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Seismic Reliability Analysis of Cable Stayed Bridges Against First Passage Failure

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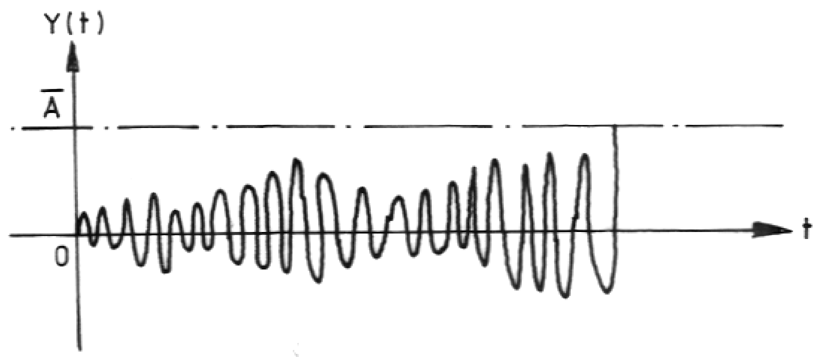
1. Introduction

In many structural applications, the ultimate purpose of using stochastic analysis is to determine the reliability of a structure, which has been designed to withstand the random excitations. The present study is concerned with one of the failures, which are the results of the dynamic response of the stable cable stayed bridges. If Y(t) is the dynamic response (either deflection, strain or stress) of the bridge at a critical point, the bridge may fail upon the occurrence of one of the following cases (Y.K. Lin, 1967):

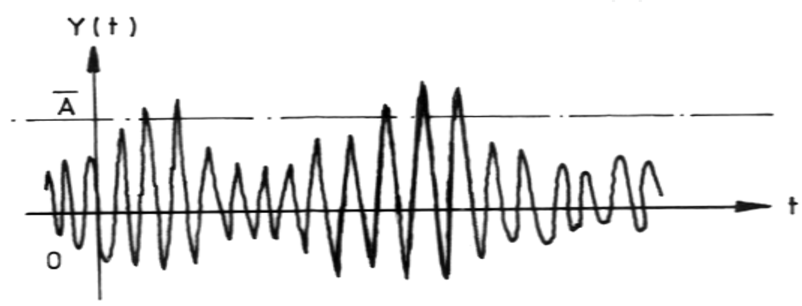
i. Y(t) reaches, for the first time, either an upper level A or lower level –B, where A and B are large positive numbers (Fig. 1(a)).
ii. Damage to the structure accumulated as Y(t) fluctuates at small or moderate excursion which are not large enough to cause a failure of the first type, and failure occurs when the accumulated damage reaches a fixed total.
iii. Failure may occur if a response spends too much of its time off range, i.e., it is. outside the set limits for more than a minimum fraction of its lifetime (Fig. 1(b))

The first case, which is called first passage failure (also called first excursion failure), is the objective of this chapter. According to definition, the first passage failure analysis means determining the probability that a prescribed threshold level (displacement, stress or other response level) will be exceeded, for the first time, during a fixed period of time. The first passage failure does not lead to the catastrophic failure of the bridge, but in view of serviceability consideration it is important. Therefore, the purpose of designing the structures against first passage failure is to reduce the probability of such failure, over the expected lifetime of the structure, to an acceptable level. In most random vibration problems, there is a probability close to unity that any given high response threshold level will be exceeded if the structure is excited for a long enough period of time.

Seismic response of cable stayed bridge due to the random ground motion is obtained in this chapter using frequency domain spectral analysis. The ground motion is assumed to be a partially correlated stationary random process. In the response analysis, the quasi-static response of the bridge deck produced due to the support motion is dully considered. The probability of first passage failure for the possibility of future earthquake is then presented for the analysis of various threshold levels (taken as fraction of yield stress) is considered. A parametric study is performed to show the effect of some important parameters such as threshold level, soil conditions, degree of correlation, angle of incidence of earthquake etc. on the reliability of cable stayed bridge against the first passage failure.



(a) Failure occurs when Y(t) first reaches the level Y(t) = $\bar{A}$.



(b) Failure occurs if Y(t) > $\bar{A}$ for more than an acceptable fraction of the total elapse time.

Fig. 1. Possible modes of failure under random excitation.

1.1 Brief review of earlier works

A brief review of the earlier works on different aspects of reliability analysis of structures especially cable supported bridges subjected to seismic forces / dynamic excitations is presented below in order to highlight the need for the present work.

1.1.1 Seismic response of cable supported bridges

The vibration of cable stayed bridges under the earthquake excitation has been a topic of considerable research for many years. The dynamic behaviour of cable stayed bridges, subjected to earthquake ground motion, has been studied by several researchers.

Morris (1974*)* utilized the lumped mass approach for the linear and non-linear dynamic responses of two dimensional cable stayed bridges due to sinusoidal load applied at a node. It was concluded that four mode solutions were sufficient for both linear and nonlinear solutions. However, a time increment of 10% of the period corresponding to the highest mode gave satisfactory numerical results for all the dynamic analyses carried out.

Abdel Ghaffer and Nazmy (1987) investigated the effects of three dimensionality, multi-support excitation, travelling waves and non-linearity on the seismic behaviuor of cable stayed bridges. Their studies indicated that non-linearities become more important as the span of the bridge increases, and the effect of multiple support seismic excitation is more pronounced for a higher structural redundancy.

Nazmy and Abdel Ghaffar (1991), (1992) studied the effects of non-dispersive travelling seismic wave and ground motion spatial variability on the response of cable stayed bridges considering the cases of synchronous and non-synchronous support motions due to seismic excitations. The responses were obtained by time history analysis. They concluded that depending on the dynamic properties of the local soils at the supports, as well as the soils at the surrounding bridge site, the travelling seismic wave effect should be considered in the seismic analysis of these bridges. Also, there is multi-modal contribution from several modes of vibration to the total response of the bridge, for both displacement and member forces.

Hong Hao (1998) analyzed the effects of various bridge and ground motion parameters on the required seating lengths for bridge deck to prevent the pull-off and -drop collapse using random vibration method. He analyzed two span bridge model with different span lengths and vibration frequencies and subjected to various spatially varying ground excitations. Ground motions with different intensities, different cross correlations and different site conditions were considered in the analysis.

Soyluk and Dumanoglu (2000) carried out asynchronous and stochastic dynamic analysis of a cable stayed bridge (Jindo Bridge) using finite element method. In the asynchronous dynamic analysis, various wave velocities were used for the traveling ground motion. They found that response in the deck obtained from asynchronous dynamic analysis are much higher than the response obtained by the stochastic analysis. Further, shear wave velocity of ground motion greatly influences the response of the Jindo cable stayed bridge.

1.1.2 Reliability analsysis of structures

Lin (1967), Bolotin (1965) and Crandall, Mark (1963), Abbas and Manohar (2005a, b; 2007) discussed various models of structural failure under random dynamic loading and classified them into the following types: (i) failure due to the first excursion of the response beyond a safe level; (ii) failure due to the response remaining above a safe level for too long a duration and (iii) failure due to the accumulation of damage.

All these types of failure are associated with the reliability estimate of Structures subjected to dynamic loading like earthquake. A number of investigations are reported in the literature on the reliability analysis of Structures against the aforementioned types of failure. Relatively recent ones and a few important old ones are reviewed here.

Konishi (1969) studied the safety and reliability of suspension bridges under wind and earthquake actions. He treated the suspension safety of the structure under random ground motion from the standpoint of threshold crossing of a specified barrier.

Vanmarcke (1975) dealt with the problem of the probability of first-passage beyond a threshold value by a time dependent random process. Barrier was classified into three distinct categories: a single barrier, a double barrier, and a barrier defined for the envelope of the random process. The assumption that barrier crossings are independent, so that they constitute a Poisson process, is nearly true for high barrier levels. But for relatively low barrier levels, the Poisson process assumption is in error, as it does not account for the clustering effect in case of a narrow-band process while for a wide-band process, the actual time spent in the unsafe domain is not considered. Vanmarcke suggested improvements in the Poisson process assumption to allow for the above effects.

Chern (1976) dealt with the reliability of a bilinear hysteretic system, subjected to a random earthquake motion, considering first excursion beyond a specified barrier and low-cycle fatigue.

Solomon and Spanos (1982) studied the structural reliability under a non-stationary seismic excitation, based on the first excursion beyond a specified barrier by the absolute value and envelope of the response process.

Schueller and stir (1987) reviewed various methods to calculate the failure probabilities of structural component or system in the light of their accuracy and computational efficiency.

They analyzed that for problems of higher dimensions, approximation techniques utilizing linearization like FOSM introduced considerable error. In view of these difficulties, they provided an alternate method to calculate the failure probabilities, which combines the advantages of both the importance sampling technique and the design point calculation.
Mebarki et al. (1990) presented a new method for reliability assessment, called Hypercone method based on the principles of level-2 methods of reliability analysis. The main aim of the method was to evaluate reliability index 'beta' and to deduce the values of the probability of failure by considering the whole geometry of the failure domain. The restrictions and the practical application of the method were discussed. The results reported showed that the value of probability of failure deduced from the Hasofer-Lind index beta, assuming linearity of the limit state surface, and from Monte Carlo Simulations are in accordance with those deduce from the Hypercone method.
Zhu (1993) reviewed several methods for computing structural system reliability. A discretization or cell technique for determining the failure probabilities of structural system is proposed. The gaussian numerical integration method is introduced to improve its computational accuracy and can be applied to gaussian or non-gaussian variables with linear or non-linear safety margin. Harichandran et al. (1996) studied the stationary and transient response analyses of the Golden Gate suspension bridge, and the New River Gorge and Cold Spring Canyon deck arch bridges subjected to spatially varying earthquake ground motion (SVEGM). They found that transient lateral displacements of the suspension bridge center span significantly overshoot the corresponding stationary displacements for the filtered Kanai-Tajami excitation power spectrum; this spectrum may therefore be unsuitable for analyzing very flexible structures.

2. Assumptions

The following assumptions are made for the First Passage Failure analysis:

i. It is assumed that the response process is stationary Gaussian with zero mean value.
ii. As in spectral analysis method, the evaluated response is always positive, therefore the single barrier level (called type B barrier, according to Crandall et al.1966) is used.
iii. It is assumed that the threshold level crossing occur independently according to a Poisson process.
iv. The structure is assumed to be linear and lightly damped.
v. The bridge deck (girder) and the towers are assumed to be axially rigid.
vi. The bridge deck, assumed as continues beam, does not transmit any moment to the towers through the girder-tower connection.
vii. Cables are assumed to be straight under high initial tensions due to the dead load and well suited to support negative force increment during vibration without losing its straight configuration.
viii. An appropriate portion of the cable is included (in addition to deck mass) in the dynamic analysis of the bridge deck and is assumed to be uniformly distributed over the idealized deck.
ix. Beam-column effect, in the stiffness formulation of the beam is considered for the constant axial force in the beam and its fluctuating tension in the cable is ignored. Further, cable dynamics is ignored for the bridge deck vibration, i.e., the tension fluctuations in the cables are assumed to be quasi-static, and do not introduce any nonlinearity in the system.

