STUDY OF COMPARISON OF SOIL EFFECTS ON SEISMIC RESPONSE OF BRIDGE DUE TO MODIFICATION OF SEISMIC CODE SPECTRA

A DISSERTATION

submitted in partial fulfilment of the
requirements for the award of the degree
of
MASTER OF ENGINEERING
in
EARTHQUAKE ENGINEERING
(With Specialization in Soil Dynamics)

By

SACHIN JAISWAL

DEPARTMENT OF EARTHQUAKE ENGINEERING
UNIVERSITY OF ROORKEE
ROORKEE-247 667 (INDIA)

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CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation titled **"Study of comparison of soil effects on seismic response of bridge due to modification of seismic code spectra"** , in partial fullfilment of the requirements for the award of degree of Master of engineering in the Earthquake engineering with specialisation in soil dynamics , submitted to the department of earthquake engineering, University of Roorkee Roorkee is the record of my own work carried out during the period from October 1998 to March 1999 under the supervision of Dr. S.K .Thakkar, Professor, Department of Earthquake engineering, University of roorkee

The matter embodied in this dissertation has not been submitted for the award of any other degree.

Dated 23/3/99

Place:Roorkee

Sachin Jaiswal

Sachin Jaiswal

This is to certify that the above statements made by the candidate are correct to the best of my knowledge.

S.K. Thakkar
23.3.99

Dr. S.K.Thakkar

Professor

Department of Earthquake Engineering

University of Roorkee

Roorkee,

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Sachin Jaiswal

(SACHIN JAISWAL)

ABSTRACT

The present dissertation work deals with the comparative study of existing IS:1893-1984 Code Spectra, Rock and soil spectra of revised draft code and site spectra. Three bridges have been considered for this purpose. These bridges are analysed using the transfer matrix method/stiffness matrix method for the seismic responses (Bending Moment, Shear Force and Horizontal Displacement).

Dhaleswari and Gandhak bridge have been analysed for different shear wave velocities of soil and also for fixed base condition. Soil-structure interaction effect has been considered by coupled Beredugo-Novak Spring to take into account the modification on the response of the structure due to the soil surrounding the foundation. Two embedment depths have been considered for these two bridges. Ringhal bridge has been analysed for fixed based condition using four different spectras. The damping is considered as 5% of critical for all the three bridges. It has been observed that revised code spectra gives response of bridges which could be more than 50% as compared to existing seismic code for same zone and same soil conditions.

CONTENTS

INTRODUCTION

Bridges are essential links in the transportation lifeline system of highway and railway networks. They are required to remain in service during and after the occurrence of earthquake. Their damage during an earthquake hampers the relief work operation. Therefore the safety of important bridges in seismic region is a necessary consideration for post earthquake importance.

Bridges situated in highly seismic zones are most likely to be subjected to strong earthquake motion several times during their life time. Failure of the bridge substructure is one of the most common and important causes of damage or collapse of the bridge. Therefore, design and construction of the bridge substructure on the basis of detailed dynamic analysis is of paramount importance.

The response of a structure resting on soil or embedded when subject to dynamic loading is affected by properties of soil beneath and around the structure. This response is greatly influenced by the dynamic soil structure interaction. Therefore, the effect of dynamic interaction between embedded bridge substructure and the surrounding soil is to be thoroughly investigated for safe and economic design.

The first seismic design code in India was published in 1962 (IS:1893-1962); the code has since revised in 1966, 1970, 1975 and 1984.

As a result of R&D work, experience of using codes, additional seismic data collected in India and further knowledge this code has been recently revised. The fifth revision has been recently made and is available in draft form.

In the present study three bridges have been analysed using existing IS:1893-1987 code, revised draft code and using site spectras. Two spectras i.e. rock spectra and soil spectra of revised code have been used in the study. The bridges are analysed and the responses are compared using different spectras.

OBJECTIVES OF THE STUDY

Objectives of the study are as follows :

(i) To study the dynamic response of bridge substructure using existing code and revised draft code.

(ii) To study the effect of depth of embedment and the shear wave velocities on the dynamic response of bridge substructure.

(iii) To study the comparison of response of bridge substructure using site spectra and spectra given in revised draft code.

LITERATURE REVIEW

2.1 GENERAL

The dynamic inter-relationship between the response of the structure and the characteristics of its foundation medium is referred to as the 'Interaction effect'. The soil-structure interaction means the dynamic interaction between the response of the structure and the surrounding soil around and below the bridge substructure.

Soil structure interaction analysis is a coupled problem involving a structure - foundation system resting on soil which is usually semi-infinite and characteristics of the surrounding soil are nonlinear under dynamic loading condition. Therefore a complete soil structure interaction analysis must be capable of handling these geometric characteristics and it should also reflect the true mechanical behaviour.

Conceptually, the soil-foundation structure system can be visualised as three subsystem; structure, foundation and soil, two interaction interfaces viz. structure foundation interface and foundation soil interfaces and a soil boundary.

The substructure of major bridges are generally founded on deep well foundations resting on hard stratum. The wells are embedded in soil of varying stiffness. The vibratory response of substructure of earthquake motion depends on stiffness and mass distribution of substructure. The stiffness of embedded well foundation has significant role on the dynamic response of structure. The wells as such behave like rigid bodies and are massive. The resistance to lateral loads and moment in well foundation becomes available from surrounding soil and base soil.

For analytical computations of dynamic response, the lateral and base resistance of soil can be represented by springs which are called 'Soil springs'. This is merely done to make simplified mathematical model. The stiffness of soil springs depends upon the type of soil and soil properties like shear modulus, poisson's ratio and soil modulus.

The soil springs replace the rigid well, surrounding soil and base soil. The representation is usually made by linear and rotational spring. The replacement of well and soil by springs enables the consideration of soil structure interaction effect on the dynamic response. The soil structure interaction effect implies the consideration of :

(i) Foundation elasticity - soil springs

(ii) Modification in ground motion due to soil layering and

(iii) dissipation of energy in foundation by radiation damping

There are two methods commonly employed for working out soil springs :

(i) Modulus of subgrade reaction method

(ii) Soil springs based on elastic half space formulations - such as Berdugo-Novak springs.

2.2 MODULUS OF SUBGRADE REACTION METHOD

Modulus of subgrade reaction method for obtaining the soil springs is evaluated by Terzaghi in 1955. For cohesionless soil and normally consolidated clays, the soil modulus value increases linearly with depth. For pre-consolidated clays, the soil stiffness remain constant along the horizontal depth and is based upon a basic factor K_{si} to which the vertical and horizontal subgrade reaction are related. The portion of the well below scour level has been considered as a rigid body, subjected to tilting about the base. The elastic resistance of the soil spring is considered at scour level.

Linear spring (K_t) and rotational spring (K_r) replace the embedded portion of the well as shown in Fig.1. If the top of the embedded part of the well is subjected to moment M_o and shear V_o, then we have

$M_o = C\,V_o$.

If θ_o = rotation of well due to M_o and V_o;

Δ_o = deflection at scour level due to M_o and V_o;

θ_v = slope of the well due to $V_o = 1$;

Δ_v = deflection of the well at scour level due to $V_o = 1$;

θ_M = slope of well due to $M_o = 1$;

Δ_M = deflection of the well at scour level due to $M_o = 1$

Then

$$\theta_o = \theta_v V_o + \theta_M M_o = (\theta_v/C + \theta_M)\, M_o$$

$$\Delta_o = \Delta_v V_o + \Delta_M M_o = (\Delta_v + C\,\Delta_M)\, V_o$$

Moment and shear force at c.g. of the embedded portion of the well

$$M = M_o + V_o \cdot \frac{H}{2}$$

$$V = V_o$$

Now transitional spring constant at c.g. of the embedded portion of the well

$$K_t = \frac{V_o}{\Delta} = \frac{V_o}{\Delta_o/2} = \frac{2V_o}{\Delta_o} = \frac{2V_o}{(\Delta_v + C\,\Delta_M)V_o}$$

Rotational spring constant at c.g. of the embedded portion of the well

$$K_\theta = \frac{M}{\theta} = \frac{M_o + V_o \cdot \frac{H}{2}}{\theta_o}$$

$$= \frac{M_o + \frac{M_o}{C}\,\frac{H}{2}}{\theta_o} = (1 + \frac{H}{2C})\,\frac{M_o}{\theta_o}$$

$$= \frac{(2C + H)}{2(\theta_v + C\,\theta_M)}$$

(a) Well in sand

Net resisting moment about base

$M = D_1\,\Delta$

Where $D_1 = \eta_h H^3/12 + (2/3)\,K_v r^2/H + \mu\eta_h H^2 r/6$

If $V_o = 1$ and $M_o = 0$

$1 \times H = D_1\,\Delta_v$

$\Rightarrow \Delta_v = \frac{H}{D_1}$

If $M_o = 1$ and $V_o = 0$

$1 = D_1\,\Delta_M \Rightarrow \Delta_M = \frac{1}{D_1}$

Again $\theta_V H = \Delta_V \Rightarrow \theta_V = \frac{\Delta_V}{H} = \frac{H}{D_1}\frac{1}{H} = \frac{1}{D_1}$

and $\theta_M H = \Delta_M \Rightarrow \theta_M = \frac{\Delta_M}{H} = \frac{1}{D_1}\frac{1}{H} = \frac{1}{D_1 H}$

(b) Well in clay

Net resisting moment about base

$M = D_2 \Delta$

where $D_2 = K_h H^2/3 + (2/3) K_v r^3/H + \mu H_h Hr/2$

Similarly

$\Delta_V = \frac{H}{D_2}, \Delta_M = \theta_V = \frac{1}{D_2}$ and $\theta_M = \frac{1}{D_2 H}$

In above expression

K_h : Coefficient of horizontal subgrade reaction $= K_{si} \frac{(H/D+0.5)}{1.5\ H/D}$

K_v : Coefficient of vertical subgrade reaction $= K_{si}$, when base of the foundation is circular;

K_{si} : Soil modulus value;

D : Transverse dimension of the well;

H : Depth of embedment;

r : Radius of well;

μ : Coefficient of friction;

Δ : Deflection at c.g. of the embedded portion of the well; $= \Delta_o/2$

2.3 BERDUGO-NOVAK SPRING

Berdugo-Novak developed an approximate analytical solution for embedded footing based on the following assumptions.

(i) The foundation is assumed to consist of elastic half-space base layer and an elastic side layer resting over the base layer. The side layer provides the embedment to the well.

(ii) The base layer and side layer have constant elastic properties

(iii) The buried portion of the well is considered to be rigid.

(iv) The embedded structure is assumed to be cylindrical.

They replaced the elastic resistance for a embedded footing in soil by coupled linear and rotational springs at the center of gravity of the rigid foundation. The frequency independent spring constant are given by

(i) Translational spring constant

$$K_{xx} = G r_o \left(4.78 + \frac{G_s}{G} \delta\, 4.033\right)$$

(ii) Rotational spring constant

$$K_{\theta\theta} = G r_o^3 \left[2.5 + \frac{Z_c^2}{r_o^2} 4.78 + \frac{G_s}{G} \delta\, 2.5 + \frac{G_s}{G} \delta \left(\frac{\delta^2}{3} + \frac{Z_c^2}{r_o^2} - \delta \frac{Z_c}{r} \right) 4.033 \right]$$

(iii) Cross spring constant

$$K_{x\theta} = - G r_o \left[Z_c\, 4.78 + \frac{G_s}{G} \delta \left(Z_c - \frac{L}{2} \right) 4.033 \right]$$

The frequency independent damping constant are

(i) Translational damping constant

$$C_{xx} = (\rho G)^{0.5}\, r_o^2 \left[2.97 + \delta \left(\frac{\rho_s}{\rho} \frac{G_s}{G} \right)^{0.5} 9.60 \right]$$

(ii) Rotational damping constant

$$C_{\theta\theta} = (\rho G)^{0.5} r_o^4 \left[0.43 + \frac{Z_c^2}{r_o^2} 2.97 + \delta \left(\frac{\rho_s}{\rho} \frac{G_s}{G} \right)^{0.5} \left\{ 1.8 + \left(\frac{\delta^2}{3} + \frac{Z_c^2}{r_o^2} - \frac{\delta Z_c}{r_o} \right) 9.60 \right\} \right]$$

(iii) Cross damping constants

$$C_{x\theta} = - (\rho G)^{0.5} r_o^2 \left[Z_c\, 2.97 + \left(\frac{\rho_s}{\rho} \frac{G_s}{G} \right)^{0.5} \delta \left(Z_c - \frac{L}{2} \right) 9.60 \right]$$

In the above expressions

G : Shear modulus of base layer;

G_s : Shear modulus of side layer;

r_o : Radius of foundation;

L : Depth of embedment;

δ : Embedment ratio $= L/r_o$;

Z_c : Distance of center of gravity of foundation from the base;

ρ : Mass density of the base soil;

ρ_s : Mass density of the side soil;

CODE PROVISION

3.1 INDIAN CODE, IS: 1893-1984 (Fourth Revision)

The soil effects on the structures have been considered in Indian code for designing earthquake resistant structure. In this chapter, this aspect has been discussed as recommended by the code in practice now a days i.e. IS 1893:1984 (Fourth revision).

In clause 3.4; these are two methods given for computing the design seismic forces on the structures. These are -

3.1.1 Seismic Coefficient Method

The design value of horizontal seismic coefficient α_h, by this method is computed as

$$\alpha_h = \beta I \alpha_o$$

here,

β = a coefficient depending upon soil foundation system given in Table 3.1.

I = a factor depending upon the importance of structure

= 1.5 for important bridges.

and

α_o = Basic horizontal seismic coefficient given in Table 3.2.

3.1.2 Response Spectrum Method

The design value of horizontal seismic coefficient α_h, by this method in computed as -

$$\alpha_h = \beta I F_o \, S_a/g$$

here

β = a coefficient depending upon soil foundation system

I = A factor depending upon the importance of structure

F_o = seismic zone factor for average acceleration spectra (Table 3.2)

& S_a/g = average acceleration coefficient for appropriate natural period and damping of structure given in Fig 2.

Hence, it can be seen that the soil effect on the structure is incorporated in the factor β, which takes in to account of type of soil on which structure is built and also the type of foundation provided.

The values of Basic horizontal seismic coefficients α_o , in seismic coefficient method is given in the code for each zone (Table 3.2). For foundation at 30 m depth or below, the basic seismic coefficient is taken as 0.5a0 . For foundation placed between ground level and 30 m. depth it is linearly in interpolated between α_o and $0.5\alpha_o$

Table 3.2 Values of Basic Seismic coefficients and Seismic zone factor in different zones

S.No.	Zone.No.	Basic horizontal seismic coefficient, α_o	Seismic zone factor F_o
1	V	0.08	0.40
2	IV	0.05	0.25
3	III	0.04	0.20
4	II	0.02	0.10
5	I	0.01	0.05

3.2 IS 1893: (FIFTH REVISION)

In the fifth revision, the country is divided in to four seismic zones instead of five zones in the fourth revision. Zone I has been upgraded to zone II. The revised seismic zone map of India is given in Fig.3.

3.2.1 Horizontal seismic force

The revised code has given the design horizontal seismic coefficient A_h for design basis earthquake (DBE) as

$$A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{R/I}$$

here Z = zone factor, refers to zero period acceleration values for maximum credible earthquake (MCE) in a zone given in Table 3.3.

Table 3.3 Zone factor, Z

Seismic zone	II	III	IV	V
Seismic intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

The factor 2 in the numerator is because DBE is half the MCE

I = Importance factor. I = 1.5 for important bridge,

R = Response reduction factor to take into account ductility of the structure.

= 4 for bridge substructure constructed in R.C.C. and 1.0 if constructed in brick/stone masonry. The ratio (R/I) should not be less then 1.0.

