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SEISMIC ANALYSIS OF AN AQUEDUCT FOUNDED ON PILES

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ABSTRACT

The aqueducts are vulnerable to earthquake damage because of several reasons like excessive displacement in bearings, cracking of substructure and superstructure and settlement of foundation. In this study, the detailed dynamic analysis of an aqueduct considering soil-structure interaction has been carried out to evaluate the earthquake forces and displacements in the longitudinal and transverse directions. Resultant hydrodynamic pressure on the walls of trough has been calculated. Seismic design considerations for safe performance of the aqueduct in an earthquake have also been discussed.

KEYWORDS

Earthquake analysis ; aqueduct ; pier ; bearing ; soil springs ; hydrodynamic pressure.

THE STRUCTURE

The aqueduct is a 548.10 m long structure consisting of 27 spans of length 20.30 m each resting on piers and pile foundations. Fig. 1 shows the key elevation of piers with well foundation. There are 44 piles under each pier. The piles are taken into sound rock and anchored in a depth of 1.5 m. It lies in Seismic Zone III of India. The maximum discharge of the canal is 1129 cumecs at its head. The water is carried in three independent trough units consisting of three barrels each of inner size 7.0m X 5.43m. The piers are of wall type of size 2.5m X 81.3m. The bearings used are steel rocker cum PTFE type with stopper blocks at the ends. There are 24 number of bearings on a single pier in two rows. The foundation rock consists of gneiss, schist and granite. The overburden consists of soil/rock deposits.

ASSUMPTIONS OF ANALYSIS

Following assumptions have been made in the analysis:

(i) Live Load: For analysis in longitudinal direction, no live load is considered on the aqueduct. In transverse direction, 25% of design live load (without impact) is considered on the aqueduct for the purpose of dynamic analysis.

(ii) The Added Mass of Water: Some portion of water surrounding the pier is assumed to vibrate along with the pier. The mass of water in the enveloping cylinder for the submerged part of the pier is, therefore, considered to be attached with the pier for dynamic analysis. The enveloping cylinder is made as per the guide lines of Indian standard IS:1893-1984. The size of cylinder is different in longitudinal and transverse directions.

(iii) Inertia of Water in the Trough: The inertia of flowing water in the trough is considered in the transverse direction and not in the longitudinal direction. The hydrodynamic pressure of the water in the trough in the transverse direction on the walls of the trough is also considered.

(iv) Foundation Soil Springs: The embedded part of the structure (soil-pile system) in the foundation strata is replaced by linear and rotational springs.

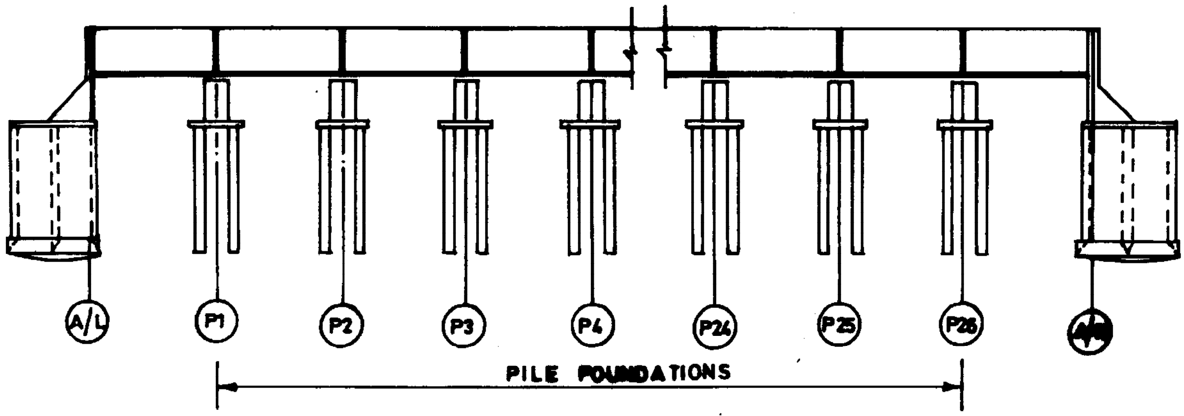

Fig. 1. Key elevation of piers with pile foundation

DESIGN EARTHQUAKES

Two types of design earthquakes have been considered, namely, Design Basis Earthquake (DBE) and Maximum Credible Earthquake (MCE) and these are shown in Fig. 2. The aqueduct has been analyzed under DBE and MCE conditions for various components depending upon their relative importance. For example, the piers have been analyzed for DBE while the bearings have been analyzed for MCE as these are more critical for seismic safety of the aqueduct. The foundation should be checked for MCE. The internal walls of water carrying trough should be checked for hydrodynamic pressure including sloshing effect of water in transverse direction.