3. Seismic excitation

The seismic excitation is considered as a three component stationary random process. The earthquake ground motion is assumed as stationary random although in many cases it is assumed as a uniformly modulated non-stationary process. The response analysis remains the same for both cases. The response derived by assuming the process to be stationary can be multiplied by an envelope function to take care of the non-stationary. The components of the ground motion along an arbitrary set of orthogonal directions will be usually statistically correlated. However, as observed by Penzien and Watable (1975), the three components of ground motion along a set of principal axes are uncorrelated. These components, directed along the principal axes, are usually such that the major principal axis (u) is directed towards the expected epicenter, the moderate principal axis (w) is directed perpendicular to it (horizontally) and the minor principal axis (v) is directed vertically as shown in Fig.2. Nigam and Naranayan (1995) highlight the critical orientation of the alpha angle between the two sets of axes. Der Kiureghian (1996) developed the model for the coherency function describing spatial variability of earthquake ground motions. The model consists of three components characterizing three distinct effects of spatial variability, namely, the incoherence effect that arises from scattering of waves in the heterogeneous medium of the ground and their differential super positioning when arriving from an extended source, the wave-passage effect that arises from difference in the arrival times of waves at different stations, and the site-response effect that arises from difference in the local soil conditions at different stations. Abbas and Manohar (2002) developed a critical earthquake excitation models with emphasis on spatial variability characteristics of ground motion. In this study, the three components of the ground motion are assumed to be directed along the principal axes of the bridge x, y, z or shifted with an angle α. Each component is assumed to be a stationary random and partially correlated process with zero mean characterized by a psdf. The psdf of the ground acceleration is defined by Clough and Penzien (1975) as

$$S_{\ddot{f}_g \ddot{f}_g} = \left|H_1(i\omega)\right|^2 \left|H_2(i\omega)\right|^2 S_o \tag{1}$$

in which S_o is the spectrum of the white noise bedrock acceleration; $\left|H_1(i\omega)\right|^2$ and $\left|H_2(i\omega)\right|^2$ are the transfer functions of the first and the second filters representing the dynamic characteristic of the soil layers above the bedrock, where

$$\left|H_1(i\omega)\right|^2 = \frac{1+\left(2\xi_g \omega / \omega_g\right)^2}{\left[1-\left(\omega/\omega_g\right)^2\right]^2 + \left(2\xi_g \omega/\omega_g\right)^2} \tag{2}$$

$$\left|H_2(i\omega)\right|^2 = \frac{\left(\omega/\omega_f\right)^4}{\left[1-\left(\omega/\omega_g\right)^2\right]^2 + \left(2\xi_g \omega/\omega_g\right)^2} \tag{3}$$

in which ω_g , ξ_g are the resonant frequency and damping ratio of the first filter, and ω_f , ξ_f are those of the second filter. The clough and Penzien is a double filter for spectral density of the ground acceleration for which the corresponding displacement psdf does not become unrealistic. In case of Kanai-Tajimi spectrum, although the psdf of acceleration is simpler but it has problem that the corresponding displacement psdf become undefined at zero frequency.

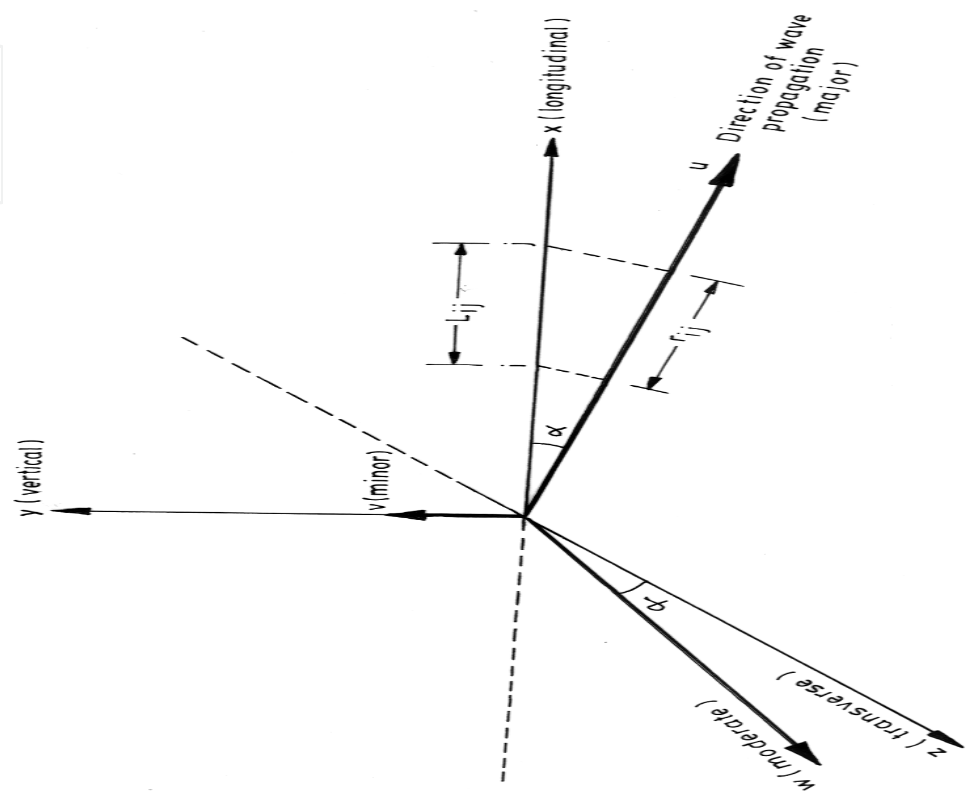


Fig. 2. Principal directions of the bridge(x,y,z) and the ground motion(u,v,w)

The cross spectrum between the random ground motions $\ddot{f}_{g_i}$ and $\ddot{f}_{g_j}$ at two stations i and j is described by that given by Hindy and Novak (1980) as

$$S_{\ddot{f}_{gi}\ddot{f}_{gh}}\left(r_{ij},\omega\right)=S_{\ddot{f}_g\ddot{f}_g}\left(\omega\right)\rho_{ij}\left(\omega\right) \tag{4}$$

in which $S_{\ddot{f}_g\ddot{f}_g}(\omega)$ local spectrum of ground acceleration as given in Eqn.(1) which is assumed to be the same for all supports and $\rho_{ij}(\omega)$ is the cross correlation function (coherence function) of the ground motion between two excitation points i, j and is represented by

$$\rho_{ij}\left(\omega\right)=\exp\left[-c\left(\frac{r_{ij}\omega}{2\pi V_s}\right)\right] \tag{5}$$

in which r_{ij} is the separation distance between stations i and j measured in the direction of wave propagation; c is a constant depending upon the distance from the epicenter and the inhomogeneity of the medium; V_s is the shear wave velocity of the soil; and ω is the frequency (rad/sec) of the ground motion.

For one sided spectrum it is well known that

$$\sigma_{\ddot{f}_g}{}^2 = S_o\left[\int_0^{\alpha} \left|H_1(i\omega)\right|^2 \left|H_2(i\omega)\right|^2 d\omega\right] \tag{6}$$

$\sigma_{\ddot{f}_g}{}^2$ is the variance of ground acceleration. The Modified Mercalli intensity of an earthquake Is is a measures of the strength of the shaking at a particular location and will vary with distance, substrate conditions and other factors. The empirical relation between the standard deviation of peak ground acceleration and earthquake intensity Is is given as

$$\sigma_{\ddot{f}_g}{}^2 = 10^{(I_s/3-0.5)} / K^* \tag{7}$$

where K^* is a peak factor given as

$$K^* = K' + \frac{0.5772}{K'} \ ; \ K' = \sqrt{2\ln(N_0 T)} \tag{8}$$

The empirical relation between the magnitude of earthquake and intensity of earthquake is given as

$$M = 1.3 + 0.6\, I \tag{9}$$

In Eqn.(9), N_0 is the mean rate of zero crossing and is given by

$$N_0 = \frac{1}{2\pi}\sqrt{\int_0^{\alpha} \omega^2 S_{\ddot{f}_g}(\omega)d\omega \Big/ \int_0^{\alpha} S_{\ddot{f}_g}(\omega)d\omega} \tag{10}$$

By defining the filter characteristics ω_g ,ξ_g , ω_f , ξ_f and specifying a standard deviation of the ground acceleration $\sigma_{\ddot{f}_g}$, the psdf of the ground acceleration can be completely defined. The psdfs $S_{f_g f_g}(\omega)$ and $S_{\dot{f}_g \dot{f}_g}(\omega)$ of the ground displacement and velocity respectively are related to $S_{\ddot{f}_g \ddot{f}_g}(\omega)$ by

$$S_{f_g f_g}(\omega) = S_{\ddot{f}_g \ddot{f}_g}(\omega) / \omega^4$$

$$S_{\dot{f}_g \dot{f}_g}(\omega) = S_{\ddot{f}_g \ddot{f}_g}(\omega) / \omega^2 \tag{11}$$

The ground motion is represented along the three principal directions (u,v,w) by defining ratio R_u , R_v and R_w along them such that

$$\ddot{u}_g(t) = R_u \ddot{f}_g(t) \ ; \ \ddot{v}_g(t) = R_v \ddot{f}_g(t) \ ; \ \ddot{w}_g(t) = R_w \ddot{f}_g(t) \tag{12}$$

and the psdfs of the ground acceleration in the principal directions of the ground motion (u,v,w) can be defined as

$$S_{\ddot{u}_g \ddot{u}_g}(\omega) = R_u{}^2 S_{\ddot{f}_g \ddot{f}_g}(\omega) \ ; \ S_{\ddot{v}_g \ddot{v}_g}(\omega) = R_v{}^2 S_{\ddot{f}_g \ddot{f}_g}(\omega) \ ; \ S_{\ddot{w}_g \ddot{w}_g}(\omega) = R_w{}^2 S_{\ddot{f}_g \ddot{f}_g}(\omega) \tag{13}$$

so that

$$\sigma_{\ddot{u}_g}{}^2 = R_u{}^2\sigma_{\ddot{f}_g}{}^2 \quad ; \quad \sigma_{\ddot{v}_g}{}^2 = R_v{}^2\sigma_{\ddot{f}_g}{}^2 \quad ; \quad \sigma_{\ddot{w}_g}{}^2 = R_w{}^2\sigma_{\ddot{f}_g}{}^2 \tag{14}$$

When the angle of incidence of the ground motion with respect to the principal direction of the bridge is defined as α , the ground motions along the principal directions of the bridge (x,y,z) are defined as

$$\begin{aligned} \ddot{x}_g(t) &= \ddot{u}_g(t)\cos\alpha - \ddot{w}_g(t)\sin\alpha \\ \ddot{z}_g(t) &= \ddot{u}_g(t)\sin\alpha - \ddot{w}_g(t)\cos\alpha \\ \ddot{y}_g(t) &= \ddot{v}_g(t) \end{aligned} \tag{15}$$

The psdfs of the ground accelerations along x,y,z can be written as

$$S_{\ddot{x}_g\ddot{x}_g} = \cos^2\alpha\ S_{\ddot{u}_g\ddot{u}_g} + \sin^2\alpha\ S_{\ddot{w}_g\ddot{w}_g} = R^2{}_x S_{\ddot{f}_g\ddot{f}_g}$$

$$S_{\ddot{z}_g\ddot{z}_g} = \sin^2\alpha\ S_{\ddot{u}_g\ddot{u}_g} + \cos^2\alpha\ S_{\ddot{w}_g\ddot{w}_g} = R^2{}_z S_{\ddot{f}_g\ddot{f}_g}$$

$$S_{\ddot{y}_g\ddot{y}_g} = S_{\ddot{v}_g\ddot{v}_g} = R_y{}^2 S_{\ddot{f}_g\ddot{f}_g} \tag{16}$$

where R_x, R_y ,and R_z are the ratios of the ground motion along the principal axes of the bridge and given as

$$\begin{aligned} R_x{}^2 &= R_u{}^2\cos^2\alpha + R_w{}^2\sin^2\alpha \\ R_z{}^2 &= R_u{}^2\sin^2\alpha + R_w{}^2\cos^2\alpha \\ R_y{}^2 &= R_v{}^2 \end{aligned} \tag{17}$$

and R_u , R_v , and R_w are ratios of the ground motion along the principal directions of the ground motion (u,v,w) as shown in Fig.2.