S_a/g = Acceleration coefficient for rock and soil sites as given in Fig.4(a) & 4(b)

Hence it can be observed that the spectra is given for soil site and rock site in the fifth revision instead of soil site spectra given in fourth revision.

DYNAMIC ANALYSIS OF BRIDGE SUBSTRUCTURES

4.1 GENERAL

For the dynamic analysis of deep embedded foundations like pier well combination, caisson, hollow box, pile foundation, raft foundation, wall foundation of bridge, consideration of influence of flexibility of the support i.e. soil-structure interaction is very essential for practical design of bridges. To consider the flexibility of the support, the different soil springs and dashpots proposed by different author are considered at the C.G. of the embedded foundation. The soil spring system signifies the soil-structure interaction between the structure and the surrounding soil system.

The bridge substructure should made sufficient strong to withstand its own inertia force during earthquake excitation. The girder which rest on the top of substructure may fail due to in sufficient capacity of the bridge substructure. Falling of girder has occurred in the past earthquake. During earthquake there may be differential movement of the substructure and superstructure, which also leads to failure and collapse of bridge.

From the above aspect, it is essential to analyse the bridge substructure dynamically for safe, economic and adequate design.

4.2 METHOD OF ANALYSIS

There are many methods for dynamic analysis of bridge substructure. They are

(i) Transfer Matrix Method

(ii) Stiffness matrix methods

(iii) Direct Integration Scheme for Non-Linear System

(iv) Step by Step Integration

(v) Random Seismic Analysis

(vi) Finite Element Analysis

In the present study, the Transfer Matrix Method and stiffness matrix method are used for numerical analysis. The details of Transfer Matrix approach and stiffness matrix method is discussed in this chapter.

4.3 TRANSFER MATRIX METHOD

4.3.1 Assumptions

The different assumptions made for the seismic analysis of the bridge are as follows.

(a) Ground Motion

The ground motion is considered to be acting in either longitudinal or transverse direction, one at a time. The design of the bridge has been based upon the acceleration response spectra for Design Basis Earthquake (DBE).

(b) Live Load

(i) The seismic force due to live load shall be ignored when acting in the direction of traffic.

(ii) In the direction perpendicular to traffic, the seismic force due to live load shall be calculated for.

- 50 percent of design excluding impact for railway bridges.
- 25 percent of design live load excluding impact for highway bridges.

(c) Added mass of water

Hydrodynamic force shall be assumed to act in horizontal direction corresponding to that of earthquake direction. The weight of the water in the enveloping cylinder for submerged part of the substructure is considered to be attached with the structure for the purpose of dynamic analysis.

(d) Damping in the structure

The damping is considered as 5% of critical.

(e) Foundation Springs

The elastic resistance of a bridge substructure embedded in soil is replaced by the coupled linear and rotational springs at C.G. of the substructure as suggested by Beredugo - Novak.

4.3.2 Mathematical Models

The structure of the bridge consists of beam type element and therefore, the idealization can be made with lumped mass at discrete points. These masses are connected with each other by mass less elastic segments. The formulation of the mathematical model is the most important in the complete analysis. There should be enough number of lumped masses in the mathematical model in order to represent the dominating frequencies of the real structure. The elastic properties of the segment corresponds to those at the middle of the segment. The mathematical model of bridge substructure and foundation are shown in Fig. 5 & 6.

The model of the bridge substructure are different in two direction mainly because a fraction of live load is considered in the transverse direction while no live load is considered in the longitudinal direction. In longitudinal direction, the superstructure D.L. is lumped at the center of bearing. In transverse direction, the D.L. and L.L. are lumped at the C.G. of the respective loads.

Weight at a node consists of the self weight contained in between the mid section of segments, the added weight of water and weight of in filled water or sand. The foundation springs are placed at the center of gravity of embedded part of the well.

4.3.3 METHOD OF DETERMINATION OF DYNAMIC CHARACTERISTICS OF SUBSTRUCTURE

The method of transfer function is most convenient for determining dynamic characteristics of substructure. General equations for an elastic member with positive co-ordinates as shown in Fig. 3 are as follows.

$$V_n = V_{n-1} + m_{n-1} p^2 Y_{n-1}$$

$$M_n = M_{n-1} + V_n h_n - \rho I_n h_n p^2 \theta_{n-1} \qquad \ldots(4.1)$$

$$\theta_n = \theta_{n-1} + \frac{h_n}{2EI_n}\left((M_n + M_{n-1}\right)$$

$$Y_n = Y_{n-1} + h_n \theta_{n-1} + \frac{h_n^2}{3\,EI_n}\left[M_{n-1} + \frac{M_n}{2}\right] - \frac{\sigma V_n h_n}{G A_n}$$

Where,

Suffix n corresponding to the quantity in n-th segment

A : Cross-sectional area;

V : Shear force;

M : Bending Moment;

θ : Slope;

y : Deflection;

p : Natural frequency in rad/sec.;

m : Mass;

h : Length of the segment;

ρ : Mass density;

I : Moment of inertia;

E : Modulus of elasticity;

G : Modulus of rigidity;

σ : Shape factor for shear deflection

4.3.4 Application of Transfer Functions

The determination of the natural frequency and the mode shape consists of application of equation (4.1) from one node point to the other and appropriately satisfy the boundary conditions at the two ends. Out of the four boundary conditions at the free end, shear and moment are zero. Transfer function are computed for two conditions at the free end as follows.

(i) $y_o = 1, \theta_o = 0$ and

(ii) $y_o = 0, \theta_o = 1$ ...(4.2)

By successive application of transfer functions in these two steps, the coefficients C_{11} C_{12}..... etc. can be computed and the following relation can be established.

$$\begin{bmatrix} y \\ \theta \\ M \\ V \end{bmatrix}_F = \begin{bmatrix} C_{11} & C_{12} \\ C_{21} & C_{22} \\ C_{31} & C_{32} \\ C_{41} & C_{42} \end{bmatrix} \begin{bmatrix} Y \\ \theta \end{bmatrix}_o \qquad \ldots(4.3)$$

The shear and moment are related with the deflection and slope at partially fixed ends by the following equation

$$V_F = K_{xx}\, y_F - K_{x\theta}\, \theta_F$$

$$M_F = K_{x\theta}\, y_F - K_{\theta\theta}\, \theta_F \qquad \ldots(4.4)$$

The negative sign in θ_F is there because of opposite sign conventions used in the transfer functions and stiffness coefficients. Upon substituting the values of V, M,Y, and θ from equation (4.3) in (4.4) we can obtain the following two equations.

$$\begin{bmatrix} \left(C_{44} - K_{xx}\, C_{11} + K_{x\theta}\, C_{21}\right) & \left(C_{42} - K_{xx}\, C_{12} + K_{x\theta}\, C_{22}\right) \\ \left(C_{31} - K_{x\theta}\, C_{11} + K_{\theta\theta}\, C_{21}\right) & \left(C_{22} - K_{x\theta}\, C_{12} + K_{\theta\theta}\, C_{22}\right) \end{bmatrix} \begin{bmatrix} y_o \\ \theta_o \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \qquad \ldots(4.5)$$

$$\Rightarrow \begin{bmatrix} d_{11} & d_{12} \\ d_{21} & d_{22} \end{bmatrix} \begin{bmatrix} y_o \\ \theta_o \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \qquad \ldots(4.6)$$

For non trial solutions for slope and deflection at free and determinant of the matrix should vanish.

Hence,

$$DT = \begin{bmatrix} d_{11} & d_{12} \\ d_{21} & d_{22} \end{bmatrix} = 0 \qquad \ldots(4.7)$$

The zero of the determinant will then give a frequency of the structure and corresponding mode shape are obtained from the deflected shape of the cantilever.

After the determination of the frequency and mode shapes, the response is found by modal analysis.

4.3.5 Modal Analysis

The dynamic response of the bridge substructure in any mode say r-th mode of vibration, due to earthquake excitation may be obtained by the relation.

$$X_r = C_r\, x_r\, S_{dr} \qquad \ldots(4.8)$$

where,

X_r = Dynamic response of the structure which may be shear, moment, deflection of any other quantity.

X_r = Modal values of deflection, bending moment or any other quantity considerations:

C_r = Mode participation factor

$$= \frac{\sum_{i=1}^{N} m_i \, y_{ir}}{\sum_{i=1}^{N} m_i \, y_{ir}^2} \qquad \ldots(4.9)$$

m_i = i-th mass point;

y_{ir} = Horizontal displacement of mass m_i at i-th node in r-th mode;

N = Number of masses used in the lumped mass modal;

S_{dr} = Spectral displacement in r-th mode of vibration

$$= \frac{S_{ar}}{p_r^2}$$

S_{ar} = Spectral acceleration;

P_r = Natural frequency in r-th mode.

The spectral displacement depends upon the earthquake motion and the time period and damping of the structure. The overall structural response is obtained by taking square root of sum of squares (S.R.S.S) of individual modal response.

4.4 STIFFNESS MATRIX METHOD

4.4.1 General

In this method displacements are taken as basic unknowns. Knowing the stress-strain relationship, the internal forces can be expressed in terms of these displacements. Finally, considering the equilibrium of each joint, a set of linear simultaneous equations can be solved for displacements.

The structure is defined in terms of coordinates of the joints, the structural characteristics and the end conditions of each member. The stiffness matrix of the structure as a whole is formed from the stiffness matrices of individual members. Mathematical treatment, ultimately, leads to a large system of simultaneous components in static analysis, or time period and model amplitude in dynamic analysis with the help of high speed digital computers. In the present analysis, deformations are assumed to be small, which implies linear behaviour. Thus the principle of superposition is valid.

The stiffness matrix method is one in which compatibility of displacement is assumed and the equilibrium equations at the nodes are formulated in terms of the nodal displacement components. To develop the stiffness matrix, beam member is assumed to be fully restrained at both the ends and subjected to unit displacement along each relevant degree of freedom one at a time, while all other displacement components are retained as zero. The member stiffness are the action exerted on the member when these unit displacements (translations and rotations) are imposed at each end.

4.4.2 Beam Member in a Plane

The stiffness coefficient for six possible types of coplanar displacements at the two ends of fully restrained member are given in table below. The deformations have been considered in the order of translation along x and y axis and rotation about z axis for both the ends.

STIFFNESS MATRIX IN MEMBER AXIS FOR FULLY RESTRAINED MEMBER IN

$$
\begin{bmatrix}
\frac{EA_x}{L} & 0 & 0 & -\frac{EA_x}{L} & 0 & 0 \\
 & \frac{13EI_z}{L^3(1+\phi)} & \frac{6EI_z}{L^2(1+\phi)} & 0 & \frac{-12EI_z}{L^3(1+\phi)} & \frac{6EI_z}{L^2(1+\phi)} \\
 & & \frac{(4+\phi)EI_z}{L(1+\phi)} & 0 & \frac{6EI_z}{L^2(1+\phi)} & \frac{(2+\phi)EI_z}{L(1+\phi)} \\
 & & & \frac{EA_x}{L} & 0 & 0 \\
\text{SYMMETRIC} & & & & \frac{12EI_z}{L^3(1+\phi)} & \frac{-6EI_z}{L^2(1+\phi)} \\
 & & & & & \frac{(4+\phi)EI_z}{L(1+\phi)}
\end{bmatrix}
$$

$$\phi = \frac{12EI_z}{GA_sL^2}$$

4.4.3 Rotation Transformation Matrices

Rotation transformation matrix R_T is required, to transform the member stiffness matrix from member axes to structural axes, R_T can be obtained from a rotational matrix (R) expressed in term of direction cosines of the member. The rotation transformation matrices for plane frame has been given below.

$$[R_T] = \begin{bmatrix} R & 0 \\ 0 & R \end{bmatrix}$$

$$[R] = \begin{bmatrix} C & S & 0 \\ -S & C & 0 \\ 0 & 0 & 1 \end{bmatrix}$$

$$C = \frac{X_2 - X_1}{L}, \quad S = \frac{Y_2 - Y_1}{L}$$

$$L = \sqrt{(X_2-X_1)^2 + (Y_2-Y_1)^2}$$

X_1, Y_1, X_2, Y_2 refer to the co-ordinates of the two ends of a member in a plane.

4.4.4 Use of Rotation Transformation Matrices

(i) Member stiffness matrix for structure axes (S_{MS}) is obtained from the member stiffness matrix for member axes (S_M) by

$$(S_{MS}) = R_T^T S_M R_T$$

(ii) Joint deformations at two ends of a member in member axes (D_M) obtained from the joint deformations at the two ends of a member in structural axes (D_S) by

$$D_M = R_T D_S$$

which is used to calculate forces at the ends of the member in local axes.

4.4.5 Static Analysis

For analysing the structure, under lateral horizontal loads and eccentric vertical forces, it is represented by a rigid jointed skeletal structure. This skeletal representation can further be simplified and structure can be idealized as planner system for plane frame analysis.

(a) Stiffness Method of Analysis

This approach of analysis has been used to plane frames. The important steps involved in the method used in conjunction with an automatic digital computer are described in the following paragraphs.

(b) Assembly of Structural Data

Number of joints, no. of members, geometric and elastic properties of each remember section, node incidents at two ends of each member, coordinates of each nodes and condition of restraints at the support of the structure are coded.

(c) Generation of Joint Stiffness Matrix

The joint stiffness matrix of the structure as a whole is formed by summing up the contributions from individual member stiffness matrices.

(d) Generation of Load Vector

The nodal loads acting on the structure are taken directly, while distributed loads or non nodal loads on the member are converted to equivalent nodal loads. These equivalent nodal loads are then added to nodal loads and the load vector is assembled.

(e) Solution of Nodal Displacements

The joint stiffness matrix of the structure, its nodal forces and deformations are related by the following linear system or simultaneous equations presents in matrix form.

$$\begin{bmatrix} K_{11} & K_{12} & K_{1n} \\ K_{1} & K_{22} & K_{2n} \\ \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot \\ K_{n1} & K_{n2} & K_{nn} \end{bmatrix} \begin{Bmatrix} \delta_1 \\ \delta_2 \\ \cdot \\ \cdot \\ \delta_n \end{Bmatrix} = \begin{Bmatrix} P_1 \\ P_2 \\ \cdot \\ \cdot \\ P_n \end{Bmatrix} \qquad \ldots(4.1)$$

Or $[K]_{nxn} \{\delta\}_n = \{P\}_n$

where, $[K]_{nxn}$ = Matrix of stiffness influence coefficients

$\{\delta\}_n$ = Deformation vector

$\{P\}_n$ = Force vector

n = Number of degrees of freedom.

Gauss elimination technique has been used to solve Eq.4.1. This method is based on triangularization of the stiffness coefficient matrix by taking the advantage of the elementary property of the determinants of the matrices. The unknown deformations δ are then evaluated by back substitution starting from the last equation.

(f) Member Forces and Reactions

From the joint deformation, obtained by solving linear simultaneous equations, support reactions and member and sections are computed as follows:

(i) The joint deformations at the two ends of a member in member axis are obtained from the joint deformations at the two ends or a member in structure axes (D_S) by

$$D_M = R^T D_S$$

where, R_T = Rotation Transformations matrix

(ii) The joint deformations (D_M) are further used to calculate forces at the ends of a member in member axes by

$$A_M = S_M D_M$$

where, S_M = member stiffness matrix

4.4.6 Dynamic Analysis

The analysis in this section is divided into free vibration analysis and dynamic response analysis.