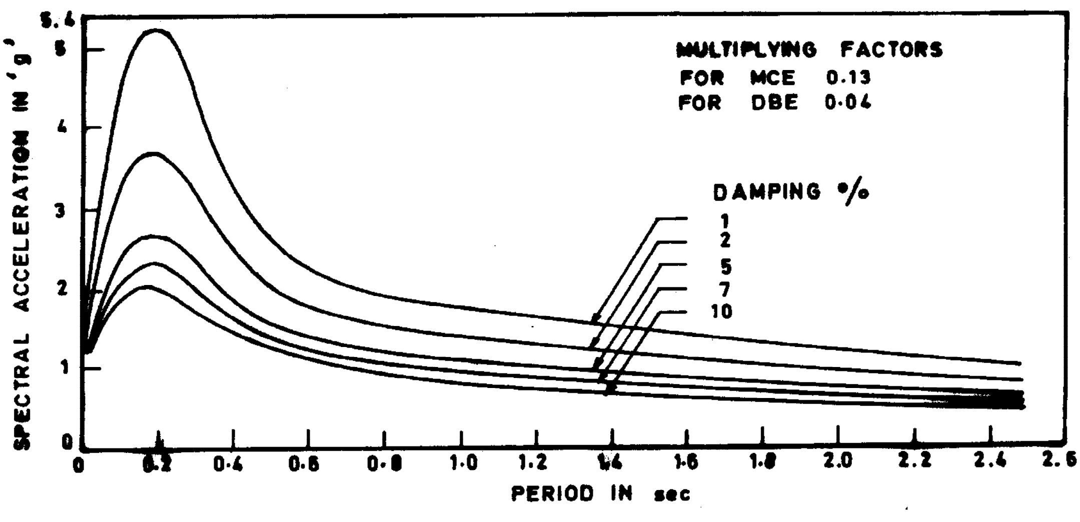

Fig. 2. Acceleration response spectra for the aquiduct.

METHODOLOGY

The aqueduct structure is idealized into a lumped mass mathematical model (Fig 3). The foundation-structure interaction is considered by evaluating equivalent spring stiffness of soil surrounding piles in the form of translational and rotational springs. Two mathematical models are used, one for analysis in the longitudinal direction, that is, in the direction of flow of canal, and another for transverse direction, that is, in the direction perpendicular to the flow of canal.

In the longitudinal direction, the weight of the trough is lumped at the top of bearing. In the transverse direction, the self-weight of the trough and weight of water are lumped at centre of gravity of water mass, the 25% of design live load is lumped at its centre of gravity. The bearing element is separately represented by an element above the top of pier. The soil springs, representing pile-soil system are placed at the top of pile cap.

The natural frequencies and associated mode shapes are determined by transfer function method. The response method has been used to find the displacements and forces in the structure. The damping values used in the analysis are 7% and 10% for DBE and MCE conditions, respectively.

Six typical piers have been selected for the analysis. The rock properties underneath these piers cover a wide range of variations, so that rock properties of the piers which are not analyzed are well covered within this range.