4. Distribution function of magnitude of earthquake

Two types of distribution functions of the magnitude of earthquake are considered in the study.

4.1 Exponential distribution

This type of probability distribution function of the magnitude of earthquake is based on the Gutenberg – Ritcher Recurrence law (Kagan, 2002a)

$$\log \lambda_m = a - b\,m \tag{18a}$$

$$\lambda_m = 10^{a\text{-}b\,m} = \exp(\alpha - \beta\, m) \tag{18b}$$

where λ_m is the mean annual rate of exceedence of magnitude m ; 10^a is the mean yearly number of earthquakes greater than or equal to zero,; and b describes the relative likelihood of large or small earthquakes. Eqn.(18b) implies that the magnitudes are exponentially distributed. Based on Eqn. (18b), the probability density function (PDF) is given by

$$P_M(m) = \beta e^{-\beta(m - m_0)} \tag{19}$$

where, β = 2.303b, and m_0 is the lower threshold magnitude of earthquake, earthquakes smaller than which are eliminated, and m is the magnitude of earthquake.
The cumulative distribution function (CDF) of magnitude of earthquake for exponential distribution is given by the following expression

$$F_M(m) = \{1-\exp(-\beta(m-m_0))\} \tag{20}$$

4.2 Gumbel type-I distribution

The cumulative distribution function of magnitude of earthquake for Gumbel type-I distribution is given by the following expression

$$F(m) = \mathrm{Exp}(-\exp - \alpha\,(m - u)) \tag{21}$$

where α and u are the parameters for Gumbel Type-1 distribution given by

$$\bar{M} = u + 0.5772/\alpha \tag{22a}$$

$$\sigma_m^2 = \pi^2/6\,\alpha^2 \tag{22b}$$

in which $\bar{M}$ and σ_m are the mean and standard deviation of the magnitudes of earthquake respectively.

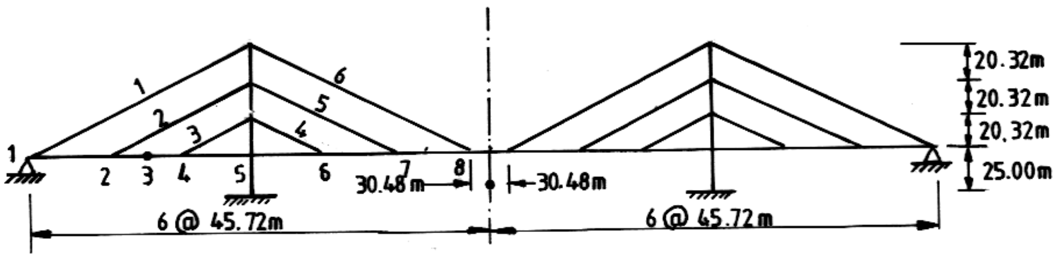
5. Theoretical analysis

5.1 Free vibration analysis of cable stayed bridge deck (girder)

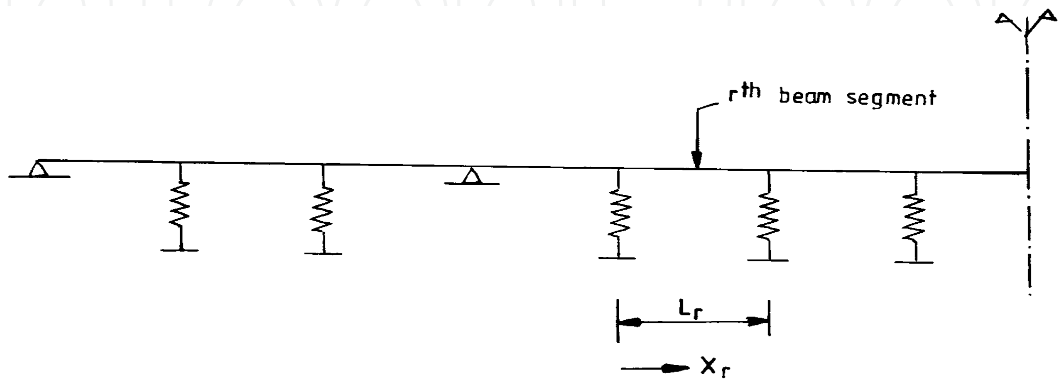
The bridge deck, as shown in Figs.3(a) is idealized as a continuous beam over the outer abutments and the interior towers, and the effect of cable is taken as vertical springs at the points of intersections between the cables and the bridge deck shown in Fig.3(b). Further, the effect of the spring stiffness is taken as an additional vertical stiffness to the flexural stiffness of the bridge.
Referring to Fig.3(c) and Fig.3(d), the fluctuations of tension in the cable at any instant of time (t) can be written as

$$h_i(t) = K_i V(x_i, t)\sin\psi_i + K_i \Delta_j(t)\cos\psi_i \tag{23}$$

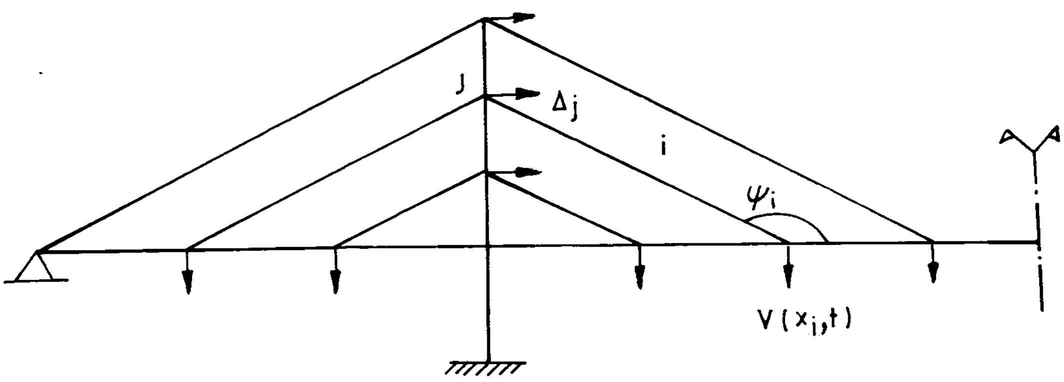
where, $K_i = E_c A_i / L_i$ is the stiffness of the i^{th} cable; $V(x_i,t)$ is the displacement of the girder at time t at the joint of the i^{th} cable with the girder; $\Delta_j(t)$ is the horizontal sway of the tower at the i^{th} tower cable joint connecting the i^{th} cable; ψ_i is the angle of inclination of the i^{th} cable to the horizontal (measured clockwise from the cable to the horizontal line as shown in Fig.3.3(c); A_i, L_i are the cross-sectional area and the length of the i^{th} cable respectively and E_c is the equivalent modulus of elasticity of the straight cable under dead loads.



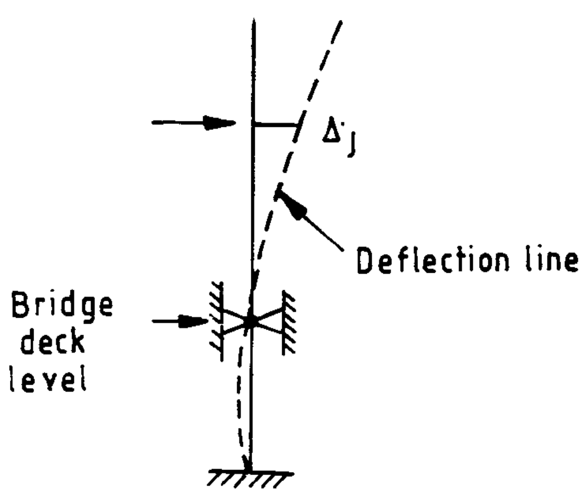
(a) Harp type cable stayed bridge considered for parametric study(Bridge-I)



(b) Idealization of the bridge deck



(c)Displacement due to the fluctuation of the i^{th} cable



(d)Main system and the displacement of the tower

Fig. 3. (a, b, c, d) Problem identification

Following Eqn.(23), the changes in tensions in the array of cables can be put in the following matrix form

$$\{h\}_{Ncx1} = \mathbf{(A)}_{NcxNd}\{V\}_{Ndxl} + \mathbf{(B)}_{Ncx1}\{\Delta\}_{Nt\,x1} \quad (24)$$