(a) Free Vibration Characteristics

Free vibration characteristics are basically system characteristics and consist of natural frequencies and the corresponding mode shapes. The method of determination of the above characteristics are described in the following paragraphs.

$$\{M\}_n \{\ddot{X}\} + \{K\}_n \{X\}_n = 0$$

$\{M\}$ = mass matrix

$\{K\}$ = stiffness matrix

$\{K\}$ = displacement vector

$\{\ddot{X}\}$ = acceleration vector

n = order of matrices and vector

If we assume X = x Sinpt, above equation is converted to

(i) $K_n X_n = P^2 M_n X_n$

(ii) $K_n^{-1} M_n X_n = \frac{1}{P^2} X_n$ ...(4.2)

Above equations are the forms of standard eigen value problem whose solution leads to evaluation of natural frequencies and corresponding mode shapes. Since we are interested in only the first few modes of the physical system, we would require model frequencies and corresponding mode shape starting from minimum frequency of the system, sequentially. Hence, the later form is generally preferred for the sequential determination of eigen pairs, since the 'power iteration' yields the maximum root and this provides p_{min}, which is useful for determining the eigen pairs, 'Deflation' has to be adopted in such a manner so as to preserve the banded nature and symmetry of the matrices involved because of the immense computational advantage gained. The product K^{-1} M of above equation is able to maintain symmetry condition if $m_1 = m_2$ where $m_1, m_2, \ldots\ldots m_n$ are the diagonal elements of [M] of order n. In general, this condition may never be achieved and the symmetry can be enforce by resorting to the following transformations.

$$\left[M^{1/2}\right]_n \left[K^{-1}\right]_n \left[M^{1/2}\right]_n \left[M^{1/2}\right]_n [X]_n = \frac{1}{p^2} \left[M^{1/2}\right]_n \{X\}_n$$

Substituting

$\left[A_n\right]$ (The modified dynamic matrix) $= \left[M^{1/2}\right]_n \left[K^{-1}\right]_n \left[M^{1/2}\right]_n$

$\left[Y\right]_n = \left[M^{1/2}\right]_n \{X\}_n$ = mode shape vector of the modified dynamic matrix and $\frac{1}{P^2}$ = eigen value of the system

We get $\left[A_n\right] \left\{Y\right\}_n = \lambda \left\{Y\right\}_n$

The eigen vector of the original system may be calculated by

$$\left[X\right]_n = \left[M^{-1/2}\right] \left\{Y\right\}$$

The method of inverse iteration technique. Coupled with Gram Schmidt orthogonalisation, has been used for the solution of the eigen value problem to obtain the first Six modes of vibration.

(b) Method of Solution

There are two types of method for eigen solution namely, transformation methods, and iteration methods. Transformation methods, are used when it is required to determine all the eigen values and eigen vectors, those are not generally required for structural analysis/design. The iterative method, on the other and, can avoid necessity of storing the entire matrix. This method yields a sequence of scalars and vector that converges to some particular eigen value and its corresponding eigen vector. Equation 3.2 can be rewritten in the following modified form.

$$\left[K\right] \left\{\frac{1}{P^2_{i+1}} X_{i+1}\right\} = \left[K^{-1}\right] \left[M\right] \left[X_i\right]$$

If $\left\{Y_{i+1}\right\} = \left\{\frac{-1}{P^2_{i+1}} X_{i+1}\right\}$ and

$$\{Z_i\} = [M]\{X_i\}$$

Than the above equation can be written as $[K]\{Y_{i+1}\} = \{Z_i\}$ in which $[K]$ and $\{Z_i\}$ are known. The above represents a system of linear equations which can be solved by standard techniques. Then separation of P^2_{i+1} and X_{i+1} can be achieved easily.

The major problem then is the sequential determination of the eigen values (and eigen vectors) or the natural frequencies (and mode shapes) of the system. These can be achieved, by deflation technique, by suppressing or eliminating established eigen pairs.

(c) Response Spectra Analysis

From the free vibration analysis, the natural periods of vibration and associated mode shapes could be determined. The relative displacement Z of a mass, along any of its degree of freedom in a particular mode of vibration due to horizontal component of an earthquake is given by

$$Zi_d^{(r)} = \phi i_d^{(r)}\, C_j^{(r)}\, Sd_j^{(r)}$$

where ϕ represents the mode shape coefficient,

'i' = denotes the mass location

'd' = denotes the degree of freedom

'r' = denotes the mode of vibration

'j' = represents the direction of ground motion

The mode participation factors

$$C_j = \frac{\{\phi\}\ [M]\ \{L_j\}}{(\phi)^T\ [M]\ \{\phi\}}$$

where $\left[L_j\right]^T = \{\ 1,0,0,0,0,0\ 1,0......\}$ for $j=1$

The spectral displacement S_d is equal to the maximum relative displacement of the mass (relative to ground) of a single degree of freedom system having the same period as that of modal period and damping same as that of modal damping.

Using the generalised displacement vector and from element stiffness matrix of each member, the generalised member forces are determined. Thus forces and displacements can be determined in each mode of vibration for any particular ground motion. In this manner, the dynamic response in each mode of vibration is obtained. The total response is a combination of various modes of vibration.

(d) Conventional Approach

Usually the inertia forces are calculated corresponding to the period of vibration. The dynamic displacements and consequently the member forces are obtained by means of a static analysis. The step involved are as follows

(i) $F_{ir} = m_i\ S_a^r\ C^r\ \phi_i^r$
$= S^r\ C^r\ [M]\ \{\phi\}^r$

where, S_a^r is spectral acceleration for rth mode

ϕ_i^r is the modal displacements for the ith d.o.f. in rth mode

C_r is the mode participation factor corresponding to the direction of applied ground motion.

(ii) $[K]\{\delta\}^r = [F]^r$ is solved for dynamic displacement $\{\delta\}^r$

$$\{\delta\}^r = \left[K^{-1}\right] S_a^r C^r \left[M\right] \{\phi\}^r$$

$$\{\delta\}^r = S_a^r C^r \left[K^{-1}\right] \left[M\right] \{\phi\}^r$$

(iii) member forces can be calculated

$$\left\{f_m\right\}_r = \left[K_m\right] \left\{\delta_m\right\}$$

$$= \left[K_m\right] S_a^r C^r \left[K^{-1}\right] \left[M\right] \left\{\phi^r\right\}$$

RESULT AND DISCUSSION

5.1 GENERAL

In order to study the comparison of soil effect on seismic response of bridges due to modification in code spectra three different bridges have been selected.

Dynamic analysis has been carried out for all the bridges using transfer matrix/stiffness matrix method as discussed in Chapter 4.

The bridges selected for study are :

(a) Dhaleswari Bridge

(b) Gandhak Bridge

(c) Ringhal Khad Bridge

5.2 Description of Bridges

The description of these bridges are given below :

5.2.1 Dhaleswari Bridge

Type	:	Simply Supported
Zone	:	V
Spans	:	7 x 31.926 + 2 x 13.10
Superstructure	:	Steel truss bridge resting over rocks and roller bearings
Pier	:	Single solid circular concrete pier (M20)
Well	:	8.0 m overall diameter (concrete M20) hollow circular R.C. well
Height from base to top of pier	:	52.725 m
Base diameter	:	8.0 m
Depth of embedment	:	23.0 m

The cross-sectional details with mathematical model are shown in Figure 8. The site spectra is given in Fig.9. The bridge is analysed in longitudinal direction for Design Basis Earthquake. The multiplying factor for DBE site spectra is 0.097. The damping is considered as 5% of critical.

The bridge is analysed for founding soils having four different shear wave velocities of 200m/s, 400m/s, 600m/s, 800m/s, and under fixed base conditions with two embedment depths of 23m and 35.55m. The soil springs used are Beredugo-Novak springs as discussed in Chapter - 2.

5.2.2 Gandak Bridge

Type	:	Simply Supported
Zone	:	IV
Spans	:	14 x 64.0
Superstructure	:	Rail-cum-Road Bridge
Pier	:	Circular concrete pier (M20)
Well	:	Outer diameter = 12.00 m Inner diameter = 6.40 m
Height of well	:	41.50 m
Base diameter	:	12.0 m
Depth of embedment	:	19.32 m

The cross-sectional details with mathematical model are shown in Figures 10 & 11. The site spectra is given in Fig.12. The bridge is analysed in longitudinal direction for Design Basis Earthquake. The multiplying factor for DBE used for site spectra is 0.075. Damping factor of 5% is considered.

The bridge is analysed for founding soils having four different shear wave velocities and under fixed base conditions with two embedment depths 19.32m and 41.5m. The soil springs used are Berdugo-Novak springs as discussed in Chapter - 2.

5.2.3 Ringhal Khad Bridge

Type	:	Continuous prestressed concrete box girder
Zone	:	IV
Spans	:	80 + 2 x 56 m
Width of superstructure	:	6.9m
Height of tallest pier	:	36.355 m
External diameter of hollow pier	:	6.5 m
Wall thickness of pier	:	0.5 m
Thickness of footing	:	3.25 m
Plan size of open foundation	:	15.25 m x 15.25 m

The cross-sectional details with mathematical model are shown in Figures 13 & 14 . The site spectra is given in Fig.15. The bridge is analysed in longitudinal direction for Design Basis Earthquake. The multiplying factor for DBE used for site spectra is 0.095. Damping factor of 5% is considered.

The bridge is analysed for fixed base condition and also using Berdugo-Novak springs.

5.3 DISCUSSION OF RESULTS

The results are discussed as below :

5.3.1 Dhaleswari Bridge

The time periods of bridge are determined for founding soils having four different shear wave velocities and under fixed base condition. The response is compared for spectra of IS:1893-1984 (ISA), soil spectra (ISRS) and rock spectra (ISRR) of revised code and for site spectra (SR) for two embedment depths.

(a) Time periods

Time periods of six modes of vibration corresponding to different shear wave velocities and for fixed base conditions are shown in Table 5.1(a) and 5.1(b). It is observed from the table 5.1(a) that as shear wave velocity increases from 200 m/s to 800 m/s the fundamental time period decreased from 0.9841sec. to 0.5367 sec. The period for fixed base condition is 0.5086sec. The time period has decreased due to increased stiffness.

It is also observed that for increased embedment depth of 35.55 m, the time period has decreased to 0.4882sec from 0.5606 sec. for shear wave velocity of 600 m/s,

(b) Deflection at the top of the pier

The horizontal deflection at the top of the pier for different spectra is shown in Table 5.2.

It is observed from table 5.2(a) for embedment depth 23 m that the deflection is decreasing as the shear wave velocity increases and it is minimum for fixed base condition.

The effect of different spectra on the deflection can be studied for shear wave velocity 600 m/s and for spectra ISRS. It is observed that the deflection for ISRS is 118.8% more than the deflection for ISA.

The deflection for ISRS is 53.1% more than the deflection for ISRR. It is also observed that the deflection for SR is 29.73% more than for ISRS.

(c) B.M. near the base of the pier

The B.M. near the base of the pier for embedment depths 23 m and 35.55 m is shown in Table 5.3(a) and 5.3(b). It is observed that the B.M. is first increasing with increase in shear wave velocity from 200 m/s to 400 m/s and then it is decreasing with increase in shear wave velocity from 400 m/s to 800 m/s. The B.M. is minimum for fixed base condition.

The effect of embedment depth can be studied for shear wave velocity 600 m/s and using ISRS. It is observed that the B.M. for embedment depth 23 m is 153% more than the B.M. for embedment depth 35.55 m.

The effect of different spectra on the B.M. can be studied for $V_s = 600$ m/s and for embedment depth 23 m. It is observed that the B.M. for ISRS is 138.4% more than B.M. for ISA.

The B.M. for ISRS is 50.2% more than the B.M. for ISRR. The B.M. for SR is 32.5% more than the B.M. for ISRS.

(d) Shear near the base of the pier

The shear near the base of the pier for embedment depth 23 m and 35.55 m is shown in Table 5.4(a) and 5.4(b). The effect of shear wave velocity can be studied for embedment depth of 23 m and for ISRS spectra. It is observed that the shear is increasing with increase in shear wave velocity from 200 m/s to 800 m/s but the shear has decreased for fixed base condition.

The effect of embedment depth can be studied by comparing the shear for ISRS using shear wave velocity 600 m/s. It is observed that the shear for embedment depth 23 m is 22.7% more than the shear for 35.55 m.

The effect of different spectra on the shear can be studied for V_s = 600 m/s and for embedment of 23 m. It is observed that the shear for ISRS is 118% more than shear for ISA.

It is also observed that the shear for ISRS is 0.83% more than the shear for ISRR. The shear for SR is 73.6% more than the shear for ISRS.

5.3.2 Gandak Bridge

The time periods of bridge are determined for founding soils having four different shear wave velocities and under fixed base condition. The response is compared for spectra of IS:1893-1984 (ISA), soil spectra (ISRS) and rock spectra (ISRR) of revised code and for site spectra (SR) for two embedment depths.

(a) Time periods

Time periods of six modes of vibration corresponding to different shear wave velocities and for fixed base conditions are shown in Table 5.5(a)& (b) and 5.5(b). It is observed from the table 5.5(a) that as shear wave velocity increases from 200 m/s to 800 m/s the fundamental time period decreased from 2.1528 sec. to 0.6684 sec. The period for fixed base condition is 0.4226sec. The time period has decreased due to increased stiffness.

It is also observed that for increased embedment depth of 41.5 m, the time period has decreased to 0.2314 from 0.8136 sec. for shear wave velocity of 600 m/s,

(b) Deflection at the top of the pier

The horizontal deflection at the top of the pier for different spectra is shown in Table 5.6.

It is observed from table 5.6(a) for embedment depth 19.32 m that the deflection is decreasing as the shear wave velocity increases and it is minimum for fixed base condition.

The effect of different spectra on the deflection can be studied for shear wave velocity 600 m/s and for spectra ISRS. It is observed that the deflection for ISRS is 115% more than the deflection for ISA.

The deflection for ISRS is 77.6% more than the deflection for ISRR. It is also observed that the deflection for ISRS is 11.7% more than for SR.

(c) B.M. near the base of the pier

The B.M. near the base of the pier for embedment depths 19.32 m and 41.5m is shown in Table 5.7(a) and 5.7(b). It is observed that the B.M. is first increasing with increase in shear wave velocity from 200 m/s to 400 m/s and then it is decreasing with increase in shear wave velocity from 400 m/s to 800 m/s. The B.M. is minimum for fixed base condition.

The effect of embedment depth can be studied for shear wave velocity 600 m/s and using ISRS. It is observed that the B.M. for embedment depth 19.32 m is 203% more than the B.M. for embedment depth 41.5 m.

The effect of different spectra on the B.M. can be studied for V_s = 600 m/s and for embedment depth 19.32 m. It is observed that the B.M. for ISRS is 112.5% more than B.M. for ISA.

The B.M. for ISRS is 70% more than the B.M. for ISRR. The B.M. for SR is 10.5% less than the B.M. for ISRS.

(d) Shear near the base of the pier

The shear near the base of the pier for embedment depth 19.32 m and 41.5m is shown in Table 5.8(a) and 5.8(b). The effect of shear wave velocity can be studied for embedment depth of 19.32 m and for ISRS spectra. It is observed that the shear is increasing with increase in shear wave velocity from 200 m/s to 800 m/s and it is maximum for fixed base conditions.

The effect of embedment depth can be studied by comparing the shear for ISRS using shear wave velocity 600 m/s. It is observed that the shear for embedment depth 41.5 m is 8.6% more than the shear for 19.32 m.

The effect of different spectra on the shear can be studied for V_s = 600 m/s and for embedment of 19.32 m. It is observed that the shear for ISRS is 112.8% more than shear for ISA.

It is also observed that the shear for ISRS is 65% more than the shear for ISRR. The shear for ISRS is 9.4% more than the shear for SR.

5.3.3 Ringhal Bridge

The time periods of bridge are determined for fixed base condition (Table 5.9) and the responses are compared for spectra of IS:1893-1984, soil spectra and rock spectra of revised draft code and for site spectra.

(a) Deflection at the top of the Pier P1

It is observed that the deflection for ISRS is 139% more than for ISA. The deflection for ISRS is 174.4% more than for ISRR. It is also observed that the deflection for ISRS is 73.5% more than for SR.

