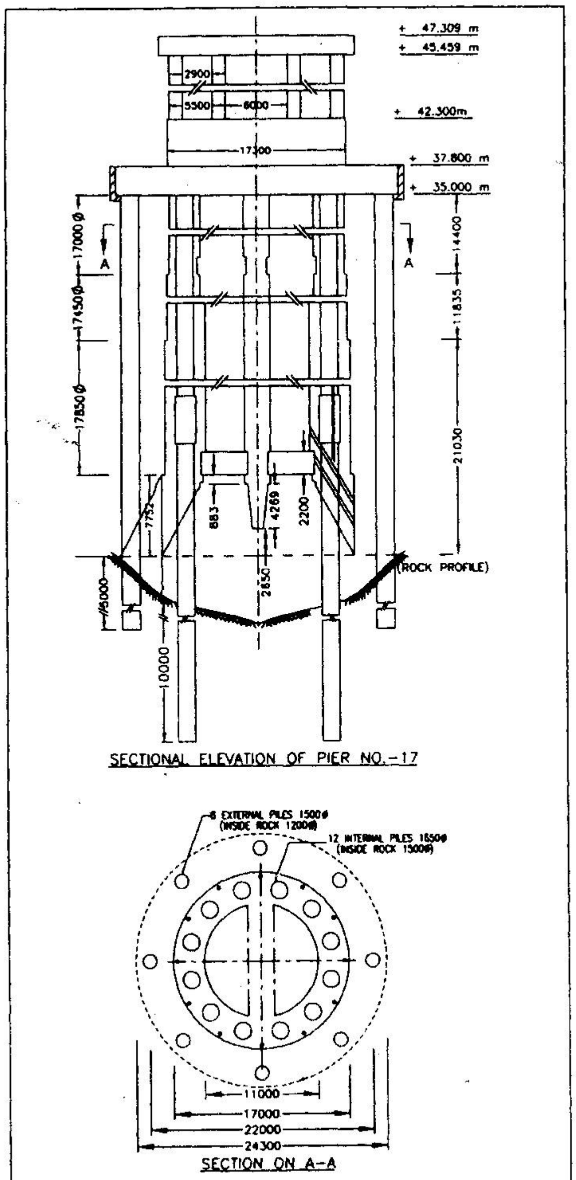


Fig:2

The normal well foundations have been sunk through the river bed material and are usually founded on the dense sand. The founding level is generally well below the deepest anticipated scour level so that the well has sufficient grip around the steining. During the worst scour conditions, the available grip around the well foundation prevents the overturning of the pier under the overturning moments caused by applied loads.

The detailed geotechnical investigations of the site conducted during the final location survey had revealed the presence of rocky strata at varying depths towards the southern side of the bridge alignment. Two foundations, namely P-17 and P-18 were affected by the presence of the undulating bed rock. The anticipated maximum scour was expected to reach upto the rock level leaving no grip around the wells. Due to the lack of lateral resistance, the foundations were unsafe under seismic conditions from stability considerations. Since the bedrock was 45 m below LWL, it was not possible to adopt pneumatic working for socketing the well into the bed rock. The design was governed by seismic forces. Since the earthquake can strike from any direction, a large size circular well was required. Since it was not possible to socket the well into the bed rock by further sinking, other means of positive anchorage of well with the bed rock was required.

The design of these foundations comprises of circular wells of 18m diameter with 12 numbers of 1.50m diameter anchor piles socketed through the well steining into the bed rock. The buoyancy effect at the rock base was taken @75% instead of usual 50%. The diameter of the wells has been reduced from 18m at the base to 17m at the well cap level. The anchoring of wells with the rock has been done with 12 nos. 1.50m diameter equally spaced reinforced concrete piles through the steining of the wells. The piles are anchored by 10m inside the rock and fully bonded with the well steining. The well has been provided with an 18m diameter solid M-30 grade concrete bottom plug at the base for transmitting the weight of the structure to the base rock. In addition to the internal piles, 8 numbers of 1.50m diameter external piles were also constructed at Pier No. 17 subsequently to strengthen it. These piles have been integrated with the well cap by a common well-pile cap. Various design aspects of the well base and internal and external piles are briefly discussed in the following sections.