Where Nc is the number of cable (or pair of cable in case of two-phase cable stayed bridge); Nd is the number of unrestrained vertical degrees of freedom of the girder at the cable-girder joints; Nt is the number of horizontal tower degrees of freedom at the cable-tower joints; {V}and {Δ} are the girder and the tower displacement vectors; {h} is the vector of incremental cable tensions; **(A)** and **(B)** are the matrices which are formed by proper positioning of the elements $K_i \sin\psi_i$ and $K_i \cos\psi_i$ respectively.
The deflections of the tower at the cable joints can be obtained by assuming that the tower behaves like a vertical beam fixed at the bottom end and constrained horizontally at the level of the bridge deck and subjected to the transverse forces h_i (t) $\cos\psi_i$ (i=1, Nc) at the cable tower joints as shown in Fig.3(b) and are given by

$$\{\Delta\} = (C)\{h\} \quad (25)$$

where the elements of the matrix **(C)** can be easily obtained from the deflection equation of the tower (vertical beam) subjected to concentrated load, as mentioned above, using standard structural analysis procedure. Eliminating {Δ} from Eqns.(24) and (25), the relation between the vectors of incremental cable tensions and girder deflections may be written as

$$\{h\} = ((I)-(B)\,(C))^{-1}\,(A)\{V\} \quad (26)$$

where **(I)** is the unit matrix of order Nc.
Premultiplying both sides of Eqn. (26) by a diagonal matrix **(D)** of order Nc, where the diagonals consists of the terms of $\sin\psi_i$ (i= 1 to Nc), Eqn. (26) can be written as

$$\{h_{v\cdot b}\} = \mathbf{(K_{c,v})}\,\{V\} \quad (27)$$

where $\{h_{v,b}\}$ is the vector containing the vertical components of incremental cable tensions, and $\mathbf{(K_{c,v})} = \mathbf{((D)(I)\text{-}(B)(C))^{-1}(A)}$ is the stiffness matrix of the bridge contributed by the cables in vertical vibration.
The equation of motion for the relative vertical vibration $Y(x_r,t)$ of the beam segment r of the idealized deck with constant axial force N_r , neglecting the shear deformation and rotary moment of inertia is given by

$$E_d I_r \frac{\partial^4 Y}{\partial X_r{}^4} + N_r \frac{\partial^2 Y(x_r,t)}{\partial X_r{}^2} + C_r \frac{\partial Y(x_r,t)}{\partial t} + \frac{\bar{W}_r}{g}\frac{\partial^2 Y(x_r,t)}{\partial t^2} = P(x_r,t) \quad \text{for r=1,2,3 ----. } N_b \quad (28)$$

in which E_d and I_r are the modulus of elasticity of the bridge deck and vertical moment of inertia of the beam segment r of the deck respectively.
$P(x_r,t)$ is defined as the load induced due to seismic excitations at different support degree of freedom and is given by

$$P(X_r,t) = -\frac{\bar{W}_r}{g}\sum_{j=1}^{8} g_{jr}(x_r)\,\ddot{f}_j(t) \quad (29)$$

where $\ddot{f}_j(t)$, j=1,2, --------, 8 are the accelerations at the different support degrees of freedom and $g_{jr}(x_r)$ is the vertical displacement of the r^{th} segment of the bridge deck due to unit displacement at the j^{th} degree of the supports. In Eqns. (28) and (29), it is assumed that for lightly damped system, the effect of damping term associated with quasi-static movement of the supports is negligible (Clough and Penzien, 1993).
The expression for n^{th} mode shape (undamped) for vertical vibration of the r^{th} segment of the bridge deck is given by (Chatterjee, 1992)

$$\phi_n(x_r) = A_{nr}\cos\beta_{nr}x_r + B_{nr}\sin\beta_{nr}x_r + C_{nr}\cosh\gamma_{nr}x_r + D_{nr}\sinh\gamma_{nr}x_r \tag{30}$$

where A_{nr}, B_{nr}, C_{nr} and D_{nr} are the integration constants expressed in terms of n^{th} natural frequency of vertical vibration ω_{bn} and

$$\beta_{nr} = \sqrt{\frac{N_r(Z_{nr}+1)}{2E_dI_r}} \;;\; \gamma_{nr} = \sqrt{\frac{N_r(Z_{nr}-1)}{2E_dI_r}}$$

where

$$Z_{nr} = \sqrt{1+\frac{4E_dI_r\bar{W}_r/g\omega_{bn}^{2}}{N_r^{2}}}$$

in which the suffix r is used to mean the r^{th} segment of the beam. The origin for the r^{th} segment is fixed at the left end as shown in Fig.3.3(c).Utilizing Eqn.(30), a relation between end displacements (vertical deflection and slope) and end forces(shear forces and bending moments) for the r^{th} segment may be written as

$$\{F\}_r = (\mathbf{K})_r\{x_r\} \tag{31}$$

where $\{F\}_r$ and $\{x_r\}$ are the end forces and end displacement vectors and $(\mathbf{K})_r$ is the flexural dynamic stiffness matrix of the r^{th} beam segment. The integration constants A_{nr}, B_{nr}, etc. are related to the end displacements as

$$\{C\}_r = (\mathbf{T})_r\{x_r\} \tag{32}$$

where $\{C\}_r$ is the vector of integration constants containing A_{nr} etc., and $(\mathbf{T})_r$ is the matrix integration constants. The sign conventions used in the dynamic stiffness formulation are shown in Fig.4. The explicit expressions for the elements of $(\mathbf{K})_r$ and $(\mathbf{T})_r$ are given by Chatterjee (1992). Assembling the stiffness $(\mathbf{K})_r$ for each element (r) and adding the vertical stiffness due to cables $(\mathbf{K}_{C,V})$, the overall stiffness of the bridge $(\mathbf{K})$ is obtained. The condition for the free vibration of the bridge deck may then be written as

$$(\mathbf{K})\{U\}=\{0\} \tag{33}$$

where {U} is the unknown end displacement vector for the beam corresponding to the dynamic degrees of freedom Fig.4. Using Eqn.(33) leads to

$$\det(K) = 0 \tag{34}$$

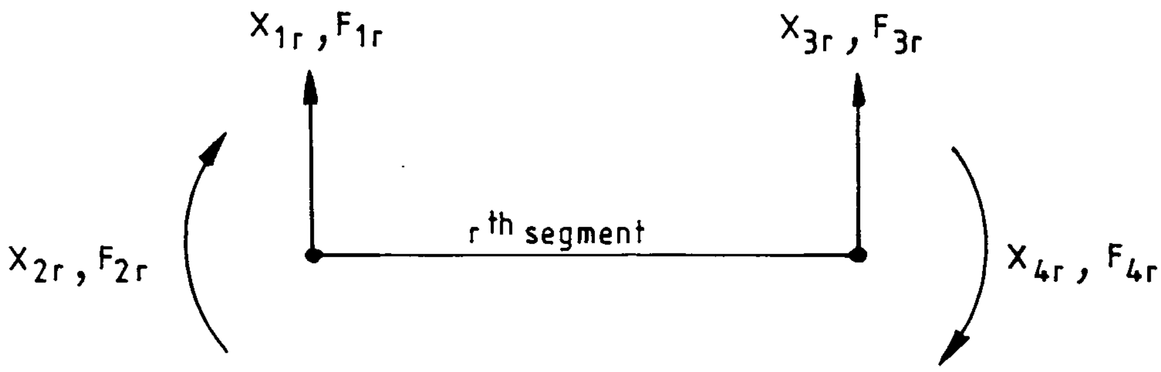


Fig. 4. Sign conventions used in the dynamic stiffness formulation.

Using Regula Falsi approach, the natural frequencies for the system are determined from the solution of Eqn.(33). Once the natural frequencies are obtained, mode shapes can be known through the use of Eqns.(32 & 33).

5.2 Modal transformation

The modal transformation of the relative vertical displacement $y(x_r,t)$ for any point in the r^{th} deck segment is given as

$$y(x_r,t)=\sum_{n=1}^{\alpha}\varphi_n(x_r)q_n(t) \quad r = 1,2,----,N_b \tag{35}$$

in which $\phi_n(x_r)$ is the n^{th} mode shape of the r^{th} beam segment of the bridge deck and $q_n(t)$ is the n^{th} generalized coordinate. Substituting Eqn.(35) into Eqn.(28), multiplying by $\phi_m(x_r)$, integrating w.r.t. L_r and using the orthogonality of the mode shapes leads to

$$\ddot{q}_n(t)+2\xi_n\omega_n\dot{q}_n(t)+\omega_n^2 q_n(t)=\bar{P}_n(t) \quad n =1, ----, M \tag{36}$$

in which ζ_n and ω_n are the damping ratio and the natural frequency of the n^{th} vertical mode; M is the number of modes considered and $\bar{P}_n(t)$ is the generalized force given as

$$\bar{P}_n(t)=\sum_{j=1}^{8}R_{jn}(x_r)\ddot{f}_j(t) \tag{37}$$

where R_{jn} is the modal participation factor given by

$$R_{jn}=-\frac{\sum_{r=1}^{N_b}\frac{\bar{W}_r}{g}\int_0^{L_r}g_{jr}(x_r)\phi_n(x_r)dx_r}{\sum_{r=1}^{N_b}\frac{\bar{W}_r}{g}\int_0^{L_r}\phi_n^2(x_r)dx_r} \tag{38}$$

in which $g_{jr}(x_r)$, the quasi static function, is the vertical displacement of the r^{th} beam segment of the bridge deck due to unit displacement given in the j^{th} direction of support movement.

Eqn.(37) can be put in the following matrix form

$$\bar{P}_n(t) = [G_n]\{f\} \tag{39}$$

in which

$[G_n]=\{G_{1n}$ ---------$G_{8n}\}$ and $\{f\}^T = \left\{ \ddot{f}_1(t) ------ \ddot{f}_8(t) \right\}$

where $[G_n]$ is the generalized force coefficients for the nth mode and can be obtained by Eqns.(36 & 37) i.e. $G_{1n}=R_{1n}$, -----, $G_{8n} = R_{8n}$.

5.3 Spectral analysis

5.3.1 Evaluation of the relative displacement

Applying the principles of modal analysis, the nth generalized coordinate in frequency domain can be written as

$$q_n(\omega) = H_n(\omega)\bar{P}_n(\omega) \tag{40}$$

in which $H_n(\omega)$ is the nth modal frequency response function given by

$$H_n(\omega) = \left[\left(\omega_n{}^2 - \omega^2\right) + i\left(2\xi_n\omega_n\omega\right)\right]^{-1} \tag{41}$$

Similarly, the mth generalized coordinate can be written as

$$q_m(\omega) = H_m(\omega)\bar{P}_m(\omega) \tag{42}$$

The cross power spectral density function between the two generalized coordinate $q_n(\omega)$ and $q_m(\omega)$ is given by

$$S_{q_nq_m}(\omega) = H_n(\omega)H_m(\omega)S_{\bar{P}_n\bar{P}_m} \tag{43}$$

$H_n(\omega)$ denotes the complex conjugate of the $H_n(\omega)$ and $S_{\bar{P}_n\bar{P}_m}$ can be written in the matrix form as

$$S_{\bar{P}_n\bar{P}_m} = [G_n]\left[S_{ff}\right][G_m]^T \tag{44}$$

$[S_{ff}]$ is the psdf matrix for the ground motion inputs (of size 8x 8) which are the support accelerations i.e. $\ddot{f}_1(t)$, $\ddot{f}_2(t)$, $\ddot{f}_3$, $\ddot{f}_4$, $\ddot{f}_5$, $\ddot{f}_6$, $\ddot{f}_7$, $\ddot{f}_8$.