(b) Moment near the base of the pier P1

It is observed that the moment for ISRS is 64.6% more than for ISA. The moment for ISRS is 52.4% more than for ISRR. It is also observed that moment for SR is 65.5% more than for ISRS.

(c) Shear near the base of the pier P1

It is observed that the shear for ISRS is 60.8% more than for ISA. the shear for ISRS is 48.9% more than for ISRR. It is also observed that the shear for SR is 72.1% more than for ISRS.

SUMMARY AND CONCLUSIONS

6.1 Summary

The study of seismic response (B.M. shear force and deflections) of bridge substructure using spectra of IS:1893-1984 (ISA), Rock (ISRR) and soil (ISRS) spectra of fifth revision and site spectra (SR) have been done in the present dissertation. For this purpose three bridges have been considered. These bridges are analysed using Transfer Matrix Method/Stiffness Matrix Method.

Dhaleswari and Gandhak bridges have been analysed for shear wave velocities of 200 m/s, 400 m/s, 600 m/s, 800 m/s and for fixed base conditions. Beredugo-Novak spring have been considered at the C.G. of the embedment. Two embedment depths have been considered for analysis. Ringhal bridge has been analysed for fixed base condition. The bridges are analysed in longitudinal direction for Design Basis Earthquake.

These bridges are analysed and responses are compared using different spectras.

6.2 Conclusions

On the basis of seismic response studies of three bridges for different spectras, following conclusions are drawn :

(1) Horizontal deflection has decreased as the share wave velocity increases and is minimum for fixed base condition. Horizontal deflection decreases as the embedment depth increases.

(2) For Dhaleswari bridge, the deflection for soil spectra of revised draft is 118.8% more than for spectra of IS:1893-1989 for shear wave velocity of 600 m/s. The deflection is 53.1% more for soil spectra of revised draft than for Rock Spectra of revised draft. The deflection for site spectra is 29.73% more than for soil spectra of revised draft.

For Gandhak bridge, the deflection for soil spectra of revised draft is 115% more than for spectra of IS:1893-1984 for shear wave velocity of 600 m/s. The deflection is 77.6% more for soil spectra of revised draft than for Rock Spectra of revised draft. The deflection for soil spectra of mixed draft is 11.7% more than for site spectra.

For Ringhal bridge, the deflection for soil spectra of revised draft is 139.1% more than for spectra of IS:1893-1984. The deflection is 174.4%more for soil spectra of revised draft than for rock spectra of revised draft. The deflection for soil spectra of revised draft is 73.5% more than site spectra.

(3) For Dhalelswari bridge, the Bending moment for soil spectra of revised draft is 138.4% more than for spectra of IS:1893-1984 for shear wave velocity of 600 m/s. The Bending Moment for soil spectra of revised draft is 50.2% more than for rock spectra of revised draft. The Bending Moment for site spectra is 32.5% more than for soil spectra of revised draft.

For Gandhak Bridge, the Bending moment for soil spectra of revised draft is 112.5% more than for spectra of IS:1893-1984. The moment for soil spectra of revised draft is 70% more than the rock spectra of the revised draft. The moment for soil spectra of revised draft is 10.5% more than for site spectra.

For Ringhal bridge, the moment for soil spectra of revised draft is 64.6% more than for spectra of IS:1893-1984. The moment for soil spectra of revised draft is 52.4% more than for rock spectra of revised draft. The moment for site spectra is 65.5% more than for soil spectra of revised draft.

(4) For Dhaleswari bridge, the shear for soil spectra of revised draft is 118% more than for spectra of IS:1893-1984 for shear wave velocity of 600 m/s. The shear for soil spectra of revised draft is 0.83% more than for rock spectra of revised draft. The shear for soil spectra for revised draft is 0.83% more than for rock spectra of revised draft. The shear for site spectra is 73.6% more than for soil spectra of revised draft.

For Gandhak bridge, the shear for soil spectra of revised draft is 112.8% more than for spectra of IS:1893-1984. The shear for soil spectra of revised draft is 65% more than for rock spectra of revised draft. The shear for soil spectra of revised draft is 9.4% more than for site spectra.

For Ringhal bridge, the shear for soil spectra of revised draft is 60.8% more than for spectra of IS:1893-1984. The shear for soil spectra of revised draft is 48.9% more than for rock spectra of revised draft. The shear for site spectra is 72.1% more than for soil spectra of revised draft.

6.3 SCOPE FOR FURTHER STUDY

There is scope for further study as follows:

1. The present study has been carried out for longitudinal directions. Study can be extended for transverse direction also.
2. Present study has considered Berdugo-Novak Springs. Study can be extended for other springs also.
3. Study can be carried out using three-dimensional modelling of the structure.

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Table 5.1(a) TIME PERIODS (Seconds) for emb.d.=23m

	V=200 m/s	V=400m/s	V=600 m/s	V=800 m/s	fixed
mode1	0.9841	0.6335	0.5606	0.5367	0.5086
mode2	0.3549	0.2969	0.2528	0.2272	0.1815
mode3	0.1208	0.0854	0.0742	0.0683	0.0592
mode4	0.0578	0.0538	0.0509	0.0483	0.0399
mode5	0.0393	0.0368	0.0347	0.0327	0.0219
mode6	0.0219	0.0219	0.0218	0.0218	0.0160

Table 5.1(b) TIME PERIODS (Seconds) for emb.d =35.55m

	V=200 m/s	V=400m/s	V=600 m/s	V=800 m/s	fixed
mode1	0.6342	0.5071	0.4882	0.4821	0.4748
mode2	0.3244	0.2224	0.1808	0.1642	0.1431
mode3	0.1305	0.0950	0.0766	0.0633	0.0542
mode4	0.0545	0.0541	0.0539	0.0534	0.0226
mode5	0.0226	0.0226	0.0226	0.0226	0.0135
mode6	0.0135	0.0135	0.0135	0.0135	0.0103

Table 5.2(a) Deflection (m) at top of the pier (emb.d.=23m) Dhaleswari Bridge

	ISRS	ISRR	ISA	SR
V=200 m/s	4.39E-01	2.38E-01	2.17E-01	5.92E-01
V=400 m/s	2.83E-01	1.76E-01	1.18E-01	3.62E-01
V=600 m/s	2.22E-01	1.45E-01	9.17E-02	2.88E-01
V=800 m/s	1.95E-01	1.27E-01	8.04E-02	2.55E-01
Fixed at G	1.59E-01	1.02E-01	6.46E-02	2.06E-01

Table 5.2(b) Deflection at top of the pier (emb.d.=35.55m)

	ISRS	ISRR	ISA	SR
V=200 m/s	3.11E-01	1.94E-01	1.30E-01	4.00E-01
V=400 m/s	1.82E-01	1.18E-01	7.41E-02	2.37E-01
V=600 m/s	1.51E-01	9.63E-02	6.90E-02	1.96E-01
V=800 m/s	1.40E-01	8.90E-02	5.66E-02	1.82E-01
Fixed at G	1.26E-01	8.09E-02	5.10E-02	1.64E-01

Table 5.3(a)Moments (t-m) at node 18 at emb. depth=23 00m for Dhaleswari Bridge

	ISRS	ISRR	ISA	SR
V= 200 m/s	2.97E+04	1.76E+04	1.45E+04	4.08E+04
V= 400 m/s	3.57E+04	2.20E+04	1.50E+04	4.61E+04
V= 600 m/s	3.29E+04	2.19E+04	1.38E+04	4.36E+04
V= 800 m/s	3.13E+04	2.17E+04	1.31E+04	4.25E+04
Fixed at G	2.88E+04	2.17E+04	1.21E+04	4.11E+04

Table 5.3(b) Moments at node 11 at emb. depth=35.55 m.

	ISRS	ISRR	ISA	SR
V= 200 m/s	1.95E+04	1.44E+04	8.34E+03	2.66E+04
V= 400 m/s	1.53E+04	1.08E+04	6.31E+03	2.09E+04
V= 600 m/s	1.30E+04	8.74E+03	5.33E+03	1.74E+04
V= 800 m/s	1.22E+04	8.04E+03	4.99E+03	1.62E+04
Fixed at G	1.12E+04	7.24E+03	4.53E+03	1.46E+04

Table 5.4(a) Shear (t) in element 18 (emb. d=23 m) for Dhaleswari Bridge

	ISRS	ISRR	ISA	SR
V= 200 m/s	2.29E+03	1.32E+03	1.12E+03	3.12E+03
V= 400 m/s	2.86E+03	2.29E+03	1.22E+03	4.13E+03
V= 600 m/s	2.97E+03	2.78E+03	1.32E+03	4.72E+03
V= 800 m/s	3.01E+03	2.92E+03	1.35E+03	5.00E+03
Fixed at G	2.76E+03	2.76E+03	1.25E+03	4.78E+03

Table 5.4(b) Shear in element 11 (emb.d=35.55m)

	ISRS	ISRR	ISA	SR
V= 200 m/s	2.31E+03	1.63E+03	9.83E+02	3.14E+03
V= 400 m/s	2.46E+03	2.33E+03	1.11E+03	4.08E+03
V= 600 m/s	2.42E+03	2.40E+03	1.11E+03	4.20E+03
V= 800 m/s	2.27E+03	2.26E+03	1.02E+03	3.91E+03
Fixed at G	1.87E+03	1.83E+03	8.73E+02	3.15E+03

Table 5.5(a) TIME PERIODS (Seconds) for emb.d.= 19.32 m

	V=200 m/s	V=400 m/s	V=600 m/s	V=800 m/s	fixed
mode1	2.1528	1.1319	0.8136	0.6684	0.4226
mode2	0.2548	0.1873	0.1714	0.1624	0.1296
mode3	0.1216	0.0981	0.0864	0.0806	0.0608
mode4	0.0584	0.0534	0.0483	0.0445	0.0313
mode5	0.0313	0.0310	0.0306	0.0301	0.0223
mode6	0.0222	0.0222	0.0222	0.0222	0.0174

Table 5.5(b) TIME PERIODS (Seconds) for emb.d.=41.5 m

	V=200 m/s	V=400 m/s	V=600 m/s	V=800 m/s	fixed
mode1	0.6417	0.3288	0.2314	0.1915	0.1564
mode2	0.1564	0.1393	0.1297	0.1170	0.0259
mode3	0.0290	0.0258	0.0251	0.0249	0.0179
mode4	0.0169	0.0156	0.0152	0.0150	0.0141
mode5	0.0139	0.0138	0.0138	0.0138	0.0093
mode6	0.0095	0.0093	0.0093	0.0093	-

Table 5.6(a) Deflection (m) at top of the pier (emb.d.=19.32m) for Gandhak Bridge

	ISRS	ISRR	ISA	SR
V=200 m/s	8.64E-01	4.71E-01	3.87E-01	9.24E-01
V=400 m/s	4.02E-01	2.24E-01	2.10E-01	3.93E-01
V=600 m/s	2.86E-01	1.61E-01	1.33E-01	2.56E-01
V=800 m/s	2.32E-01	1.34E-01	9.75E-02	2.06E-01
Fixed at G	1.25E-01	8.73E-02	5.12E-02	1.17E-01

Table 5.6(b) Deflection at top of the pier (emb.d.=41.5 m)

	ISRS	ISRR	ISA	SR
V=200 m/s	2.01E-01	1.19E-01	8.34E-02	1.80E-01
V=400 m/s	7.99E-02	7.53E-02	3.64E-02	8.44E-02
V=600 m/s	4.91E-02	5.21E-02	2.24E-02	5.62E-02
V=800 m/s	3.72E-02	3.93E-02	1.71E-02	4.13E-02
Fixed at G	1.32E-02	1.41E-02	6.06E-03	1.41E-02

Table 5.7(a) Moments (t-m) at node 18 at emb. depth=19.32 m. for Gandhak Bridge

	ISRS	ISRR	ISA	SR
V= 200 m/s	1.17E+05	8.48E+04	5.27E+04	1.27E+05
V= 400 m/s	1.71E+05	9.84E+04	8.83E+04	1.69E+05
V= 600 m/s	2.21E+05	1.30E+05	1.04E+05	2.00E+05
V= 800 m/s	2.56E+05	1.50E+05	1.43E+06	2.27E+05
Fixed at G	2.94E+05	2.04E+05	1.21E+05	2.74E+05

Table 5.7(b) Moments at node 11 at emb. depth=41.50m.

	ISRS	ISRR	ISA	SR
V= 200 m/s	4.30E+04	2.86E+04	1.81E+04	3.91E+04
V= 400 m/s	6.18E+04	5.91E+04	2.82E+04	6.54E+04
V= 600 m/s	7.29E+04	7.73E+04	3.33E+04	8.22E+04
V= 800 m/s	7.59E+04	7.92E+04	3.46E+04	8.29E+04
Fixed at G	3.45E+04	3.67E+04	1.59E+04	3.69E+04

Table 5.8(a) Shear (t) in element 18 (emb.d=19.32 m) for Gandhak Bridge

	ISRS	ISRR	ISA	SR
V= 200 m/s	5.68E+03	3.48E+03	2.55E+03	6.10E+03
V= 400 m/s	9.03E+03	5.09E+03	4.67E+03	8.91E+03
V= 600 m/s	1.16E+04	7.03E+03	5.45E+03	1.06E+04
V= 800 m/s	1.33E+04	8.15E+03	7.25E+04	1.19E+04
Fixed at G	1.38E+04	1.02E+04	5.74E+03	1.30E+04

Table 5.8(b) Shear in element 11 (emb.d=41.5m)

	ISRS	ISRR	ISA	SR
V= 200 m/s	1.02E+04	6.15E+03	4.29E+03	9.15E+03
V= 400 m/s	1.32E+04	1.25E+04	6.03E+03	1.39E+04
V= 600 m/s	1.26E+04	1.33E+04	5.73E+03	1.44E+04
V= 800 m/s	1.07E+04	1.13E+04	4.90E+04	1.19E+04
Fixed at G	4.26E+03	4.41E+03	2.83E+03	5.06E+03

Table 9
Time Periods (seconds) for Ringhal Bridge for fixed base

Mode 1	1.2849
Mode 2	0.5634
Mode 3	0.4597
Mode 4	0.3194
Mode 5	0.2554
Mode 6	0.1534