Design Loads at Well Base

The dimensions of the well are shown in figure – 2. The total dead weight of various components of the foundations, substructure and the superstructure were worked out on the basis of section properties at different elevations. The weight of the superstructure including the road deck, railway track and the services is 2646t.

The experts at the University of Roorkee carried out the dynamic analysis of the structure for determining the seismic forces induced by the weight of the structure during earthquake vibrations. For the purpose of mathematical modeling, the entire pier structure was divided into 26 masses, which were lumped at appropriate levels. The pier structure has been modeled as a vertical cantilever structure free to vibrate laterally. The response spectrum for the site was developed on the basis of typical earthquakes having epicenters at different fault zones. Based on dynamic analysis, the longitudinal and transverse seismic moments and seismic shears at different levels were worked out. The vertical seismic coefficient was taken as g/16. The moments and shears at the base of the well under seismic conditions are tabulated below.

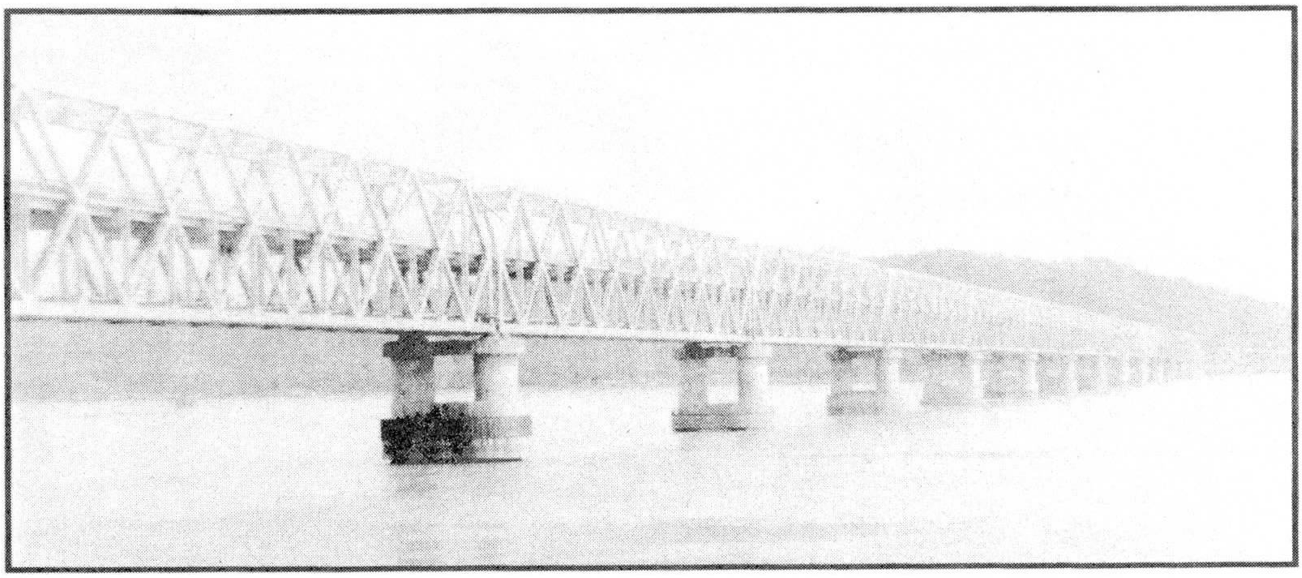


Fig:3

Design Loads at the Base of Pier No. 17

S N	Loads	Vertical Force	Along the Bridge Axis			Across the Bridge Axis		
			Force	LeverArm	Moment	Force	LeverArm	Moment
		(t)	(t)	(m)	(t-m)	(t)	(m)	(t-m)
1	Dead Loads	32355	Includes all dead loads of well, substructure and superstructure					
2	Buoyancy @75%	-9543						
3	Rly Live Loads	2034	-	-	-	-	-	-
4	Road LL @100%	366						
	@ 50%	183	Seismic case.					
5	Braking Force	-	335.3	63.192	21188	-	-	-
6	Water current	-	-	-	-	332	36.00	11952
7	Tilt & Shift	-	-	-	3742	-	-	-9710
	Total-Nonseismic	***25212***	***335.3***		***24930***	***332***		***2242***
8	Seismic-Horizontal	-	4519	-	187500	4130	-	174600
9	Seismic-Vertical	±2091	-	-	-	-	-	-
	Total-Seismic	**27120** **22938**	**4854**		**212430**	**4462**		**176842**

Analysis of the Well Base :

Normal Case : As already stated, the base of the well is 18m in diameter. The total section properties are :

Area = 254.47 m^2 Ixx = Iyy = 5153 m^4.