Any element of the matrix $[S_{ff}]$ may be written in terms of the psdf of ground acceleration $S_{\ddot{f}_g\ddot{f}_g}(\omega)$ by using the coherence function Eqn.(5), the ratio between the three components of the ground motion (R_u:R_v:R_w) and Eqns.(16 & 17) as explained earlier. For example (1,4) and (5,8) can be written in the form

$$S_{\ddot{f}_1\ddot{f}_4} = \rho_{14}(\omega) R_y{}^2 S_{\ddot{f}_g\ddot{f}_g} \tag{45}$$

$$S_{\ddot{f}_5\ddot{f}_8} = \rho_{58}(\omega) R_x{}^2 S_{\ddot{f}_g\ddot{f}_g} \tag{46}$$

Using the expression given in Eqn.(42), the elements of the matrix [S_{qq}] may be formed which has the dimension of M x M. Since the relative displacement $y(x_r,t)$ is given by

$$y(x_r,t) = \sum_{n=1}^{M} \phi_n(x_r) q_n(t) = \left[\phi(x_r)\right]_{(1\times m)} \{q\}_{(1\times m)} \tag{47}$$

The psdf of the response $y(x_r,t)$ is given by

$$S_{yy}(x_r,\omega) = \left[\phi(x_r)\right]\left[S_{qq}\right]\left[\phi(x_r)\right]^T \tag{48}$$

5.3.2 Evaluation of the quasi-static displacement

The quasi-static component of the vertical displacement at any point in the r[th] deck segment at time (t) is given as

$$g(x_r,t) = \sum_{j=1}^{8} g_{jr}(x_r) f_j(t) = [Q]\{\bar{f}\} \tag{49}$$

where

$$[Q] = \{\, g_{1r}(x_r)\ g_{2r}(x_r)\ \text{------------}\ g_{8r}(x_r)\}; \quad \left\{\bar{f}\right\}^T = \{f_1(t)\ \ f_2(t)\ \ \text{---------}\ f_8(t)\}$$

$g_{jr}(x_r)$ is the vertical displacement at any point in the r[th] beam segment of the bridge deck due to unit movement of the j[th] support d.o.f. The psdf of the quasi-static displacement at any point in the r[th] deck segment is given by

$$S_{gg}(x_r,\omega) = [Q]\left[S_{\bar{f}\,\bar{f}}\right]\ [Q]^T \tag{50}$$

where $\left[S_{\bar{f}\,\bar{f}}\right]$ is the psdf matrix for the ground displacements at the support d.o.f.s and can be obtained in terms of the psdf of ground acceleration $S_{\ddot{f}_g\ddot{f}_g}$ with the help of both the coherence function, the ratio between the three components of ground motion and Eqns.(11, 16 & 17).

5.3.3 Evaluation of the total displacement

The total displacement at any point of the r[th] beam segment of the bridge deck at any time (t) can be written as

$$Y(x_r,t) = y(x_r,t) + g(x_r,t) \tag{51}$$

The psdf of the total vertical displacement can be expressed as

$$S_{YY}(x_r,\omega) = S_{yy}(x_r,\omega) + S_{gg}(x_r,\omega) + S_{yg}(x_r,\omega) + S_{gy}(x_r,\omega) \tag{52}$$

$S_{yg}(x_r,\omega)$ and $S_{gy}(x_r,\omega)$ are the cross power spectral density functions between relative and the quasi-static displacements. Using Eqns.(--------), the expression for $S_{yg}(x_r,\omega)$ can be obtained as

$$S_{yg}(x_r,\omega) = \underset{(1\times M)}{[\phi(x_r)]} \underset{M\times M}{diag[Hn(\omega)]} \underset{M\times 8}{[G]} \underset{8\times 8}{[S_{\bar{f}\bar{f}}]} \{g_{jr}(x_r)\}_{8\times 1} \tag{53}$$

n=1, 2, ----, M; j = 1,2, -----,8; r =1, 2, ------,N_b

where $[S_{\bar{f}\bar{f}}]$ is the cross power spectral density matrix of the random vectors $\{\ddot{f}\}$ and $\{\bar{f}\}$ i.e., the support accelerations and displacements. $S_{gy}(x_r,\omega)$ is the complex conjugate of $S_{yg}(x_r,\omega)$.

5.3.4 Evaluation of the bending moment

Using Eqns.(45 & 47) and differentiating the expression for Y(x_r , t) twice with respect to x, the following expression for the bending moment can be obtained as

$$E_d l_r \frac{\partial^2 y}{\partial x^2} = \sum_{n=1}^{M} E_d l_r \frac{d^2\phi(x_r)}{dx^2} q_n(t) + \sum_{j=1}^{8} E_d l_r \frac{d^2 g_{jr}(x_r)}{dx^2} f_j(t) \tag{54}$$

Similar expressions can be obtained for the psdf of the bending moment at any point in the rth beam segment of the bridge deck as those derived for the total displacement by replacing $\phi(x_r)$ and $g_{jr}(x_r)$ by $E_d I_r d^2\phi(x_r)/dx^2$ and $E_d I_r d^2 g_{jr}(x_r)/dx^2$ respectively. $E_d I_r d^2 g_{jr}(x_r)/dx^2$ is obtained from the quasi-static analysis of the entire bridge using the stiffness approach as mentioned before.

5.4 Statistical parameters of response

For studying the statistical properties of the response process, the first few moments of the response power spectral density function are needed. The j^{th} moment of the PSDF function may be defined as follows:

$$\lambda_j = \int_0^{\alpha} w^j S_{YY}(w)\,dw \quad \text{where } j = 0, 1, 2,---- \tag{55}$$

The zeroth and second moments may be recognized as the variances of the response and the first time derivative of the response respectively.

$$\lambda_0 = \int_0^{\alpha} S_{YY}(w)\,dw = \sigma_{YY}^{\;2} \tag{56}$$

$$\lambda_2 = \int_0^{\alpha} w^2 S_{YY}(w)\,dw = \sigma_{\dot{Y}\dot{Y}}^{\;2} \tag{57}$$

The mean rate of zero crossing at positive slopes is by

$$\nu_0 = \frac{1}{2\pi}\sqrt{\frac{\lambda_2}{\lambda_0}} \tag{58}$$

Another quantity of interest is the dispersion parameter q given by

$$q = \sqrt{1 - \frac{\lambda^2{}_1}{\lambda_0 \lambda_2}} \tag{59}$$

The value of q lies in the range [0,1]. It can shown that q is small for a narrow band process and relatively large for a wide band process. The mean rate of crossing a specified level A at a positive slope by a stationary zero mean Gaussian random process z(t) can be expressed by Lin (1967) as

$$\nu_a = \nu_0 e^{\frac{-\psi^2}{2}} \tag{60}$$

where

$$\psi = \frac{A}{\sigma_{YY}}$$

It has been confirmed by theoretical as well as simulation studies that the probability of a stationary response process remaining below a specified barrier level decays approximately exponentially with time as given by the relationship (Coleman,1959 and Crandal et.al., 1966)

$$L(T) = L_o e^{-\alpha T} \tag{61}$$

Where, L_0 is the probability of starting below the threshold, α is the decay rate, and T is the duration of the response process.

At high barrier levels, L_o is practically equal to one, and the decay rate is given by the following expressions for processes with double barrier and one sided barrier respectively (Vanmarcke, 1975; Lin, 1967)

$$\alpha_D = 2\nu_a \tag{62}$$

$$\alpha_S = \nu_a \tag{63}$$

In case of relatively low threshold levels, an improved value can be obtained by using the expressions for the probability of starting below the threshold and the decay rate (Vanmarcke, 1975)

$$L_o = 1 - e^{\frac{-\psi^2}{2}} \tag{64}$$

$$\alpha_D{}^* = \alpha_D \frac{1 - e^{-\sqrt{\frac{\pi}{2}}(q\psi)}}{1\text{-}e^{-\psi^2/2}} \tag{65}$$

$$\alpha_S{}^* = \alpha_S \frac{1 - e^{-\sqrt{2\pi}\ (q\psi)}}{1\text{-e}^{-\psi^2/2}} \tag{66}$$

5.5 Reliability estimation against first passage failure

For an earthquake with given magnitude M, the probability of First Passage Failure, i.e., the probability that the response is larger than a threshold level A, can be determined from the following relationship

$$p\left[z > A \middle| M_r\right] = 1 - L(T) \tag{67}$$

where T is the duration of the response.
If f_M (M) is the probability density functions of earthquake magnitude, the probability of First Passage Failure, provided that an earthquake occurs, can be calculated from (Ang and Tang, 1975)

$$p_E = p\left[z > A\right] = \int_{M=4}^{9} p\left[z > A \middle| M_r\right] f_M(M)\, dM \tag{68}$$

If the rate of earthquake occurrence for the seismo – tectonic region considered in the study is a constant and n is the average number of earthquakes per year in the magnitude range of interest for the source region, the probability of atleast one failure due to earthquake in "m" years can be expressed as

$$P_F = 1 - \left(1 - p_E\right)^{m.n} \tag{69}$$

6. Numerical study

A double plane symmetrical harp type cable stayed bridges, (Morris, 1974) used as illustrative example is shown in Fig.3.3(a). The structural data of the bridge is shown in Table-1.
In addition, the following data are assumed for the analysis of the problem, $E_c = E_d$; $\xi = 0.02$ for all modes; and the tower – deck inertia ratio, the ratio between three components of the ground motion (R_u:R_v:R_w), α, duration of earthquake, β values for exponential distribution of magnitude of earthquake are taken 4, (1.0:1.0:1.0) , 0.0^0, 15 sec and (1.5, 2.303, 2.703) respectively unless mentioned otherwise. Also, the ground motion is assumed to be partially correlated in firm soil unless mentioned otherwise.
The random ground motion is assumed to be homogeneous stochastic process which is represented by Clough and Penzien double filter psdf given by two sets of filter coefficients representing the soft and firm soils respectively. For the soft soil , the coefficients are ω_g = 6.2832 rad/sec; ω_f = 0.62832 rad/sec ; $\xi_g = \xi_f = 0.4$, while those for the firm soils are ω_g = 15.708 rad/sec; ω_f = 1.5708 rad/sec; $\xi_g = \xi_f = 0.6$. The two psdfs corresponding to the two sets of filter coefficients are shown in Fig.5. The spatial correlation function used in the parametric study is given by Eqn.5 in which the value of c =2.0, Vs = 70 m/sec and Vs = 330 m/sec for the first and second psdfs respectively. The r.m.s (root mean square) ground acceleration is related to intensity of earthquake by empirical equation given by Eqn.7. Intensity of earthquake I_s in turn is related to magnitude of earthquake given by Eqn.9. The

input for excitation is thus the intensity of earthquake for which $\sigma_{\ddot{u}_g}$ value can be calculated using Eqn.7. From $\sigma_{\ddot{u}_g}$ the value of S_0 defining the ordinates of the double filter psdf can be obtained using equation Eqn.1.