Table 10
Deflection (m) at the top of the pier P1

ISA	1.353E-02
ISRS	3.236E-02
ISRR	1.179E-02
SR	1.865E-02

Table 11
Moments (t-m).near the base of pier P1
for Ringhal Bridge

ISA	1535.7
ISRS	2526.1
ISRR	1657.1
SR	4180.5

Table 12
Shear (t)near the base of pier p1
for Ringhal Bridge

ISA	50.82
ISRS	81.74
ISRR	54.90
SR	140.68

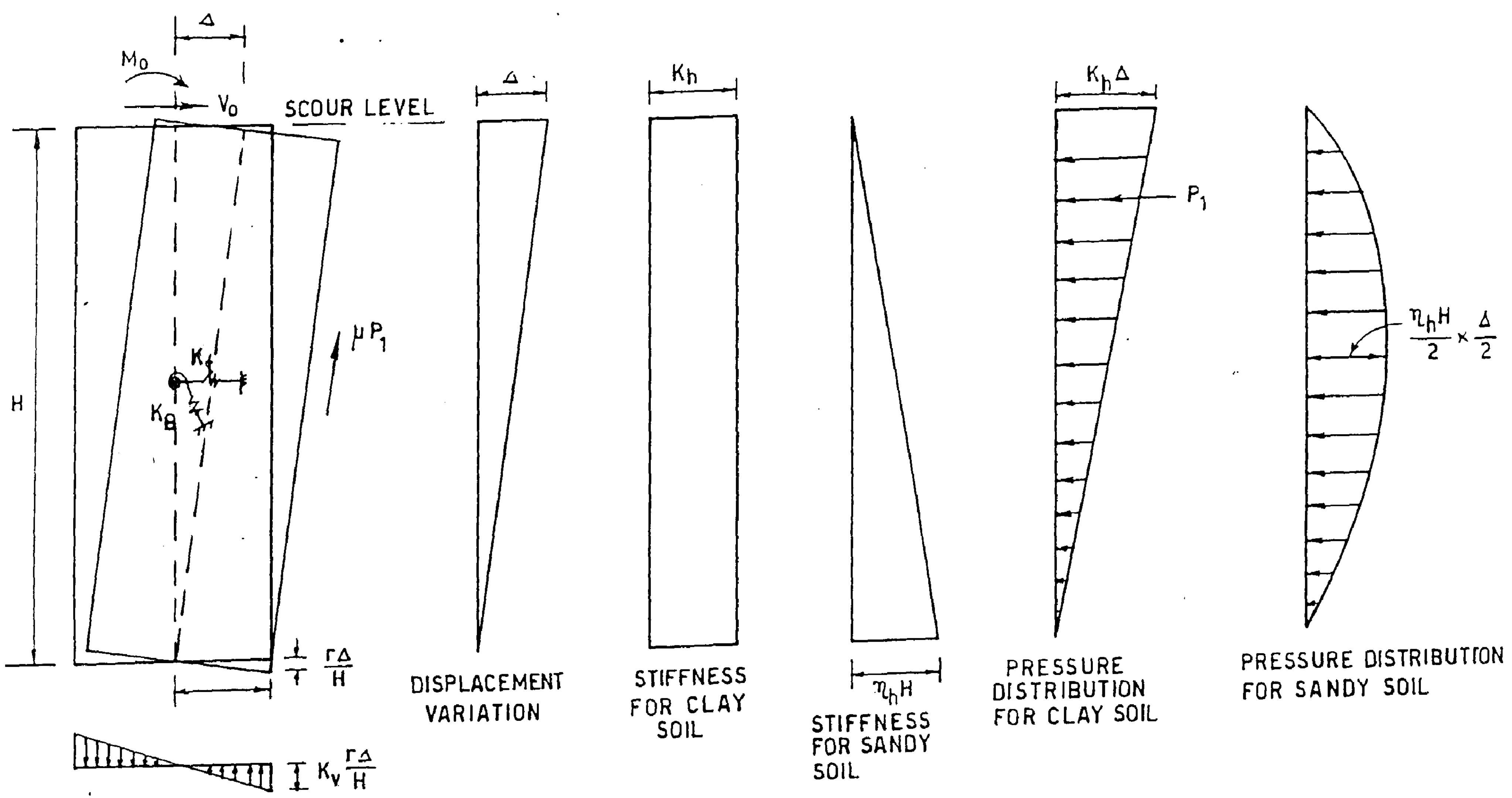


FIG. 1 _ REPLACEMENT OF SOIL BY SPRINGS AT C.G. FROM TERZAGHI'S CONCEPT

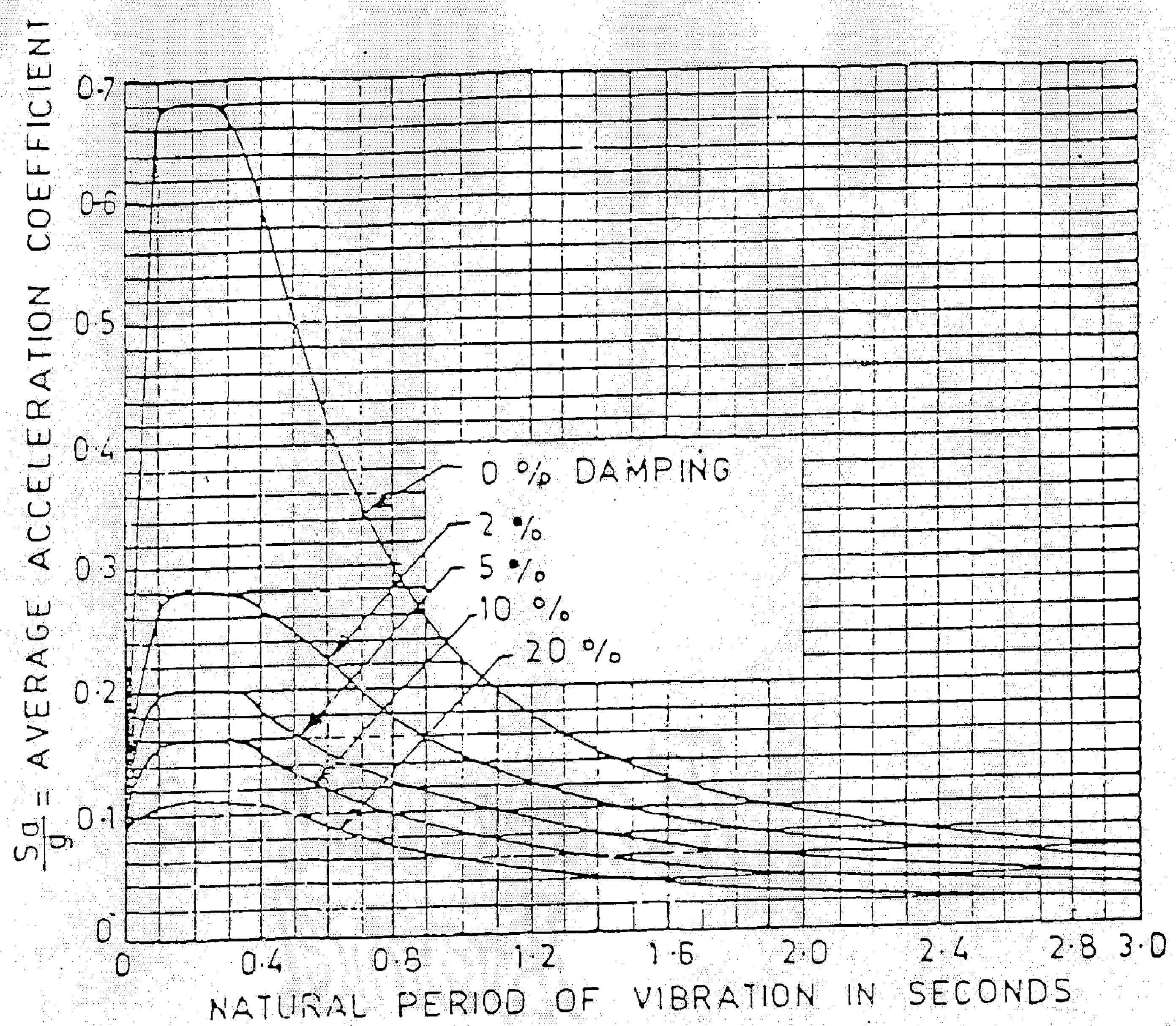


FIG. 2 AVERAGE ACCELERATION SPECTRA

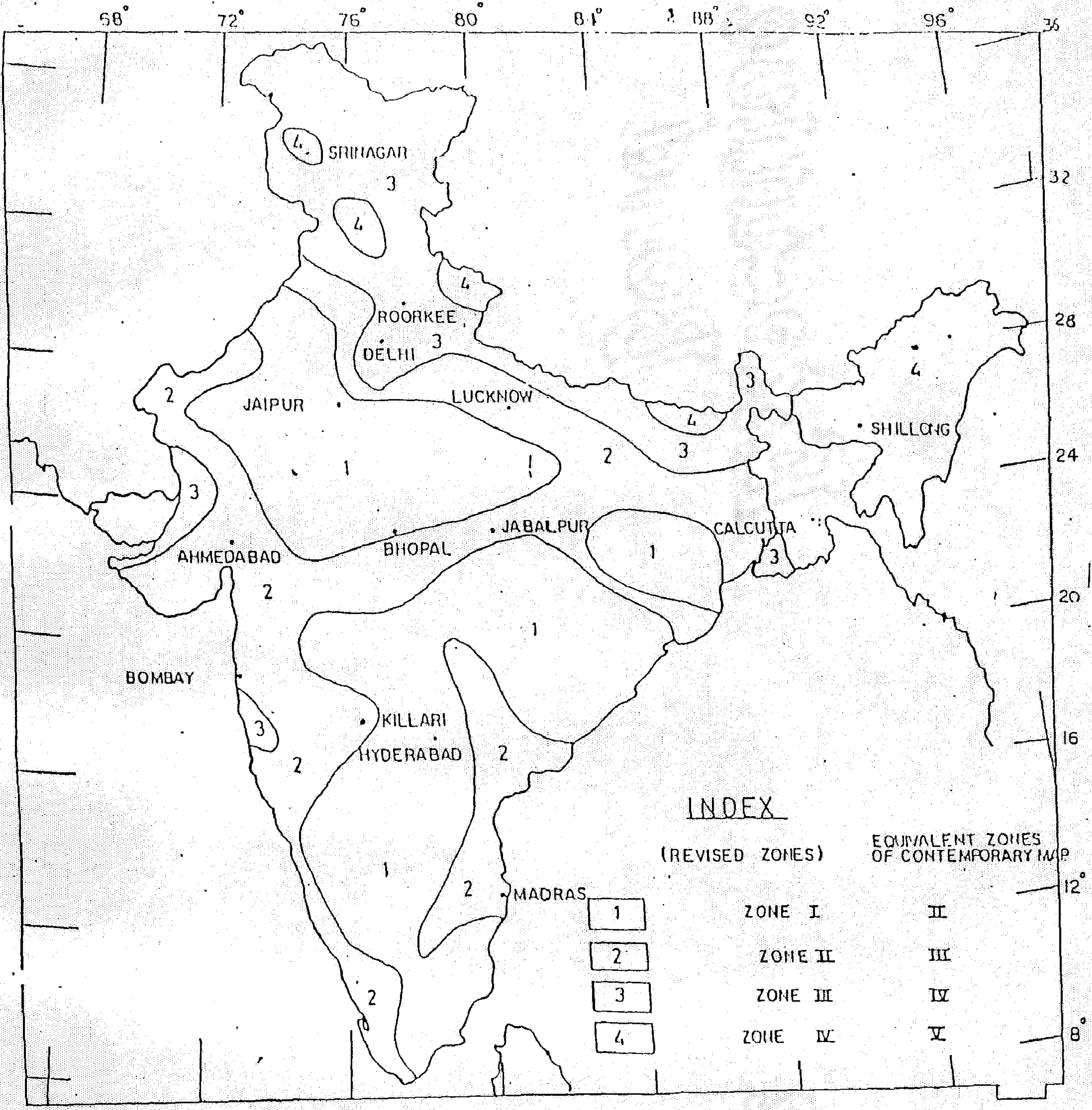


Figure 3 : Seismic Zone Map of India

Table 3 Values of β for different soil-foundation systems

Sl.No.	Type of Soil Mainly Constituting The Foundation	Values of β for					
		Piles Passing Through Any Soil but Resting on soil Type I	Piles Not Covered Under Col3	Raft Foundation	Combined or Isolated RCC Footings with Tie Beams	Isolated RCC Footing Without Tie Beams or Unreinforced Strip Foundations	Well Foundation
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i.	Type I Rock or hard soils	1.0	--	1.0	1.0	1.0	1.0
ii.	Type II Medium soil	1.0	1.0	1.0	1.0	1.2	1.2
iii.	Type III Soft soils	1.0	1.2	1.0	1.2	1.5	1.5

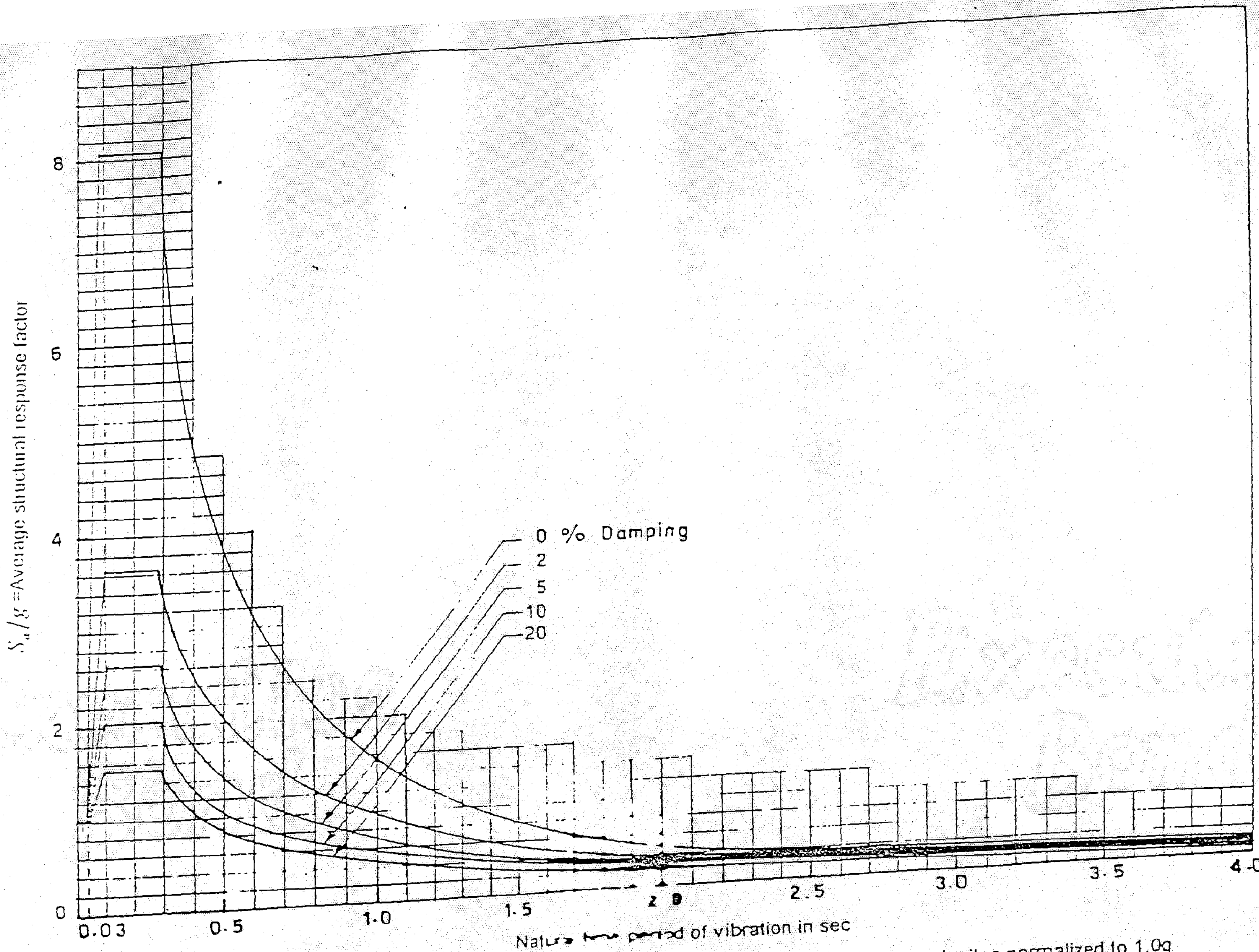


Fig.4a - Average structural response factor for 0, 2, 5, 10 and 20% dampings for rock sites normalized to 1.0g
Rock site: rock and stiff soil condition, shear wave velocity v_s > 750 m/s
or where the soil depth is less than 50 m