Maximum stress at the well base= P/A + (Mxx/Ixx). R + (Myy/Iyy).R

= (25212/254.47) ± (24930/5153) x 9.0 ± (2242/5153) x 9.0

= 99.08 ± 43.54 ± 3.92

= 146.54 t/m^2 = 14.65 kg/cm^2 - Maximum compressive stress

and 51.62 t/m^2 = 5.16 kg/cm^2 - Minimum compressive stress

Thus, no tension develops at base and the stresses are much below the permissible stress of 100 kg/cm^2 under bending compression for M-30 grade of concrete.

Seismic Case : In the seismic case, the base of the well is subjected to large moments (2,12, 442tm) in addition to the heavy vertical load of 22,938t. This combination of loads induces an eccentricity of about 9.26m. Because of the large eccentric load, the major portion of the base shall be subjected to tensile stresses. The analysis of the base as cracked RCC section is, therefore, called for. The results of the analysis of the well base under seismic condition are given in a tabulated form. The symbols used in the table are explained in the accompanying figure. The position of the Neutral Axis of the section was calculated by the process of successive iteration. To begin with, the location of the neutral axis has been assumed at the centre of the bottom plug. The sectional properties of the cracked section are then determined by standard formulae. With the load combination acting on the cracked section and the section properties thus determined, the revised location of the neutral axis is worked out and compared with the assumed location. The new location of the neutral axis is then taken as the basis for next iteration. After a few iterations, the correct location of the neutral axis is known along with the section properties of the cracked section. The stresses in the concrete and the steel are then worked out.

Axial Compression = 22938 t.

Applied Moment = $\sqrt{[(212430^2 + 2242^2)]}$ = 212442 tm.

Radius of bottom plug, R = 9.00m Modular Ratio, m_r = 16.20

Radius of internal piles, r_l = 0.75 m.

Distance of piles from centre of bottom plug R' = 7.25m and R_o= R'. Cos 15°= 7.003m.

Area of steel in each pile A_{stp}= 0.0550 m^2 (54 nos. of 36mm dia HYSD bars).

Gross area of each pile A_{cp} = 3.14/4*1.5^2=1.767 m^2.

Equivalent area of pile reinforcement = 16.2 x 0.055 = 0.891 m^2.

Determination of Neutral Axis (refer figure – 4)

The distance of the N. A. above the centre of the bottom plug (assumed), X_{na} = 2.82m.

Half the angle subtended by the Neutral Axis at the centre, $\alpha = \cos^{-1}(X_{na}/R)$ = 1.2512 rad.

Formulae for segment of circle :

$$\text{Area} = (\alpha - \sin\alpha.\cos\alpha)\, R^2;\ Xcg = Xna + [(2/3).\sin^3\alpha/(\alpha - \sin\alpha.\cos\alpha) - \cos\alpha].R$$

$$Icg = (R^4/4).[\ \alpha - \sin\alpha.\cos\alpha + 2.\sin^3\alpha.\cos\alpha - \{(16/9).\sin^6\alpha/(\alpha - \sin\alpha.\cos\alpha)\}]$$

Notations : I_{cg} = Moment of Inertia of the element about its own centroid; I_o = Moment of Inertia of the element about the centroid of the effective section; A_{eff} = Effective area of X-section after neglecting the concrete area in tension I_{eff} = Moment of Inertia of the effective section about the centroid of the effective section.