Parameter	Centre Span	Side Span
Deck length Deck Area Deck Depth Modulus of Elasticity of deck Moment of Inertia of deck	L_2 = 335.28 m A_2 =0.32 m^2 D_2 = 4.0m E_2 = 2.0683 x 10^{11} N/m^2 I_2 = 1.131 m^4	L_1 == L_3 = 137.16 m A_1 = A_3 =0.32 m^2 D_1 = D_3 = 4.0 E_1 = E_3 = 2.0683 x 10^{11} N/m^2 I_1 = I_3 = 1.131 m^4
Tower Properties	L_t = 85.96m ; E_t = 2.0683 x 10^{11} N/m^2 ; A_t = 0.236 m^2	
Cable Properties	Area of the cables (1 to 6)= 0.04, 0.016, 0.016, 0.016, 0.016 and 0.04 m^2 Tension in cables (1 to 6)= 15.5 x 10^6 , 5.9 x 10^6, 5.9 x 10^6, 4.3 x 10^6, 5.9 x 10^6 and 15.5 x10^6 N/ m^2 Modulus of cables E_c = 2.0683 x 10^{11} N/m^2	
Distributed mass of the bridge over half width deck		9.016 x 10^3 Kg/m
Properties of flexible foundation	Radius of circular foundation = 3m Poission's ratio = 0.33 Density of the soil = 12Kn /m^3	

Table 1. Structural data of the Harp Type Cable Stayed Bridge

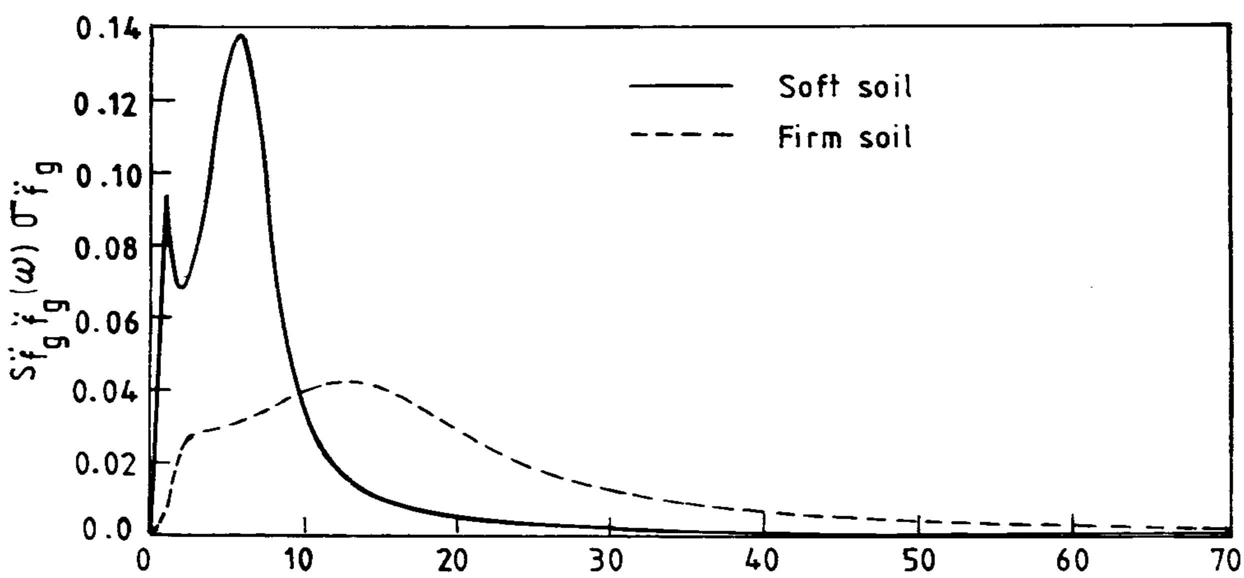


Fig. 5. Normalized PSDF of ground acceleration

Fig.6 shows the first five modes of the bridge corresponding to I_t/ I_d = 4.0. The first five frequencies and the corresponding nature of the mode shapes for the bridge is given in Table-2 for different I_t/ I_d ratios.

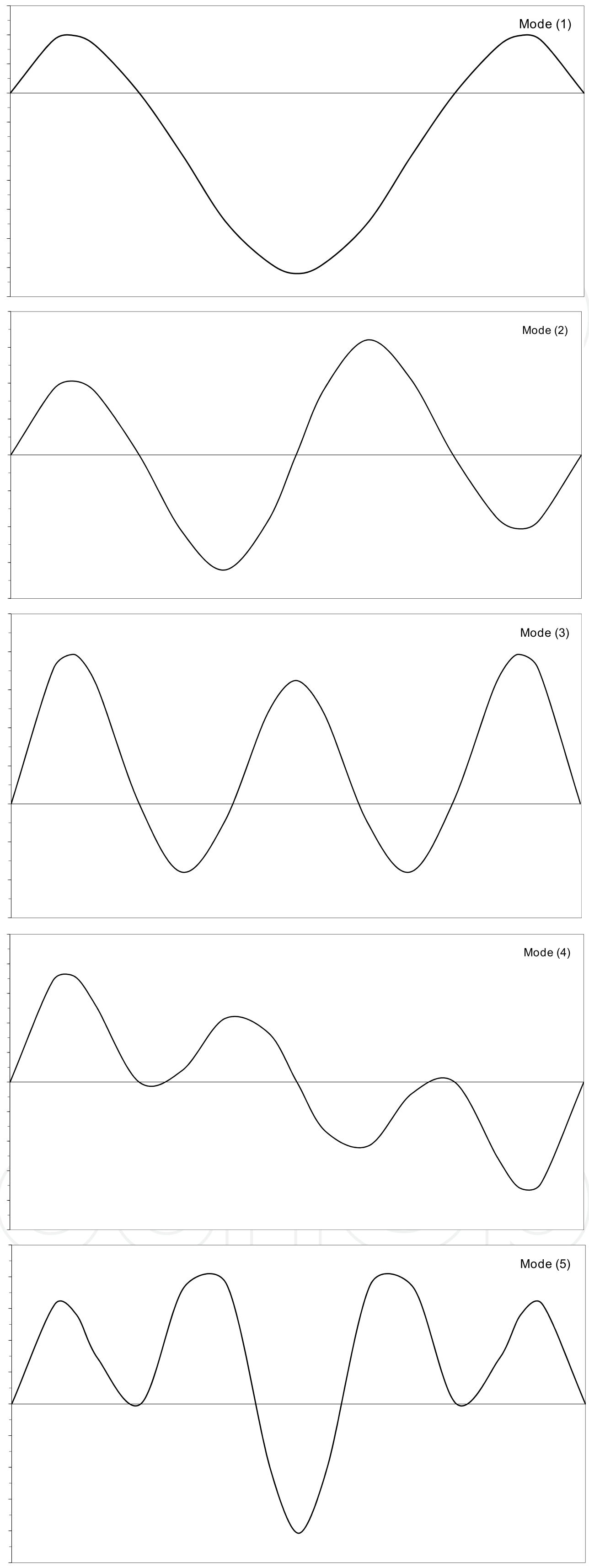


Fig. 6. First 5 mode shapes

Mode No.	Fundamental Frequencies (rad/sec)			Nature
	$I_t / I_d = 2.0$	$I_t / I_d = 4.0$	$I_t / I_d = 6.0$	
1	2.838	2.983	3.072	Symmetric
2	3.247	3.551	3.716	Anti-symmetric
3	4.443	4.706	4.885	Symmetric
4	5.319	5.374	5.409	Anti-symmetric
5	6.429	6.450	6.467	Symmetric

Table 2. Fundamental frequencies of the bridge deck for different tower – deck inertia ratio.

6.1 Effect of the tower – Deck inertia ratio (I_t/I_d)

The first 5fundamental frequencies are obtained for different ratios of the tower – deck inertias i.e. 2, 4 and 6 respectively as shown in Tables-2. It is seen from the tables that with the increase of the tower – deck inertia ratio, the frequencies of the bridge deck increase. This is due to the fact that the increase in the tower stiffness increases the component of the vertical stiffness of the bridge provided by the cables.

6.2 Effect of the quasi-static component on the response

Total and relative displacement of the bridge decks obtained for the firm soil is shown in Tables-3. It is seen from the Tables that the contribution of the quasi-static component to the total response is significant for displacement where as it is small for bending moment.

Point No.	Relative		Total	
	Displacement (m)	Moment (t-m)	Displacement (m)	Moment (t-m)
1	0.0	0.0	0.0509	0.0
2	0.0961	1424.0	0.1108	1429.0
3	0.1040	1432.0	0.1184	1435.0
4	0.0831	727.0	0.0995	740.0
5	0.0	996.0	0.0509	1009
6	0.0906	715.0	0.1099	726
7	0.1621	974.0	0.1808	976.0
8	0.1886	702.0	0.2109	699
9	0.1957	1160.0	0.2164	1160.0

Table 3. Effect of the quasi – static part of the response on the r.m.s responses

6.3 Effect of barrier level on the reliability

Effect of the barrier level on the reliability of the bridge is shown in Figs. 7. The barrier level are taken as 15%, 20%, 25%, 30%, 33%, 40%, 50% and 70% of the yield stress assuming that the barrier level is the difference between yield stress and the pre-stress in the girder (deck). It is seen from the figures that the reliability increases as the barrier levels increases as it would be expected. However, the variation is not linear; it tends to follow an S shaped curve. For certain condition, the variation of reliability with barrier level may be very steep in the lower range of barrier levels. The same figures also compare between the reliabilities for firm and soft soils conditions. It is seen from the Fig.7 that the reliability for a particular barrier level is higher for firm soil. The difference between the two is considerably more at the lower end of the barrier level.

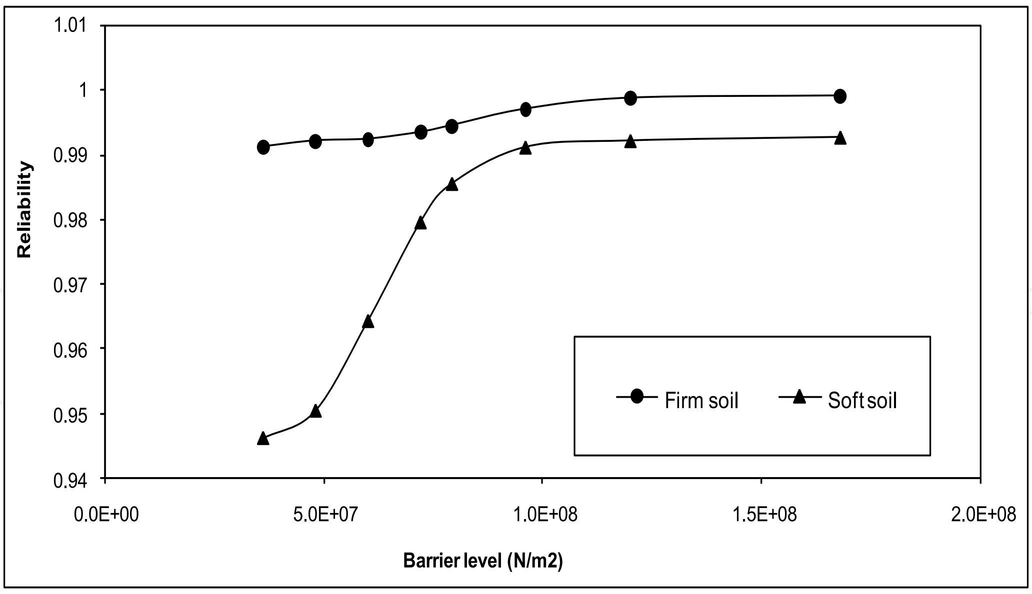


Fig. 7. Variation of Reliability with Barrier level

6.4 Effect of magnitude of earthquake on the reliability

The effect of the distribution of magnitude of earthquake on the reliability is shown in Fig.8. The figure shows the variation of reliability with barrier level for different distributions of the magnitude of earthquake obtained by exponential and gumbel distribution. It is seen from the figures that the reliability increases with the increase in beta values for exponential distribution. The difference between reliabilities obtained for two beta values considerably is more for lower values of barrier level. Above a certain value of beta, the reliability nearly approaches unity for all barrier levels. Further, Gumbel distribution provides much higher value of reliability as compared to Exponential distribution (for beta = 1.5). For soft soil condition (Fig.9), the effect of the distribution of the magnitude of earthquake is more produced.