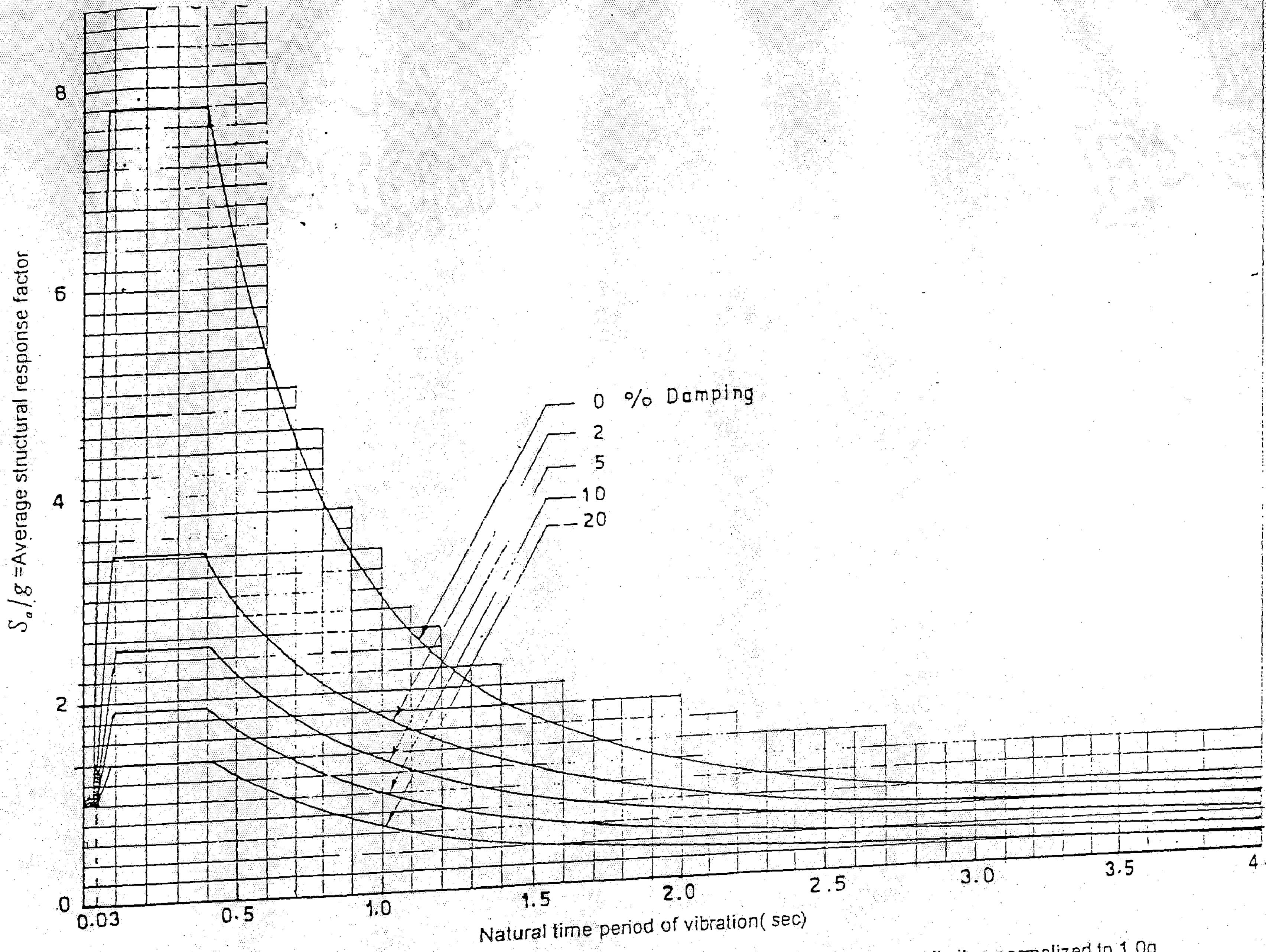


Fig4b - Average structural response factor for 0, 2, 5, 10 and 20% dampings for soil sites normalized to 1.0g
Soil site: Deep cohesionless or stiff clay soil condition, shear wave velocity $v_s < 750$ m/s
or where the soil depth is more than 50 m

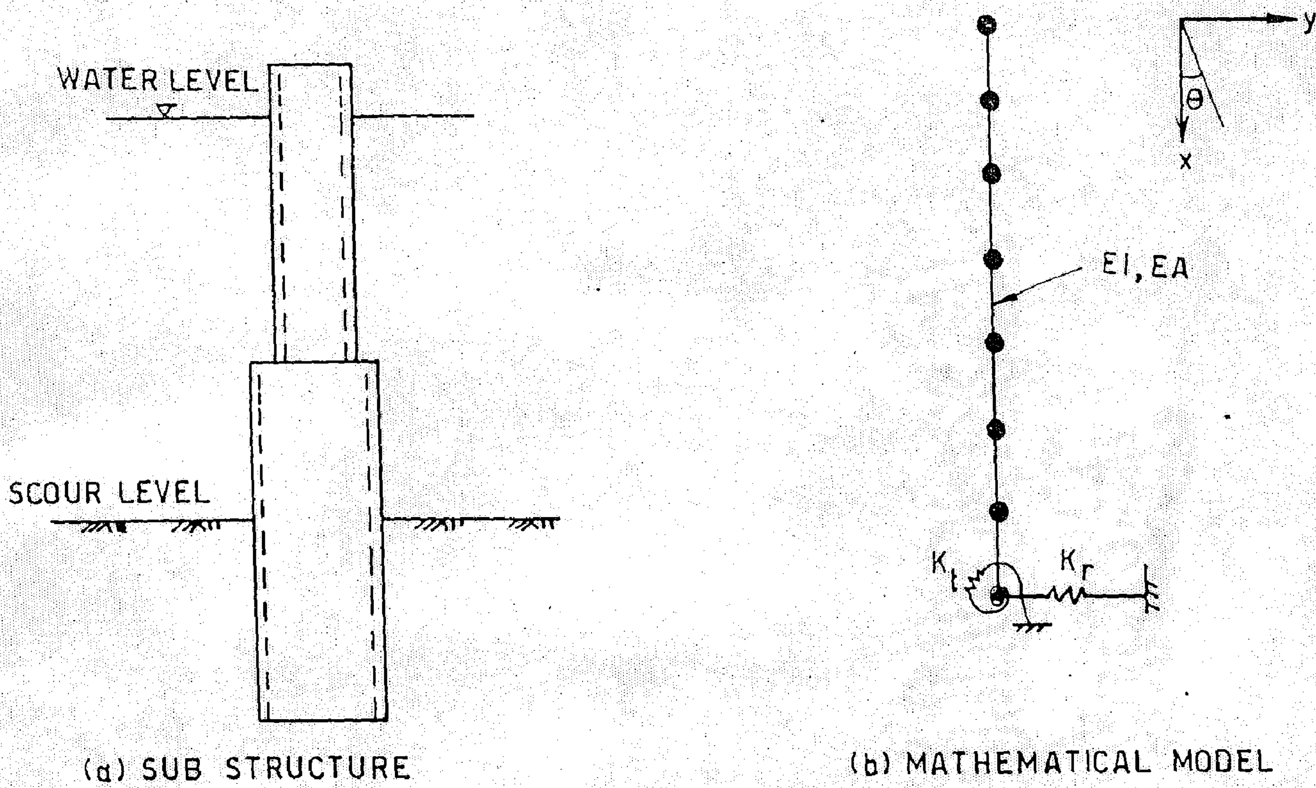


FIG 5.. SUBSTRUCTURE AND ITS MATHEMATICAL MODEL

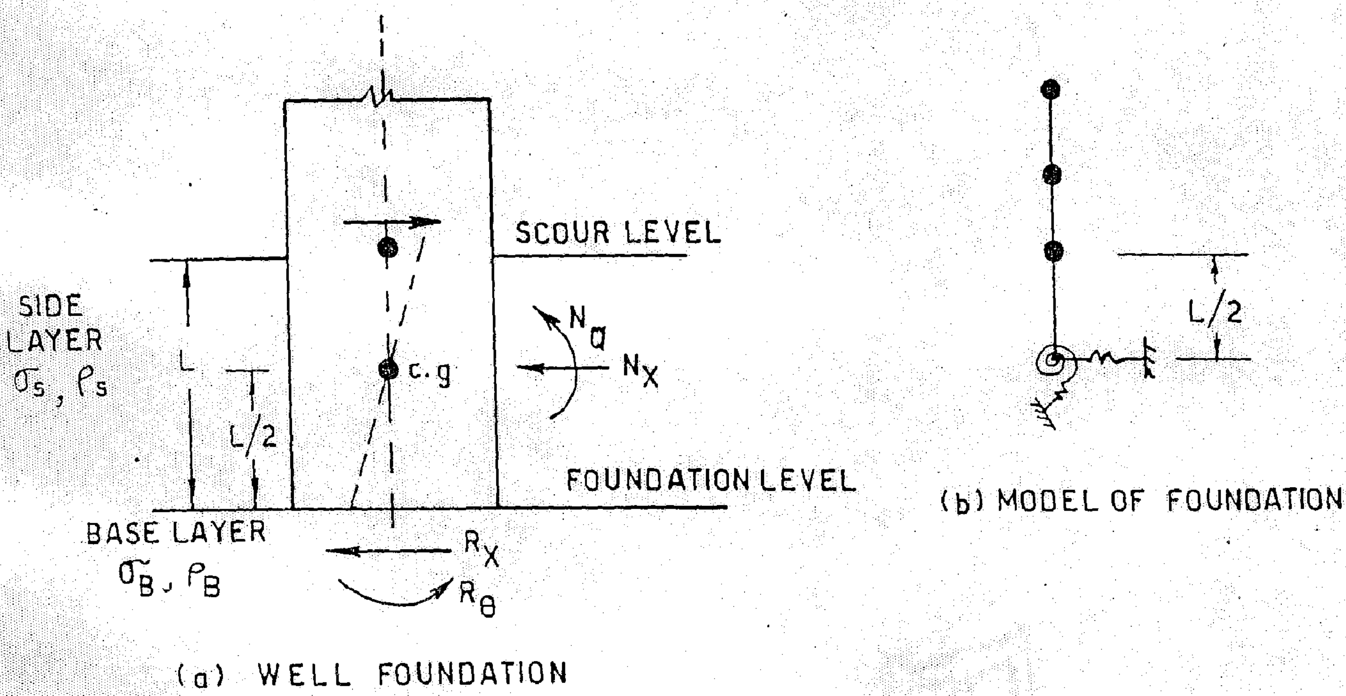


FIG. 6._ SOIL FOUNDATON MODEL

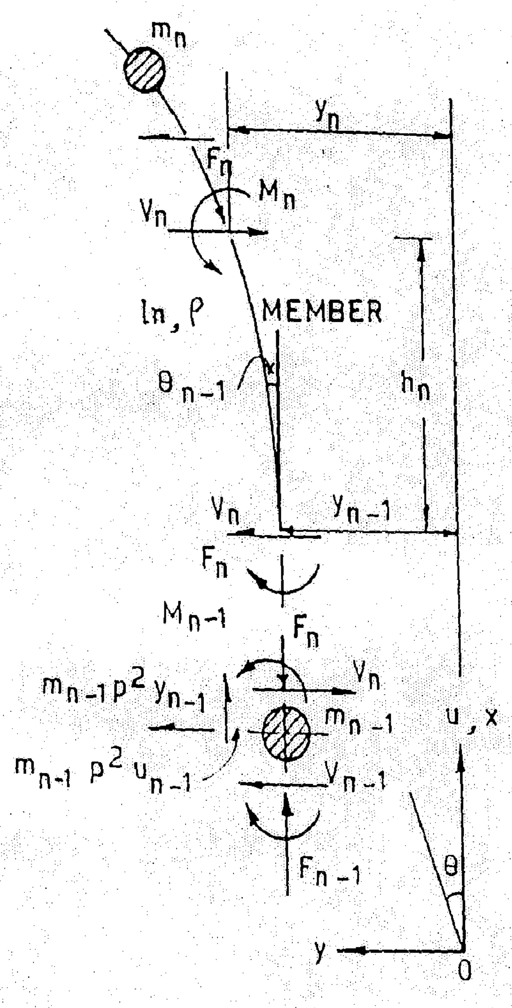


FIG.7. _ TRANSFER FUNCTIONS FOR ELASTICALLY VIBRATING MEMBER

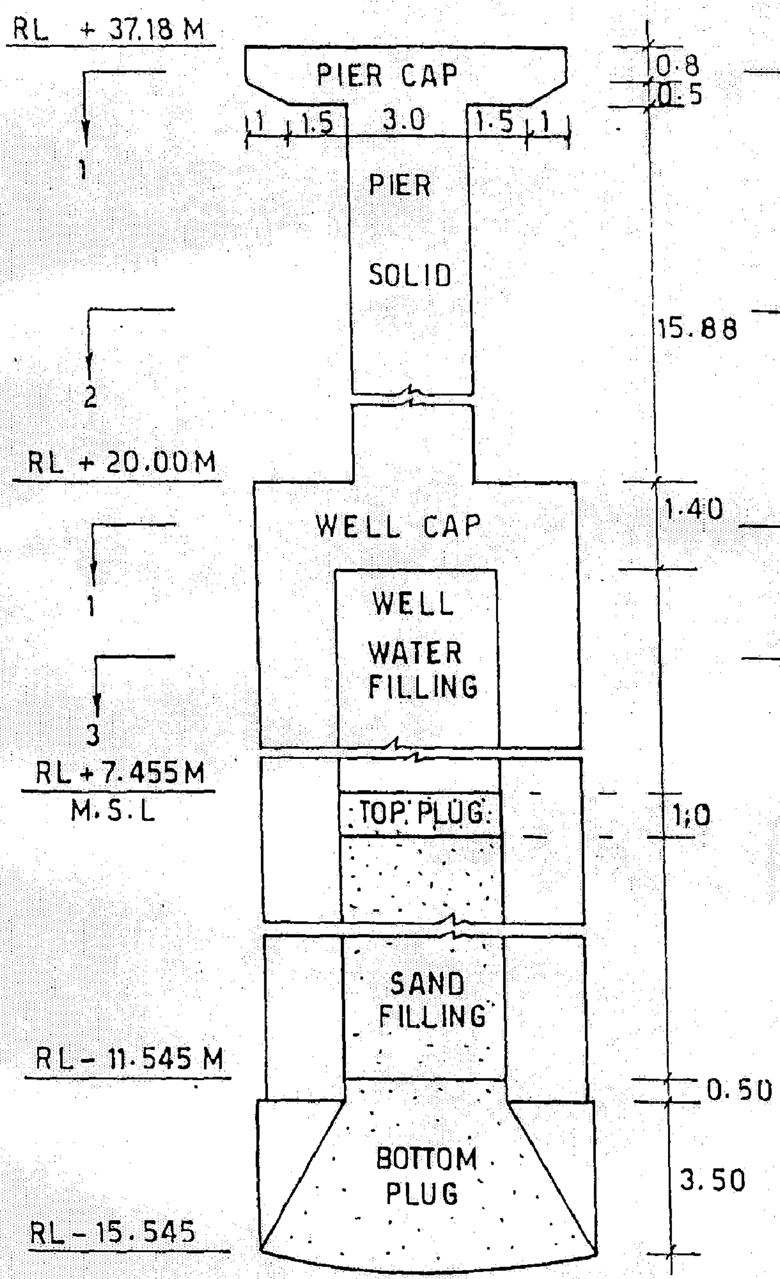


FIG. 8 (a)

SUBSTRUCTURE OF DHALESWARI BRIDGE

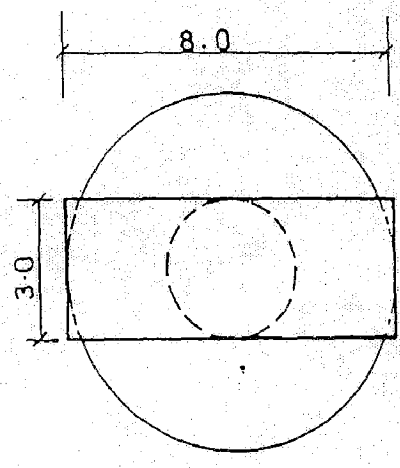


FIG. 8 (b)
SECTION 1-1

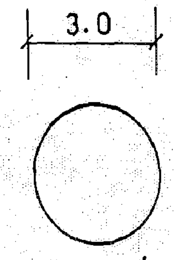
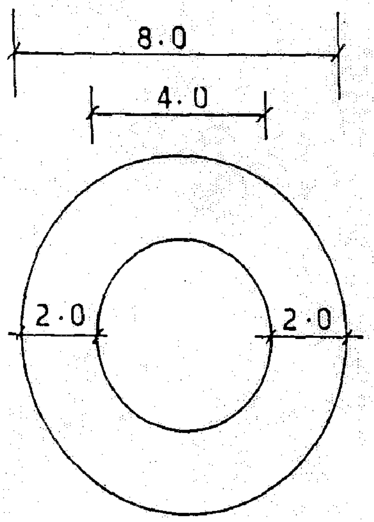