Sectional properties under load

Description	Area (m^2)	X_{cg} (m)	$A.X_{cg}$ (m^3)	I_{cg} (m^4)	$I_o = I_{cg}+A.(X_{cg}-X_o)^2$ (m^4)
1. Core concrete above N.A.	77.189	5.387	415.850	204.575	237.738
2. Pile No. 1 & 12 (steel)	1.782	7.003	12.479	0.000	9.190
3. Pile No. 2 & 11 (steel)	1.782	5.127	9.135	0.000	0.277
4. Pile No. 3 & 10 (steel)	1.782	1.876	3.344	0.000	14.531
5. Pile No. 4 & 9 (steel)	1.782	-1.876	-3.344	0.000	77.822
6. Pile N. 5 & 8 (steel)	1.782	-5.127	-9.135	0.000	173.193
7. Pile NO. 6 & 7 (steel)	1.782	-7.003	-12.479	0.000	245.398
$A_{eff} = \Sigma A$ =>	87.881		415.850		$I_{eff} = \Sigma I_o$ = 758.149

Distance of centroid of the effective section from centre, $X_o = \sum (A.X_{cg}) / \sum A = 4.732$ m.
Applied Moment about the centroid of the effective section, $M' = M - P.X_o = 103904.3$ t-m.
Distance of N.A. from centroid of the effective section, $Y_c = (P.I_{eff})/(M'.A_{eff}) = 1.904$ m.
Distance of Neutral Axis above the centre of circular section, $Y_o = X_o - Y_c = 2.828$ m.

(The distance of neutral axis, Y_o, as calculated above matches with the distance of the neutral axis, X_{na}, assumed above, since both are equal, the assumed position of neutral axis is correct.)

Calculation of Stresses in the Section

Max. comp. Stress in pile concrete = $\sigma_c = (P/A_{eff}) + (M'/Ieff).(R-Xo) = 845.93$ t/m^2
Maximum tensile stress in pile steel, $\sigma_{st} = m_r\{M'/I_{eff}).(R_o+X_o)-(P/A_{eff})\} = 21825.8$ t/m^2

Pile Forces

Max. comp. Stress in pile concrete $\sigma_c' = 845.93\ (7.003-2.828)/(9-2.828) = 572.22$ t/m^2
Maximum compression in the pile = $\sigma_c'.\{A_{cp} + (16.2-1).A_{stp}\}$ = 1489.6 t.
Maximum tension (uplift) in the outer most pile = 21825.8 x 0.055 = 1200.4 t.

The maximum tension in the piles is 1200t. Depending upon the earthquake direction, any pile can be subjected to the uplift of 1200t and hence all the piles have to be designed for the same capacity. The bottom plug and the piles are designed with M-30 grade concrete for which permissible stress under seismic condition is 1.333 x 100 = 133.3 kg/cm^2 under bending compression and 1.333 x 85 = 113.3 kg/cm^2 under direct compression. The maximum compressive stress in the bottom plug as well as pile concrete is well below these values and hence safe. The design of the anchor piles for the uplift capacity is given below :

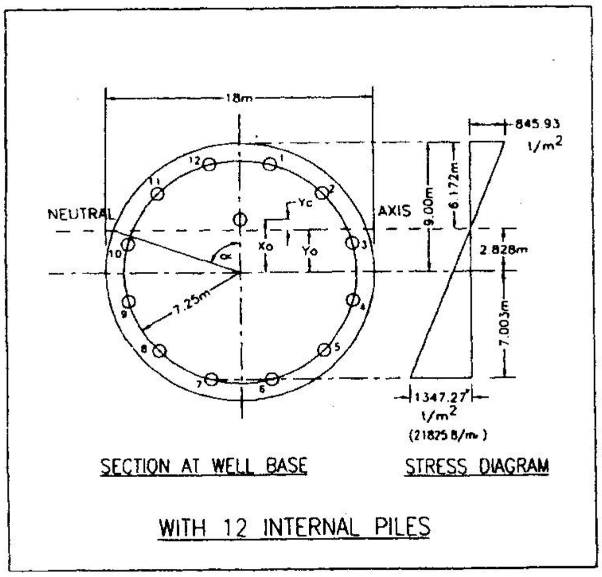


Fig:4

DESIGN OF ANCHOR PILES :

The piles have been provided essentially for resisting the tension developed at the base of the well caused due to overturning moments and resultant uplift under seismic conditions. The base of the well is treated as the solid circular section with each pile acting as steel reinforcement resisting the tension. The socket length inside the rock and the bond length of pile within the well steining have to provide adequate safety against the uplift. The brief computations are given below.