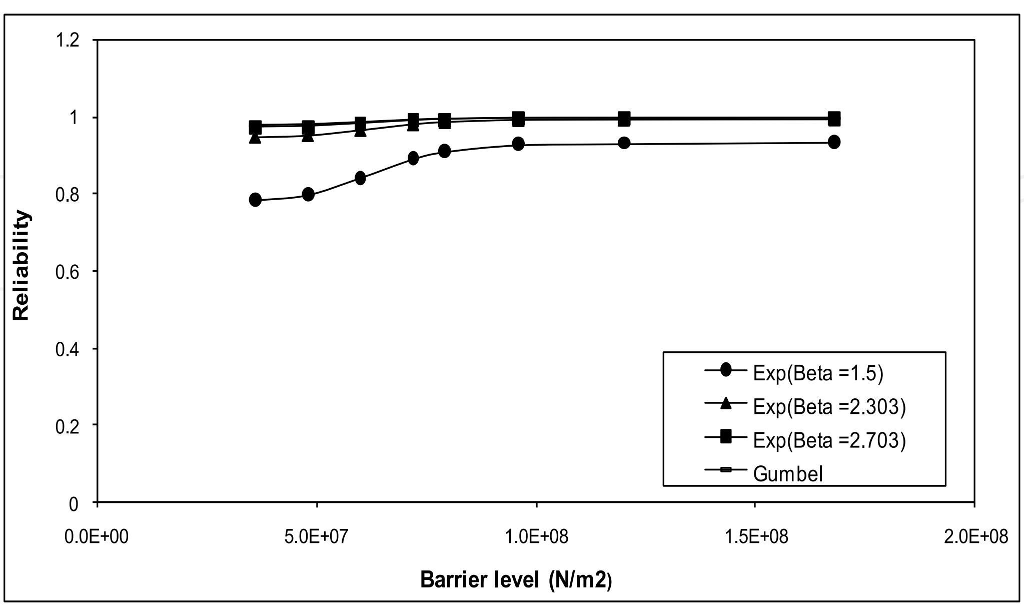


Fig. 8. Variation of Reliability with Barrier level for different distributions of magnitude of earthquake (Hard soil)

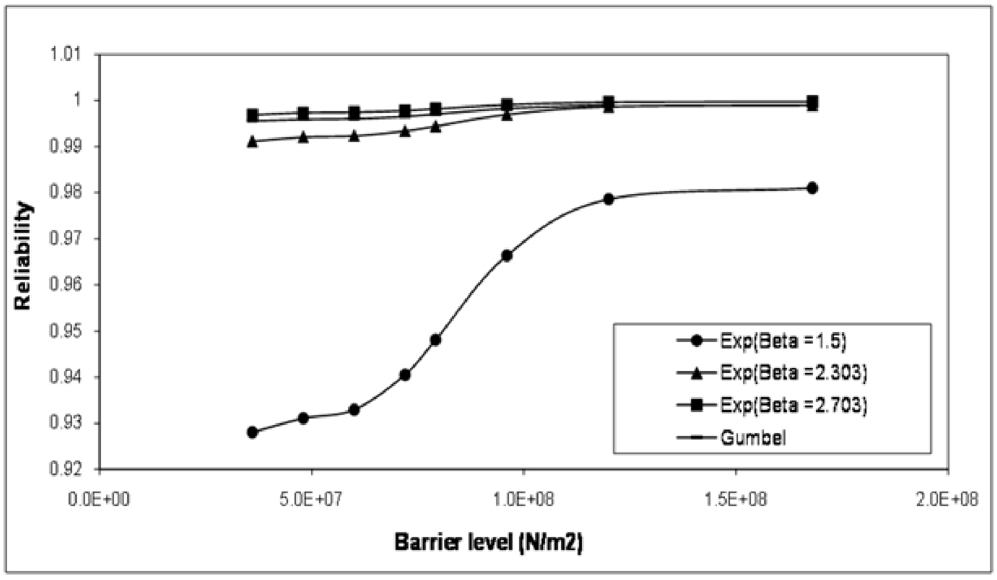


Fig. 9. Variation of Reliability with Barrier level for different distributions of magnitude of earthquake (Soft soil)

6.5 Effect of ratio between the three components of earthquake (Ru:Rv:Rw) on the reliability

The effect of this ratio on the variation of reliability with the barrier level is shown in Figs.10 and 11 for two angle of incidences of earthquake (α = 0^0 and α = 45^0). It is seen from the figures that the ratio has significant effect on this variation, especially at the lower end of the barrier level.

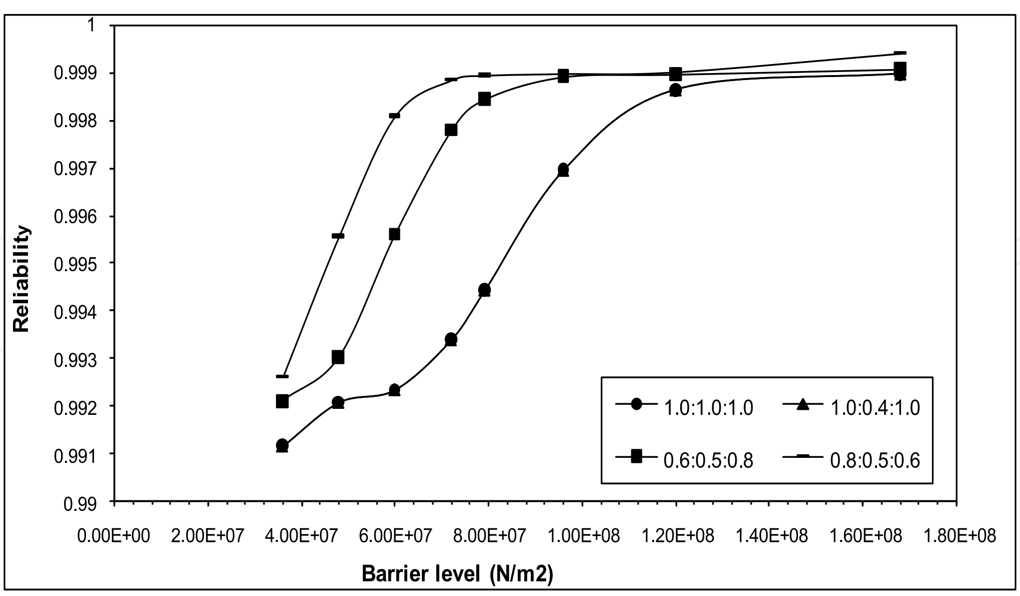


Fig. 10. Variation of Reliability for different ratios of Earthquake components (Alpha =0.0 degree)

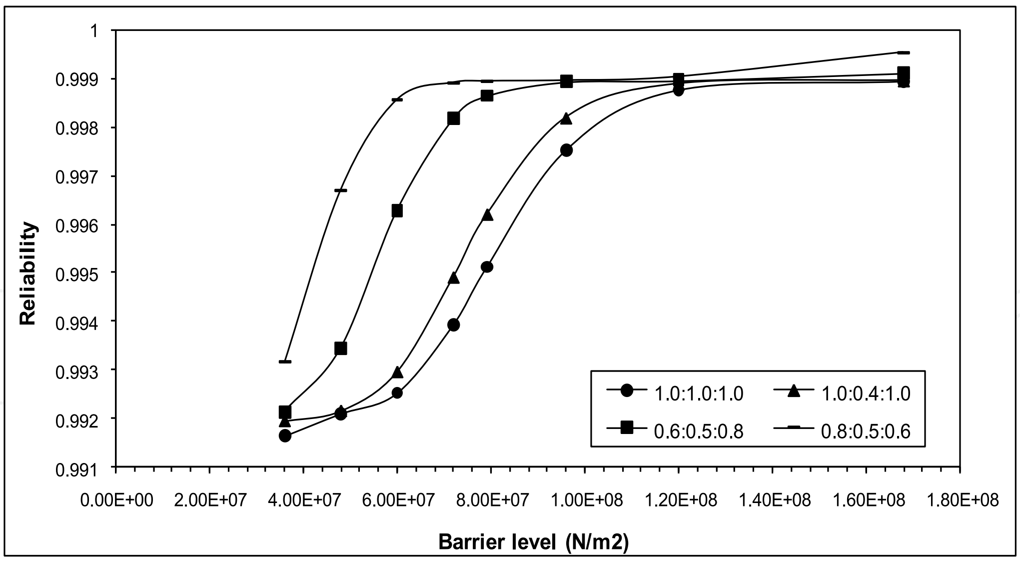


Fig. 11. Variation of Reliability for different ratios of Earthquake components (Alpha =45 degree)

6.6 Effect of angle of incidence on the reliability

Fig.12 show the effect of angle of incidence on the variation of reliability with barrier level. Three values of angle of incidence are considered namely, 0^0 i.e. major direction of earthquake is along the longitudinal axis of the bridge and the other two cases are having 30^0 and 70^0 angle of incidence with the longitudinal axis of the bridge. It is seen from the figures that 0^0 angle of incidence provides minimum reliability while 70^0 angle of incidence provides maximum reliability. This is expected because 0^0 angle of incidence produces maximum stress in the bridge.