FIG. 8 (c)
SECTION 2-2



ALL DIMENSIONS IN M

FIG. 8 (d)
SECTION. 3-3

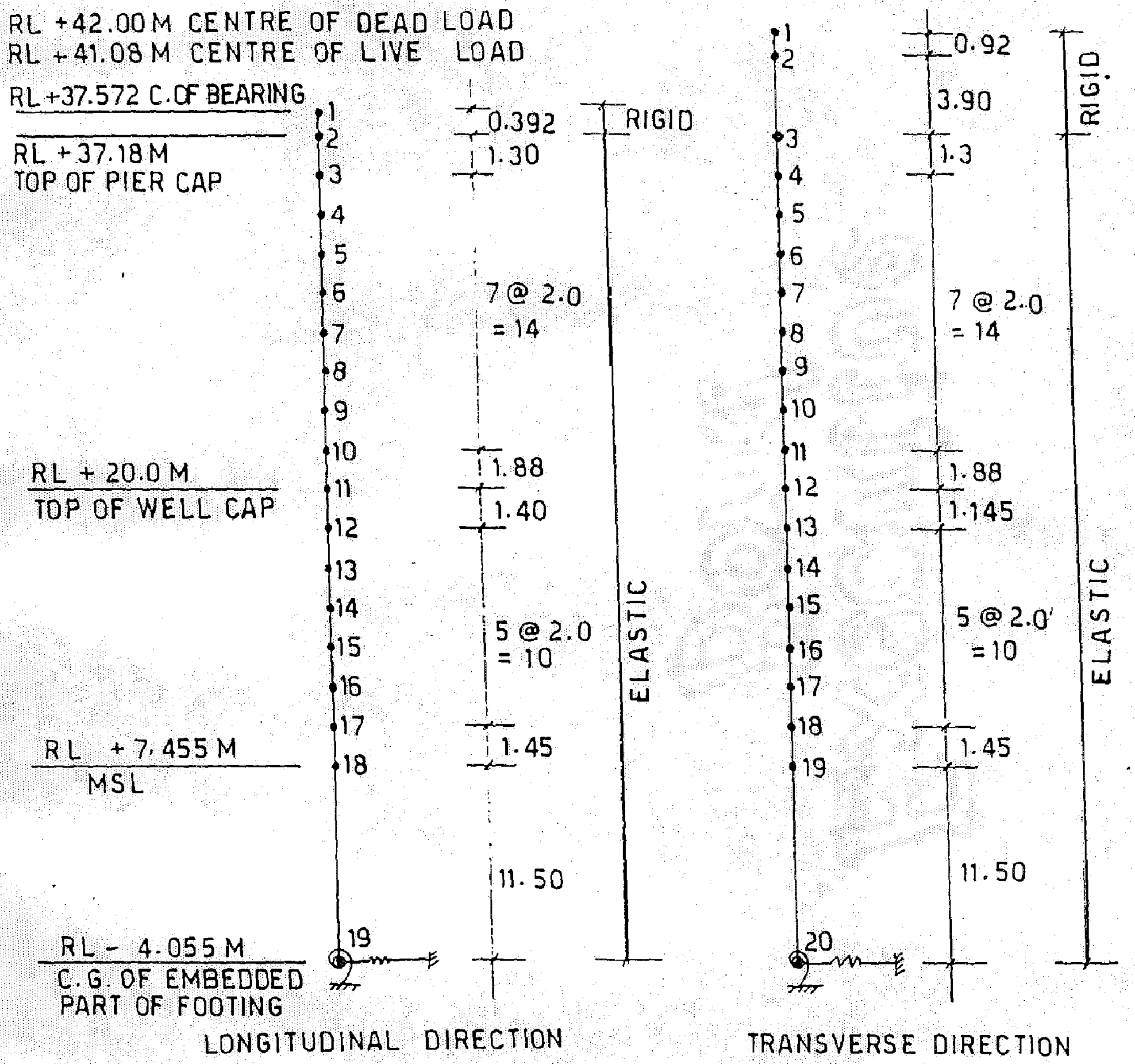


FIG. 8 (e) _ MATHEMATICAL MODEL OF DHALSWARI BRIDGE

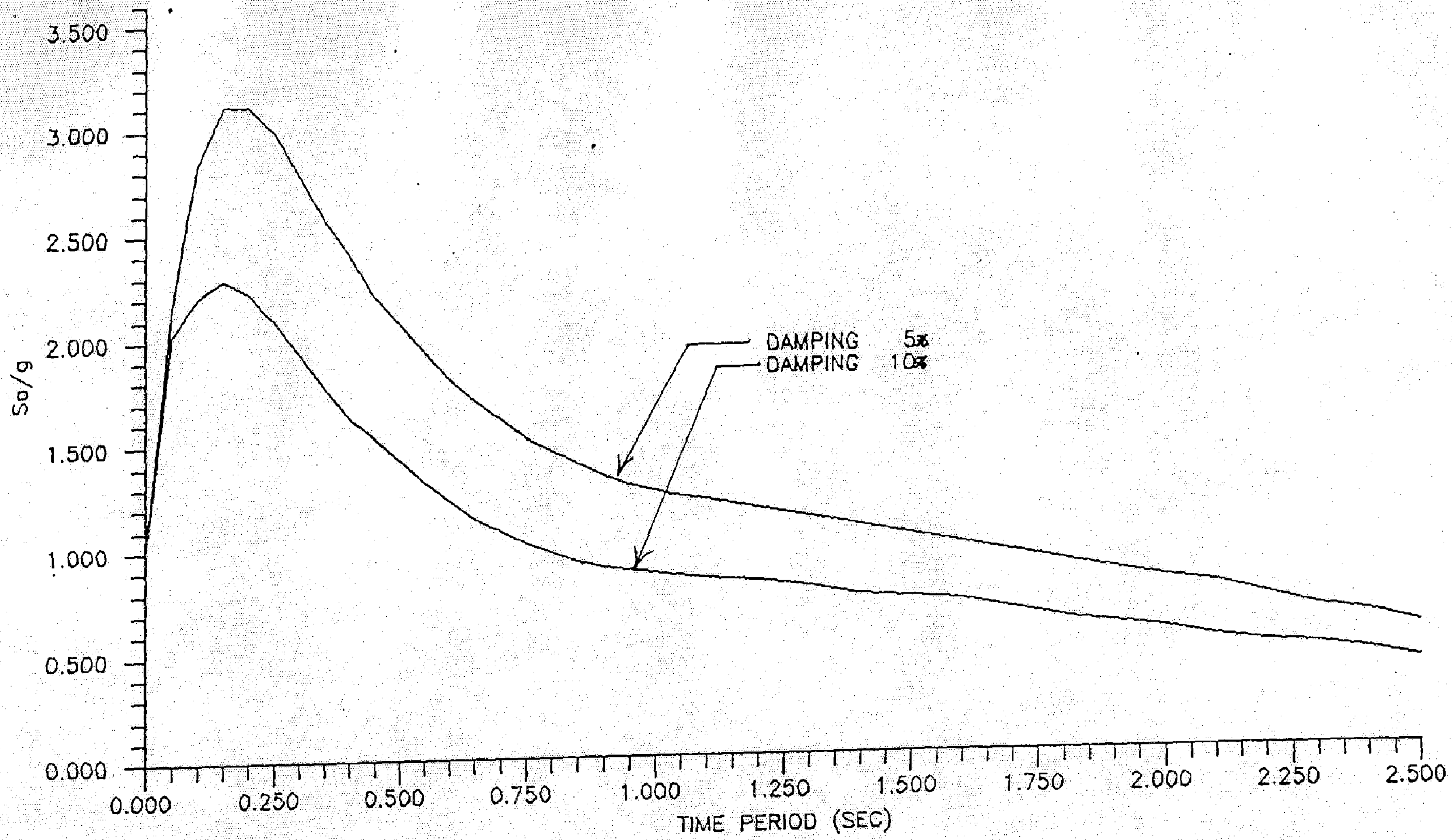


FIG 9. ACCELERATION RESPONSE SPECTRUM OF DHALESWARI BRIDGE

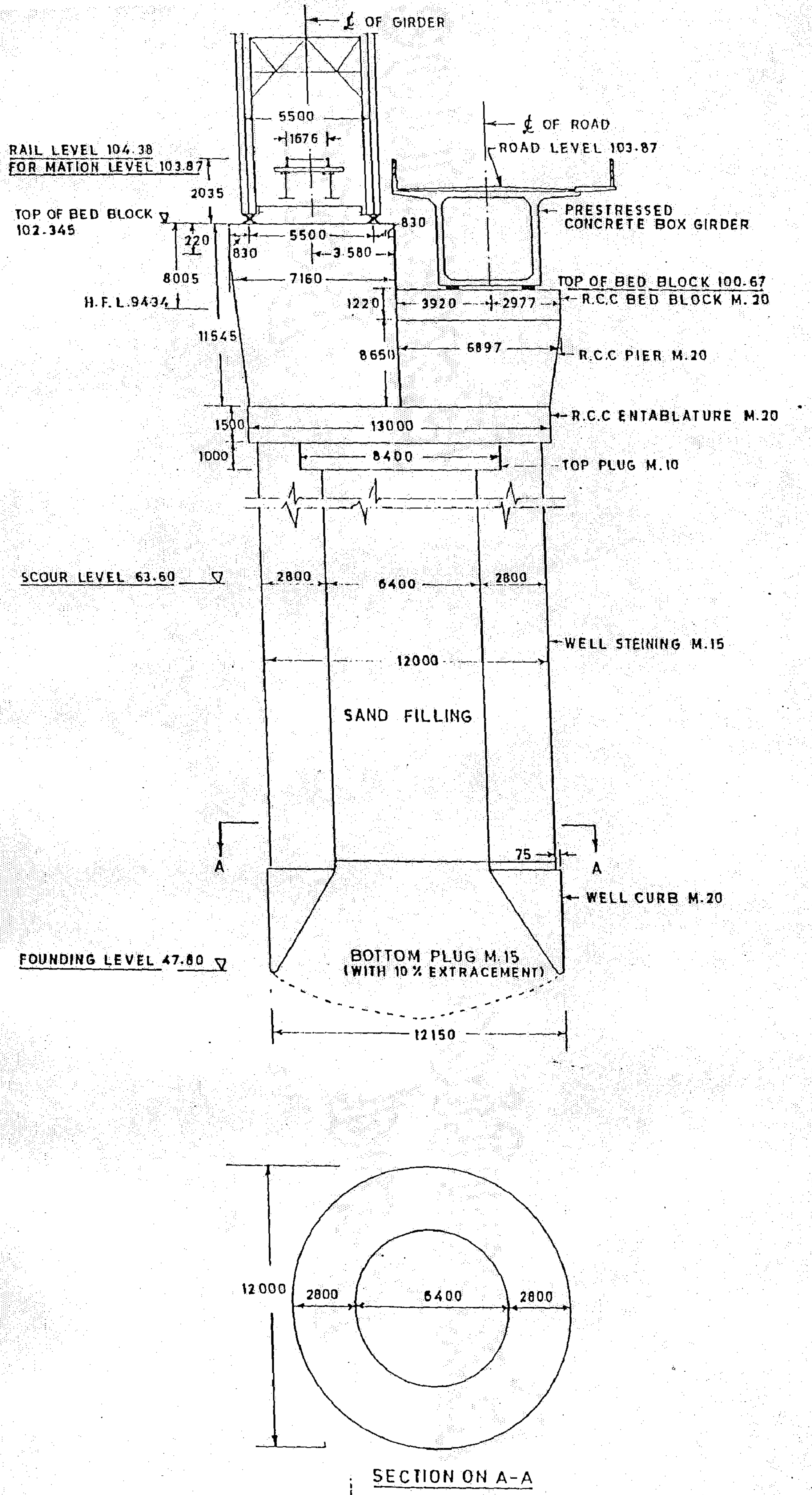


FIG. 10 — CROSS-SECTIONAL VIEW OF PIER (All Dimensions are in mm. except Levels which are in meter)

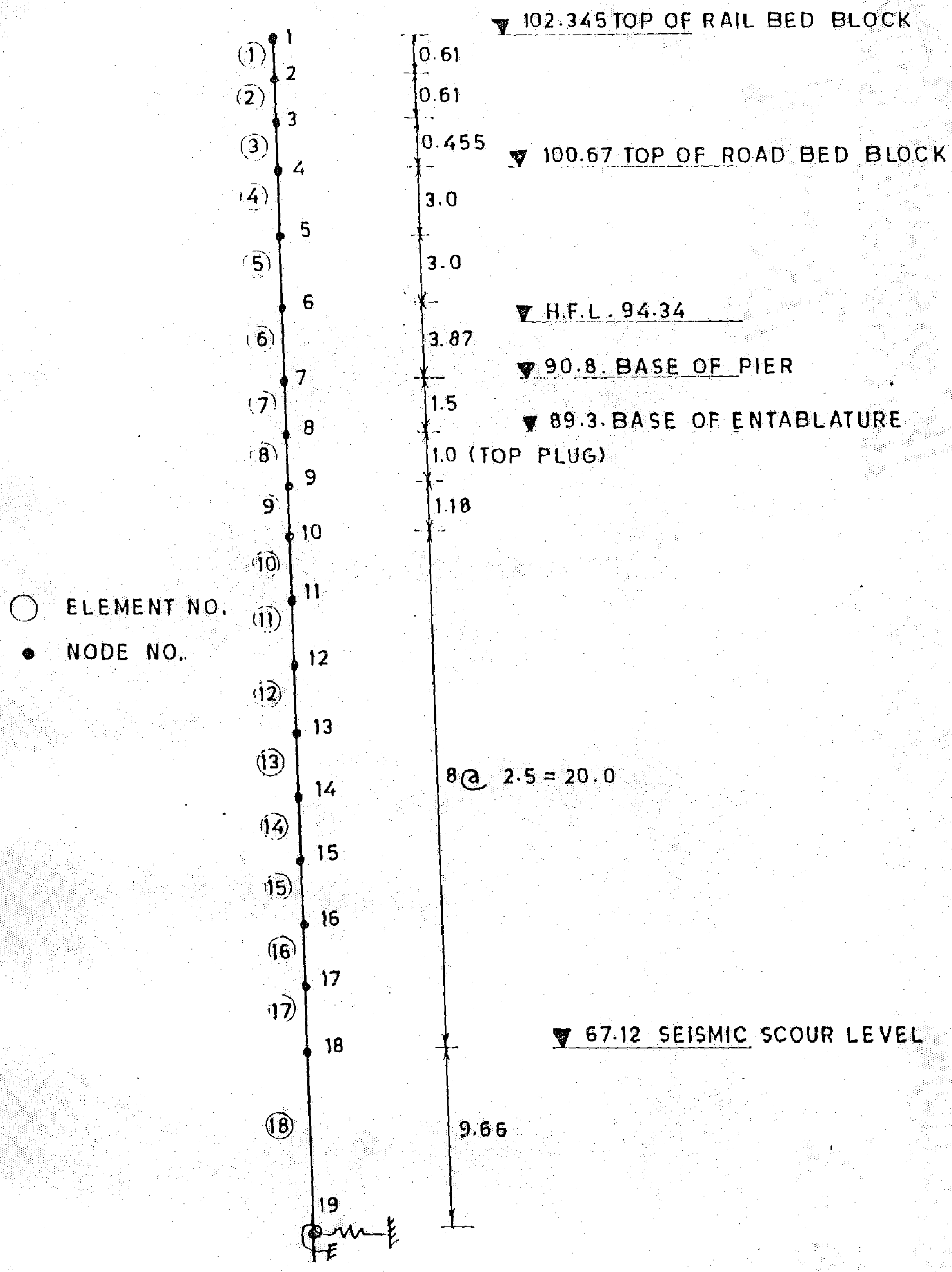


FIG.11– MATHEMATICAL MODEL OF A PIER IN LONGITUDINAL DIRECTION (ALL DIMENSIONS ARE IN Meter)