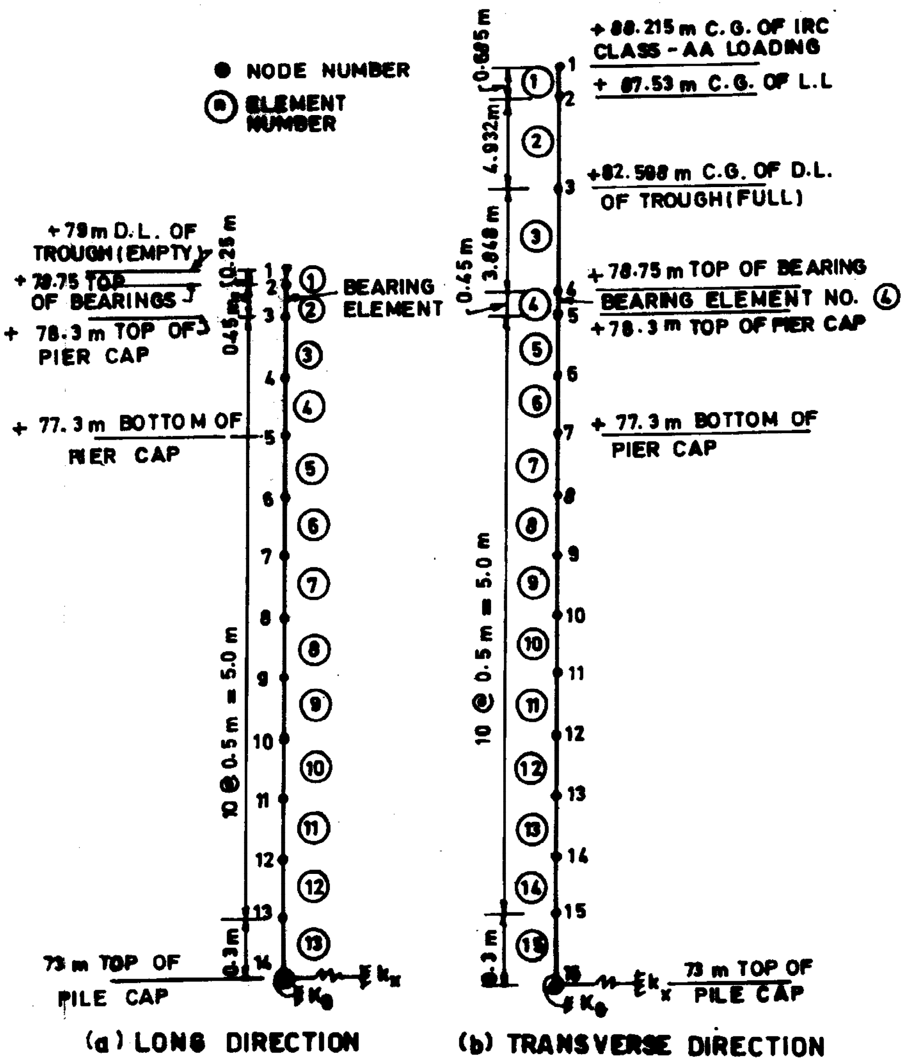

Fig. 3. Mathematical models of a typical pier.

Transfer of Shear Through Bearing

The shear force to be transferred through PTFE bearing element is restricted to 5% of total vertical load above bearings, equal to 719 t in both longitudinal and transverse directions of the aqueduct. The control of transfer of shear force through bearing is achieved in dynamic analysis by varying the stiffness of bearing element by trial such that it transfers shear force nearly equal to 5% of weight above, i.e., 719 t.

FOUNDATION SOIL SPRINGS

Elastic Properties of Foundation Rock/Soil:

The aqueduct is resting over piles which are anchored inside the rock to a depth of 1.5 m. The soil surrounding the pile is erodable. Three rock types are encountered in the foundation, namely, Gneiss, Schist, and Granite. The modulus of elasticity (E)and Poisson's ratio (υ) of these rocks and of the overburden as assumed in the analysis are given in Table 1. These values are used in finit element analysis of soil-pile system for working out spring constants of soil springs.

Table 1. Description of Cases and Material properties

		Modulus of Elasticity (E) in t/m^2			
Case No.	Pier No.	Overburden υ =0.30	Schist υ =0.33	Granite υ =0.14	Gneiss υ =0.15
1	P1	2.0-8.0	0.79	-	-
2	P1	2.0-8.0	0.15	-	-
3	P6	2.0-8.0	0.79	2.53	-
4	P6	2.0-8.0	2.15	2.53	-
5	P11	2.0-8.0	-	0.66	-
6	P11	2.0-8.0	-	4.40	-
7	P16	2.0-8.0	-	-	1.90
8	P21	2.0-8.0	0.79	-	1.90
9	P21	2.0-8.0	2.15	-	1.90
10	P25	2.0-8.0	0.79	-	-
11	P25	2.0-8.0	2.15	-	-

Idealization of Soil-Pile System

In order to take into account the soil-structure interaction, the elasticity of piles surrounding soil and rock at the base is replaced by equivalent soil springs. This replacement consists of one linear and one rotational spring at the top of pile cap. As the overburden is erodable, two situations of surrounding soil are considered: Case-A when overburden is present and Case-B when the overburden is not present. The piles in the longitudinal and transverse directions are replaced by equivalent sheet of width calculated on the basis of moment of inertia of piles.

Spring Constants of Soil-Pile System

In order to find spring constants of soil-pile system, a static finite element plane stress analysis is carried out for a horizontal load P and moment M. The spring constants are worked out from the computed deflections as follows:

$$K_x = \frac{P}{\delta} \text{ and } K_\theta = \frac{M}{\theta}$$

where δ and θ are displacements and rotations, respectively, and K_x and K_θ are the stiffnesses of linear and rotational soil springs. Two types of finite element meshes, one for longitudinal and other for transverse direction, are employed for determining spring constants.