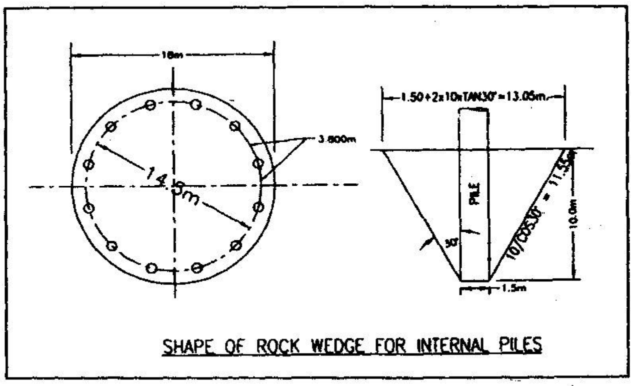


Fig:5

Pile Reinforcement

Maximum tension developed in the pile = 1200.4 t.
Use 36mm diameter HYSD bars. Area of each bar = 10.18 cm^2.
No. of bars per pile = 1200.4 x 1000/(1900 x 10.18 x 1.33) = 46.66 nos.
54 nos. of 36mm dia HYSD bars have been provided in each pile. This ensures the pile capacity of about 1392t in tension.

Pile Anchorage in the Rock (see figure – 5)

Max. uplift capacity of the pile = (1900 x 1.333) x 54 x 10.18/1000
= 1392.3 t (actual tension 1200.4t)
Assume piles in a row, the contributing length will be π x 14.5/12 = 3.80m
Anchorage length in the rock = 10m.
Width of the wedge at the top of the rock = 2 x 10 tan 30° + 1.5 = 13.05m
Buoyant unit weight of the rock @75% buoyancy = 2.30 – 0.75 = 1.55 t per cum.
Unconfined compressive strength of the rock = 253.4 kg/cm^2 = 2534 t/m^2.
Volume of the wedge = (1.50 + 13.05) x 10.00 x 3.80/2 = 276.45 cum
Buoyant weight of the rock wedge = 276.45 x (2.3 – 0.75) = 428.50 t
Shear stress = 0.01 x UCS = 0.01 x 2534 t/m^2 = 25.34 t/m^2.
Surface area = 3.80 x (10.0/Cos30°) x 2 = 3.80 x 11.55 x 2 = 87.88 m^2.
Shear resistance = 87.88 x 25.34 = 2224.3 t.
Vertical component of shear resistance = 2224.3 x Cos30° = 1926.3 t.
Total resistance = 428.5 + 1926.3 = 2352.1 t.
Load factor available = 2353.1/1392.3 = 1.90 > 1.5 OK

In the above calculations, submerged weight of soil above the rock-top upto the scour level has not been considered. This will provide additional safety against uplift.

Anchorage in the Well Steining

All the piles are fully bonded with well in the entire length of the steining. Out of 54 nos. of 36mm dia bars, 36 nos., have been curtailed after 10m of bond length in the steining. Beyond this only 18 nos. of 36 mm dia bars have been retained, which have been continued right upto the well cap.

EXTERNAL PILES AROUND P-17

This is the pier where the work of bottom plug and piling was done first. During the construction of the piles, it was found that the diameter of the bottom plug possibly did not cover the entire base of the well. This was attributable to improper cleaning of the dredge hole in the under water conditions. The jet grout columns surrounding the well base possibly did not seal the entire periphery of the well. This permitted the inflow of sand in the dredge hole under the high hydrostatic pressure of 45m during cleaning operations. The contact area of bottom plug with bedrock below was doubtful in the outer peripheral zone. After discussions with the experts, it was decided that the effective area of the truncated bottom plug of 14.0m diameter i.e. the area enclosed by the internal piles only should be considered in design. The integrity of the bottom plug in outer portion was doubtful and hence the foundations needed further strengthening. It was decided to construct 8 nos. of external piles around P-17 to strengthen this pier. 8 nos. of external piles of 1.50m diameter have been constructed outside the well in a symmetrical pattern and integrated with the foundation well by a common well cap. A socket length of 5m has been provided for these piles. The design of these external piles is covered under this section.