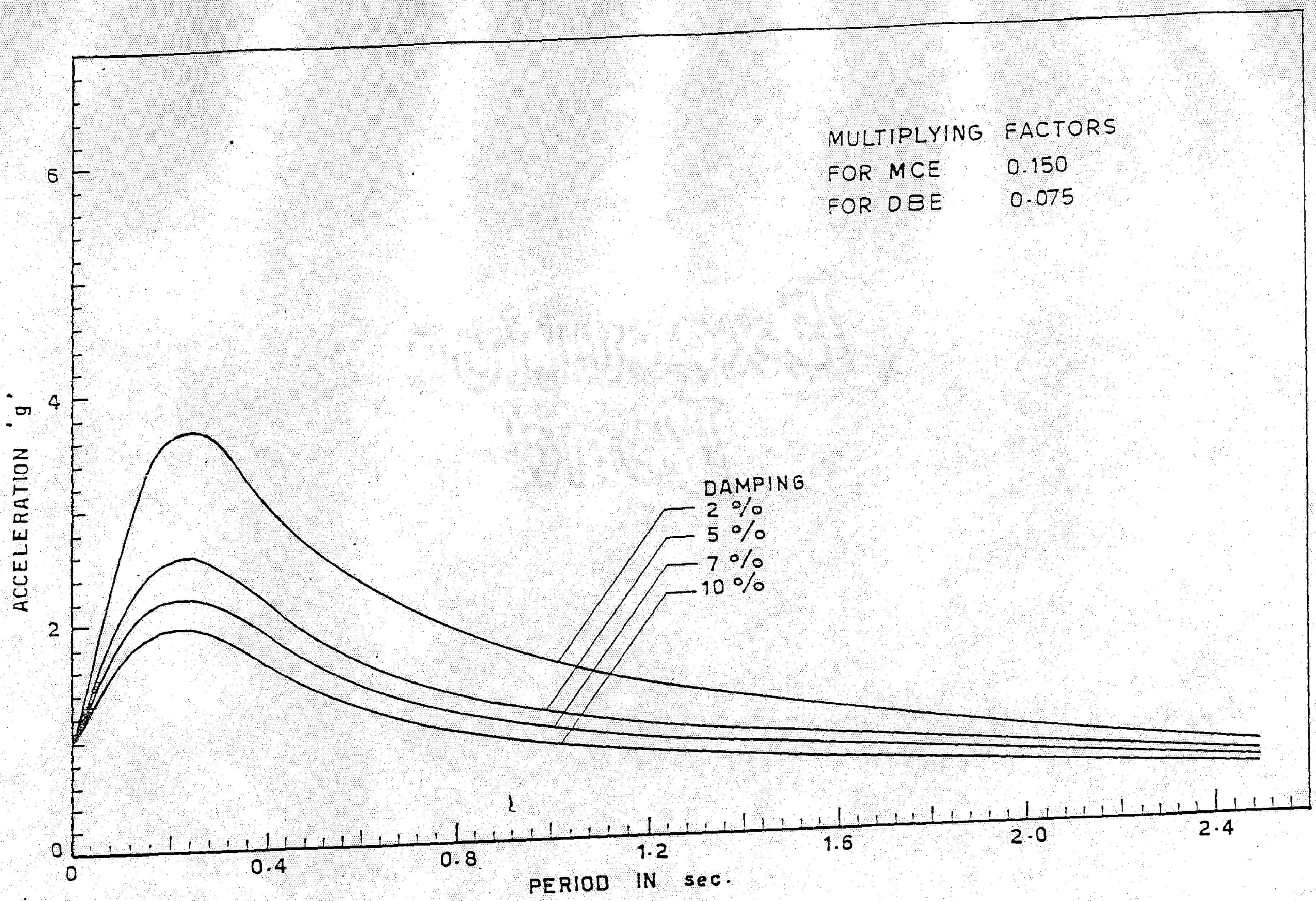


FIG.12 _ ACCELERATION RESPONSE SPECTRA FOR MCE AND DBE FOR GANDAK BRIDGE SITE

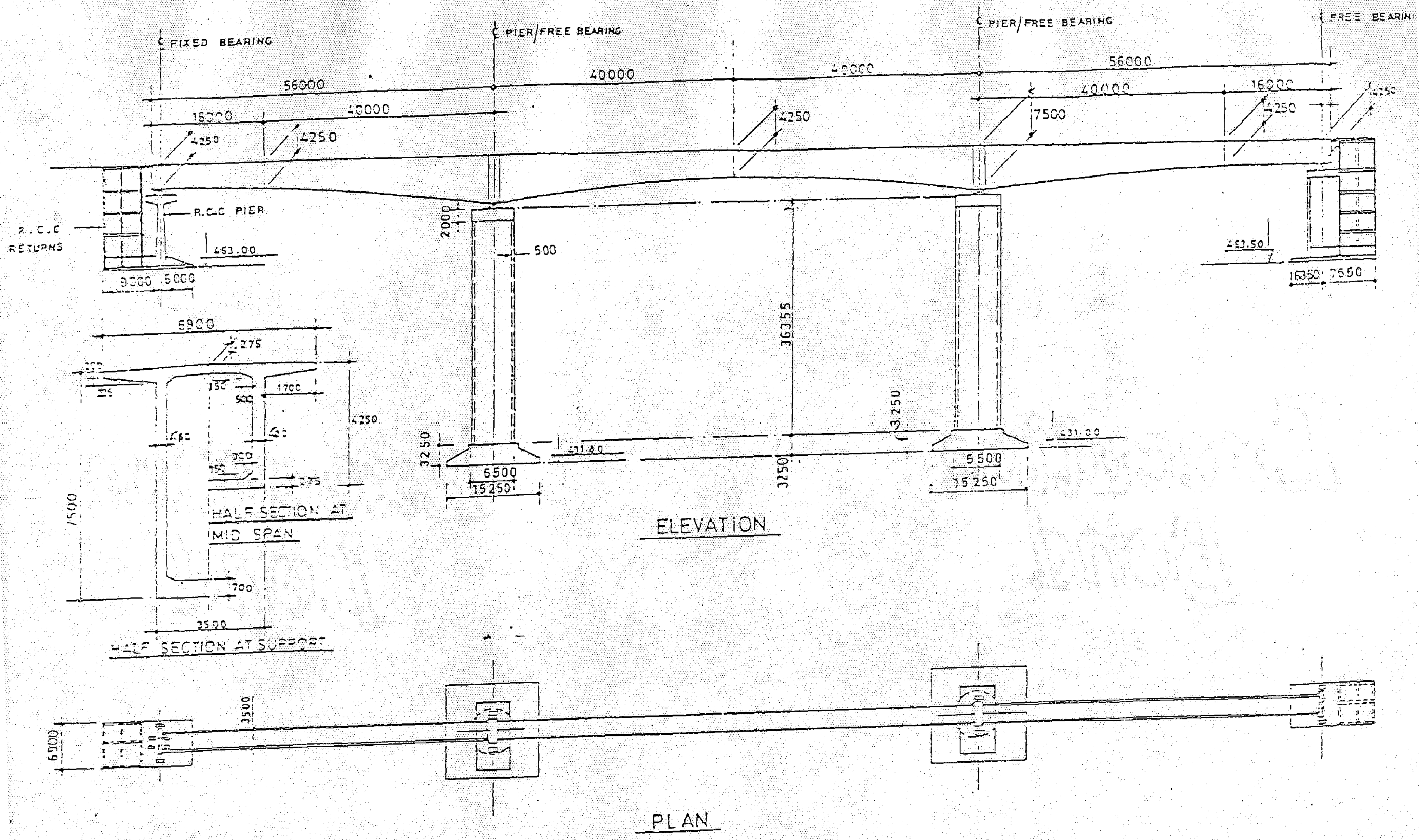


FIG.13 _ BRIDGE OVER RINGHAL KHAD

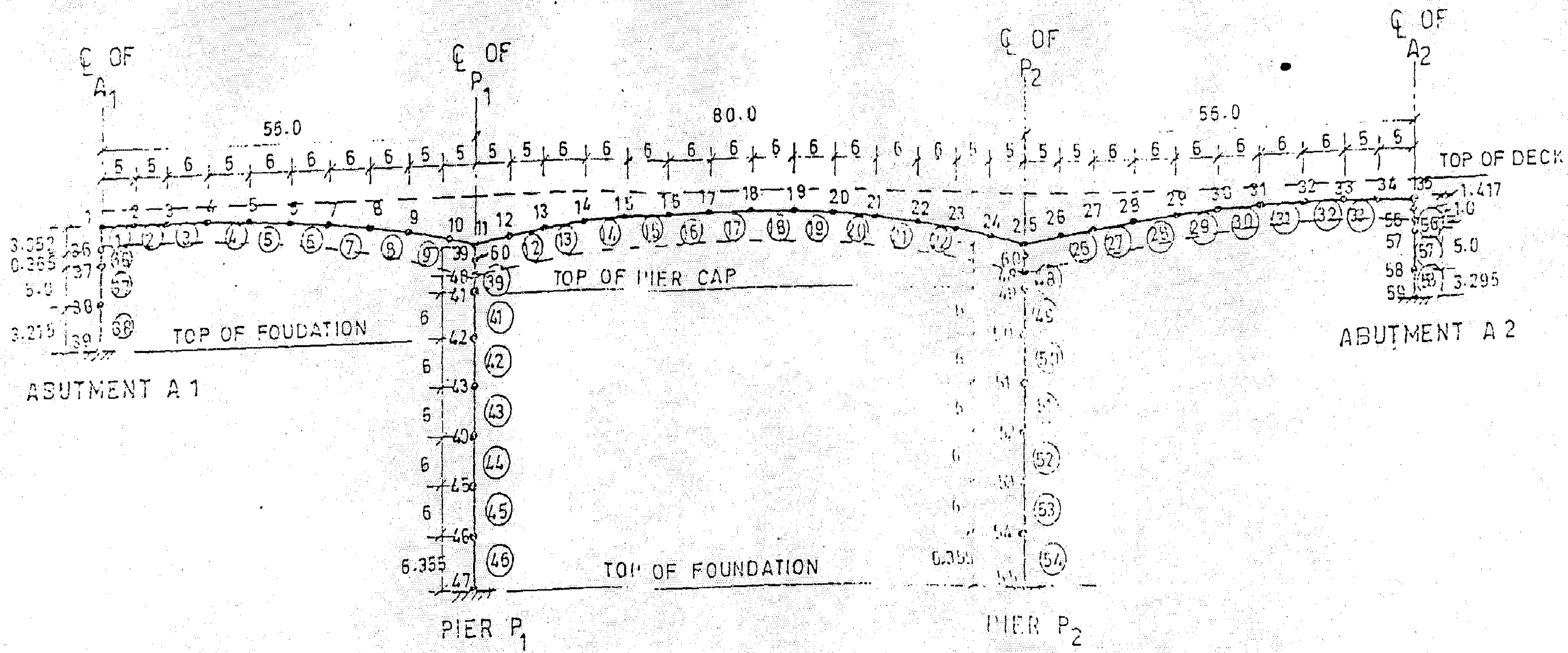


FIG. 14._ MATHEMATICAL MODEL OF BRIDGE OVER RINGHAL KHAD IN LONGITUDINAL DIRECTION

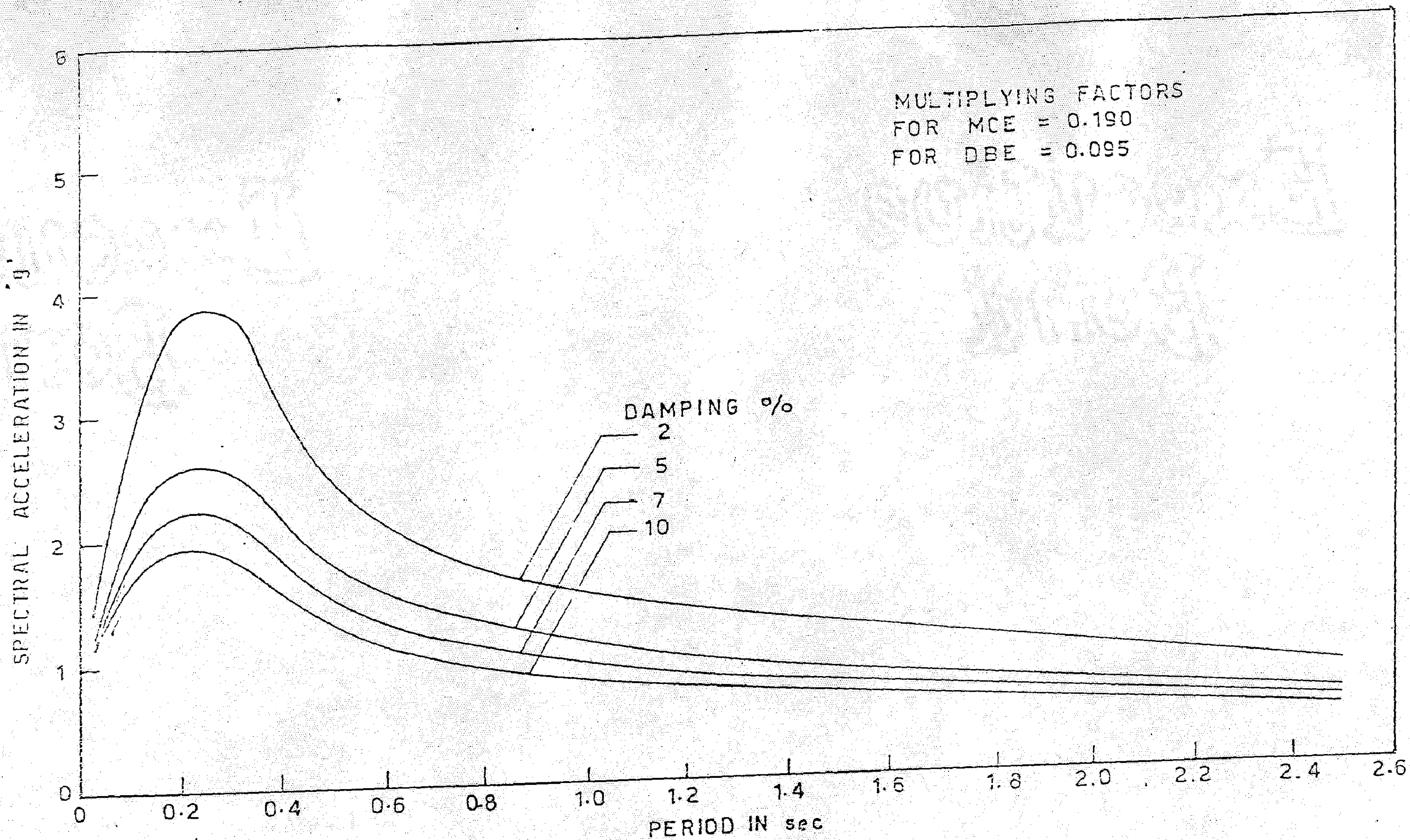


FIG.15_ ACCELERATION RESPONSE SPECTRUM FOR BRIDGE SITE AT RINGHAL