RESULTS

Based on the dynamic analysis carried out for six piers of the aqueduct, the following are the results:

Time Periods:

The natural time periods of the piers in longitudinal and transverse directions are given below in Table 2.

Table 2. Time Periods of Piers

Case No	Pier . No	Longitudinal Direction			Transverse Direction		
		Mode 1	Mode 2	Mode 3	Mode 1	Mode 2	Mode 3
1A	P1	0.3569	0.1626	0.0415	0.9107	0.2958	0.1471
1B	P1	0.3626	0.1674	0.0417	0.9108	0.2959	0.1472
2A	P1	0.3510	0.1570	0.0404	0.8901	0.2940	0.1332
3A	P6	0.4575	0.2097	0.0468	0.8961	0.2931	0.1391
4A	P6	0.4521	0.2084	0.0460	0.8918	0.2926	0.1360
5A	P11	0.5646	0.2649	0.0510	0.9269	0.2968	0.1572
5B	P11	0.9218	0.4477	0.0547	0.9279	0.2969	0.1577
6A	P11	0.5495	0.2577	0.0482	0.8905	0.2933	0.1341
7A	P16	0.5480	0.2569	0.0499	0.8991	0.2942	0.1403
8A	P21	0.3935	0.1768	0.0405	0.9148	0.2948	0.1513
8B	P21	0.4748	0.2150	0.0421	0.9155	0.2949	0.1517
9A	P21	0.3871	0.1754	0.0398	0.8939	0.2928	0.1376
10A	P25	0.3604	0.1651	0.0423	0.9157	0.2971	0.1493
10B	P25	0.4701	0.2175	0.0457	0.9160	0.2971	0.1495
11A	P25	0.3626	0.1681	0.0417	0.8941	0.2949	0.1349

A - With Overburden B - Without Overburden

It can be observed that the periods in transverse direction are large as compared to those in longitudinal direction. The fundamental period ranges between 0.357 sec and 0.565 sec in the longitudinal direction, and between 0.896 sec and 0.927 sec in the transverse direction. The overburden is found to produce change in time periods in longitudinal direction while such effect is small in transverse direction.

Maximum Deflection, Shear force and Bending moment

The maximum values of the response obtained after analyzing the piers are given in Table 3.

The maximum values of deflection considering out of phase motion of pier has been worked out on the basis of square root of square of displacements of two adjoining piers and are tabulated in Table 3 for different conditions. It may be noted that in the transverse direction under MCE condition, the maximum shear force could not be restricted to 5% of weight of superstructure. The values of forces thus obtained are on the higher side.

Table 3. Maximum Values of Forces and Displacements

S. N.	Item	Long. dir.(*)		Trans. dir(**)	
		DBE	MCE	DBE	MCE
1	Maximum displacement at top of pier (cm)	0.164	0.365	0.286	0.790
2	Max. displ. considering out of phase motion of pier (cm)	0.232	0.516	0.404	1.117
3	Max. shear force to be resisted by bearing (t)	718.5	718.0	718.6	851.8
4	Max. bending moment in pier cap (t-m)	1232	1215	5640	14639
5	Max. shear force at the base of pier (t)	1150	2780	699	1314
6	Max. bending moment at the base of pier (t-m)	5211	7535	34637	75869
7	Seismic coefficient to be taken for design of superstructure	0.099	0.099	0.050	0.059

* For Pier P25 Case 10A ** For Pier P11 Case 5B

Equivalent Seismic Coefficient

Equivalent horizontal seismic coefficients (α_h) for design are obtained from α_h= V/W, where V is the shear at the level under consideration and W is the total load coming above that level. These values have been given in Table 3 at the top of the pier which can be adopted for design of superstructure and working out hydrodynamic pressure in the trough in transverse direction.

Hydrodynamic Pressure on Aqueduct Walls

Hydrodynamic pressures on the walls of the aqueduct have been calculated as per the Code IS:1893-1984. For a trough, the impulsive pressure is computed to be 1.324 t/m length of trough acting at a height of 1.969 m from the base of trough. The convective pressure is 0.060 t/m length of trough acting at a height of 3.414 m from base of trough. The maximum amplitude of sloshing is 1.34 cm.

CONCLUSIONS

Seismic analysis of an aqueduct has been carried out in longitudinal and transverse directions subjected to postulated earthquake. The flexibility of foundation-soil has been considered. The hydrodynamic pressure on the walls of trough including sloshing effect of water under transverse component of earthquake motion have been worked out. The design of substructure should be checked for the maximum forces computed. The provision for out of phase displacements in bearings and water seals should be made corresponding to MCE condition.

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