DESIGN OF EXTERNAL PILES:

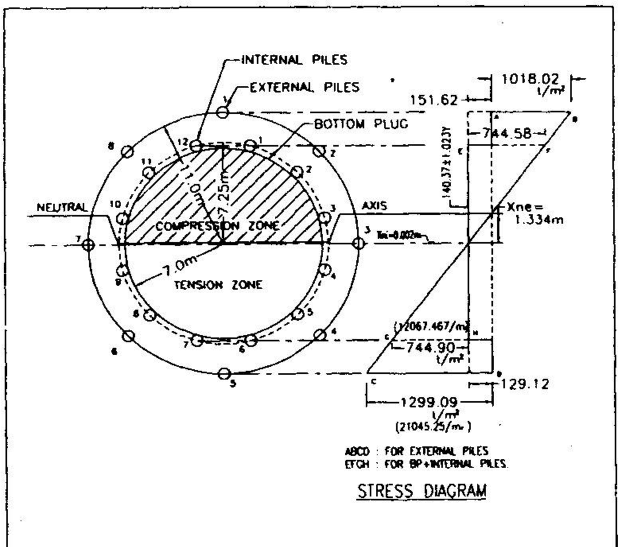
Basic of Analysis

For the design of the external piles, the fresh dynamic analysis of the pier has not been done. The pseudo-static seismic coefficients obtained from the earlier dynamic analysis were used for calculating the additional forces and moments induced at the well base due to the additional weight of the external piles and the extended well cap. The critical combination of the design loads is given below:

- Before construction of external piles i.e. all loads excluding the dead load of external piles and the extended portion of the well cap, all live loads and the seismic loads. P = 25,993 t. M = 2,773 tm.
- After the construction of external piles, i.e. full dead loads, live loads and seismic loads. P = 32,467 t. M = 2,42,057 tm.

The foundation well was found to be safe under the normal loading conditions without external piles. The external piles were required only under the seismic loading condition. The analysis of the combined well base has been done corresponding to two stages of loading. In the first stage of loading, the section consists of the well with the truncated bottom plug of 14m effective diameter and the full section of the 12 nos of internal piles. The loads operating at this stage are the full dead load of the well, piles, substructure and superstructure complete. At this stage full section is effective since the moments are small. It does not crack and therefore total section properties are applicable.

The second stage of loading will consist of the dead loads of external piles and the extended portion of the well cap, the live loads and seismic forces. At this stage, the external piles shall also be effective. The incremental effect of the second stage loads will be resisted by the composite section comprising of truncated bottom plug, internal piles and the external piles. Only the uncracked concrete along with the steel in the internal and external piles shall be effective in resisting the applied loads. Because of the composite action and the two stages of loading, the combined section will have two neutral axes, one for the initial section consisting of bottom plug and the internal piles and the other for the group of external piles. Both the stages of loading have been superimposed to obtain the final stress diagram. The position of the neutral axes has been found by trial and error. The compatibility of internal and external forces acting on the combined section of bottom plug and piles has been checked. The forces in the piles have been worked out from the final stress diagram.



COMBINED SECTION WITH 8 EXTERNAL PILES.

Fig:6

The pile forces in different internal and external piles are tabulated below :

Pile No.	Internal Piles	External Piles
1	1938.24	2650.05
2	1418.77	1766.72
3	519.04	-125.22
4	-177.95	-855.14
5	-485.91	-1157.49
6	-663.71	-855.14
7	-663.71	-125.22
8	-485.91	1766.72
9	-177.95	-
10	519.04	-
11	1418.77	-
12	1938.24	-

Design of External Piles

All the 8 external piles are of 1.50m diameter and are placed in a symmetrical pattern at 45° apart. These are reinforced by 54 nos. of 36mm diameter HYSD bars in a pattern similar to that of internal piles. The free standing length of the piles is 50m in the maximum anticipated scour condition. The effective length has been taken as 0.65L in accordance with Appendix-D of IS:456-1978.

The lateral shear acting at the bottom of the well cap has been assumed to be shared by the well and the group of external piles by frame action. The piles are fixed both at bottom and top. The well has been assumed as fixed at the bottom and free at the top. This is a conservative assumption for calculating shear in piles and gives higher shear force and consequently higher bending moment in the piles. The external piles have been designed for the tensile force of 1157 mt, compression of 2650 mt and the bending moment induced due to horizontal shear in piles. The socket length of 5.0m in the rock has been provided which is adequate for resisting the tension in the piles.

CONCLUSION :

Some of the important design features of the difficult well foundation P-17 and P-18 have been briefly discussed in this paper. These special design features were necessitated due to peculiar and difficult foundation conditions at site. The work has already been completed and the bridge has been opened to traffic.

Leere Seite
Blank page
Page vide