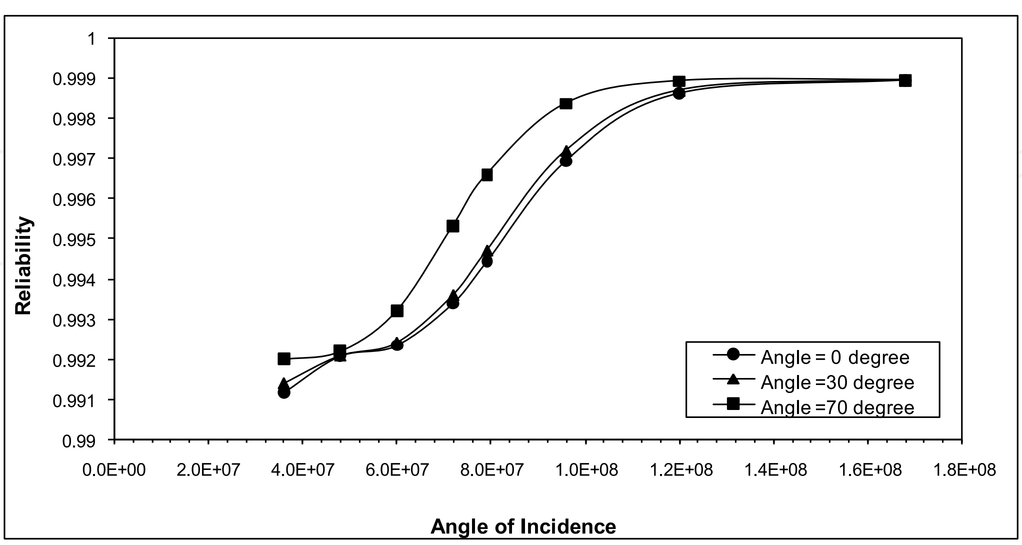


Fig. 12. Variation of Reliability for different Angles of Incidence

6.7 Effect of correlation of ground motion on the reliability

Fig.13 shows the variation of reliability with barrier level for three cases of ground motion that is fully correlated, partially correlated and uncorrelated. It is seen from the figures that fully correlated ground motion provides the highest reliability while uncorrelated ground motion gives the lowest value. The partially correlated ground motion gives reliability in between the two. This is the case because uncorrelated / partially correlated ground motion induces additional bending moment in the deck due to the phase lag of ground motion between different supports. The difference between the reliabilities for the three cases is nor very significant for the hard soil. However, the difference between them is significant for the soft soil (Fig.14). Further, the difference between the reliabilities are considerably reduced at the higher end of the barrier level.

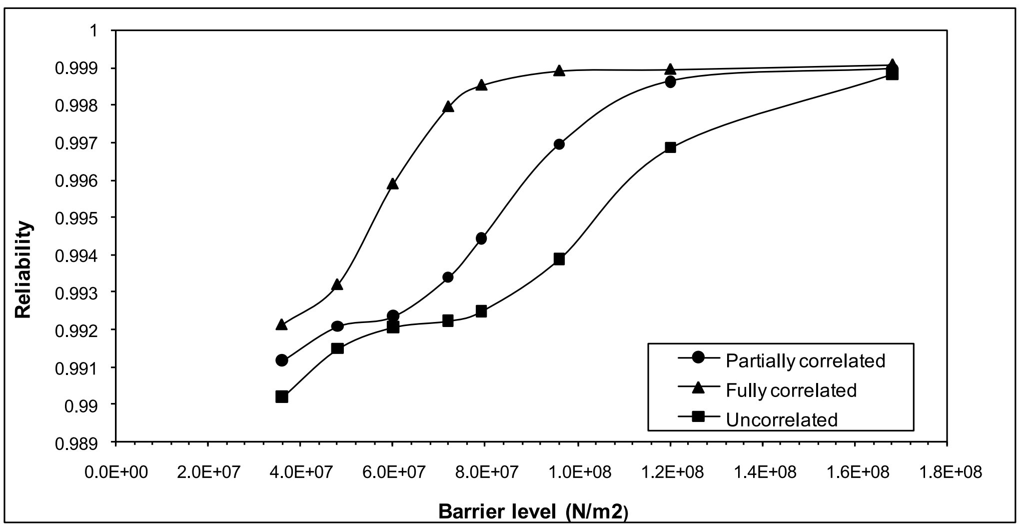


Fig. 13. Variation of Reliability with Barrier level for different degrees of correlation of ground motion (Hard soil)

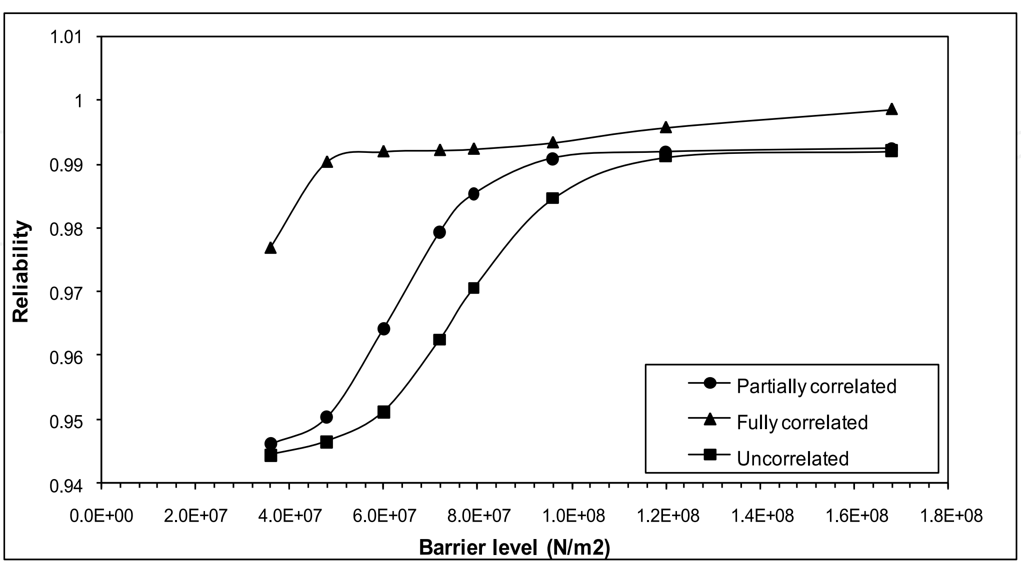


Fig. 14. Variation of Reliability with Barrier level for different degrees of correlation of ground motion (Soft soil)

6.8 Effect of duration of earthquake on the reliability

Fig.15 shows the variation of reliability with duration of earthquake for a barrier level of 33% . It is seen from the figures that reliability decreases mildly with the increase of duration of earthquake for both soft and firm soil. Thus, the duration of earthquake does not have significant influence on the reliability.

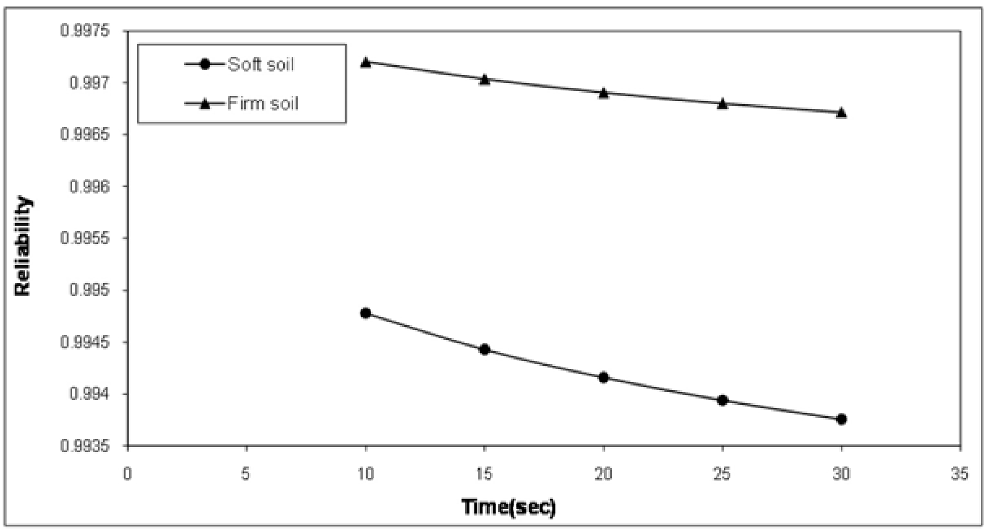


Fig. 15. Variation of Reliability with duration of Earthquakes (for a barrier level of 33%)

7. Conclusions

Reliability against first passage failure of cable stayed bridges under earthquake excitation is presented. The responses of cable stayed bridges are obtained for random ground motion which is modeled as stationary random process represented by double filter power spectral density function (psdf) and a correlation function. The responses are obtained by frequency domain spectral analysis. Conditional probability of first crossing the threshold level for a given RMS ground acceleration is obtained by using the moments of the psdf of the response. The RMS value of the ground acceleration is related to the magnitude of earthquake by an empirical equation, and the probability density function of the earthquake is integrated with the conditional probability of failure to find the probability of first passage failure. Using the above method of analysis, two cable stayed bridges are analyzed and probabilities of first passage failure are obtained for a number of parametric variations. The results of the numerical study lead to the following conclusions:

i. The reliability against first passage failure increases sharply with the increase in barrier level in the lower range of its values.
ii. For the soft soil condition, the reliability is considerably less as compared to the firm soil condition.
iii. Gumbel distribution of the magnitude of the earthquake provides a very high estimate of reliability and gives values close to those obtained by exponential distribution with high values of the parameter β.
iv. Uncorrelated ground motion provides lower estimates of the reliability as compared to the fully correlated ground motion. The difference significantly more for the soft soil.

v. The ratios between the components of ground motion have considerable influence on the reliability estimates. For soft soil condition, the difference between the reliability estimates, especially in the lower range of barrier level.

vi. In the lower range of barrier level, considerable difference between reliabilities is observed for 0^0 and 70^0 angle of incidence with respect to the longitudinal axis of the bridge. For higher value of the barrier level, there is practically no difference between the two reliabilities.

vii. The duration of ground motion does not have significant influence on the reliability estimates.

viii. It is found that reliability decreases with the increase of average number of earthquake per year. The variation is nonlinear and more steep for the soft soil condition. For an average number of earthquake 0.5 per year , the reliability could be as 0.3.

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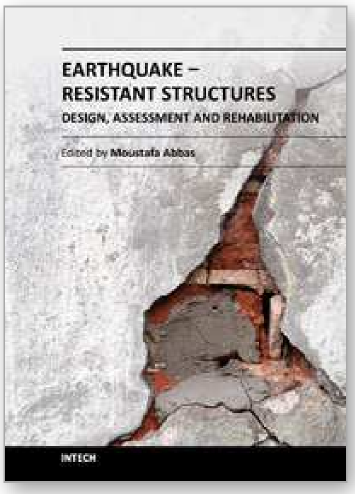

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This book deals with earthquake-resistant structures, such as, buildings, bridges and liquid storage tanks. It contains twenty chapters covering several interesting research topics written by researchers and experts in the field of earthquake engineering. The book covers seismic-resistance design of masonry and reinforced concrete structures to be constructed as well as safety assessment, strengthening and rehabilitation of existing structures against earthquake loads. It also includes three chapters on electromagnetic sensing techniques for health assessment of structures, post earthquake assessment of steel buildings in fire environment and response of underground pipes to blast loads. The book provides the state-of-the-art on recent progress in earthquake-resistant structures. It should be useful to graduate students, researchers and practicing structural engineers.

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