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Extradosed Bridges – Assessment of seismic damage using Ground Acceleration and Displacement correlations

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Abstract:- Forced vibration of structure for given Earthquake time history is governed by peak acceleration. For cable stayed structures such as Extradosed cable stayed bridge it is difficult to predict dynamic response using usual methods of dynamic analysis as applied to some other bridge structures like response spectrum analysis, accurate analysis like time history analysis is time consuming and has time and cost effects. Nonlinearities can only be considered in time history analysis. The proposed method correlates the peak ground acceleration (PGA) and earthquake deformation ratio (EDR) which can be used for simplified dynamic analysis and can prove handy tool for structural engineers to know earthquake related serviceability without much complicated analysis at initial stages. This ratio can be used to present seismic damage indices. The method is proposed considers Extradosed bridge for example.

Keywords:- Extradosed cable stayed bridge; Structural Behavior; Earthquake; Dynamic response; PGA-EDR; Seismic Damage index.

I. INTRODUCTION

Dynamic response prediction has been the matter of research for many authors, in particular as the structural design of many structures is governed by the earthquake load cases or combinations thereof. The commonly used simplified methods used for analysis are based of theory of dynamics pertaining to SDOF systems. Rules of modal combinations viz SRSS, CQC are used for MDOF systems. These combinations rules are fairly accurate and helpful since exact methods such as time history analysis involves significant skill, time and cost. Although it is acceptable that having temporal response parameters proves helpful, the structural design is always governed by peak response. The peak values correspond to peak ground motions. The approach as presented here gives fairly accurate estimation of earthquake response and may prove helpful for quick response prediction at the time of preliminary designs and further serves as a tool for seismic damage index for higher intensity ground motions for which the structure is not designed

The recent research has shown that a Extradosed bridge, variant of cable stayed bride where cables add substantial prestress to the deck because of the shallow pylon, are found to be economical for spans upto 250m. The intrados is defined as the interior curve of an arch, or in the case of cantilever-constructed girder bridge, the soffit of the girder. Similarly, the extrados is defined as the uppermost surface of the arch. The term 'Extradosed' was coined by Jacques Mathivat in 1988 [9] to appropriately describe an innovative cabling concept he developed for the Arrêt-Darré Viaduct , in which external tendons were placed above the deck instead of within the cross-section as would be the case in a girder bridge. To differentiate these shallow external tendons, which define the uppermost surface of the bridge, from the stay cables found in a cable-stayed bridge, Mathivat called them 'Extradosed' prestressing.

Some features of Extradosed bridge as given below;

- External appearance resembles cable-stayed bridge – but structural characteristics are comparable to those of conventional girder bridge
- The Girder Depth are lesser than that of conventional girder bridges
- The stay cables (prestressing tendons outside the girder) need no tension adjustment necessary for cable-stayed bridges, and can be treated as usual tendons as in girder bridges
- The height of pylon is half as that of cable-stayed bridge and hence easier to construct
- With small stress fluctuation under live load the anchorage method for stay cables can be same as that of tendons inside girder and thereby achieve economy

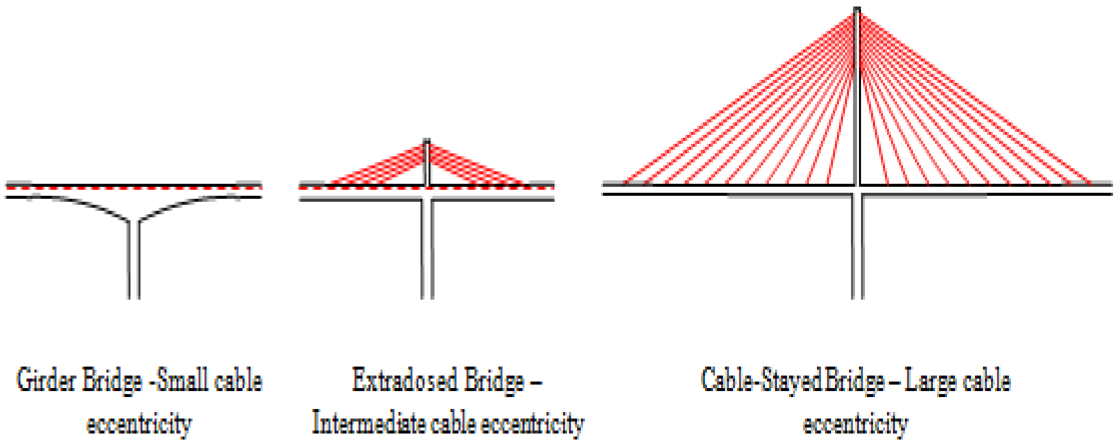

Fig. 1 Figure showing Cable arrangements in Girder Bridge, Extradosed Bridge and Cable stayed bridge

With the rapid increase in span length, combined trend and also trend of using high strength materials have resulted in slender structures and a concern is being raised over dynamic behavior of such structures, in case of cable supported structures it is more pronounced as this further includes vibrations of cable elements also. An accurate analysis of natural frequencies is fundamental to the solution of its dynamic responses due to seismic and wind and traffic loads.

Seismic analysis of structure is important aspect of structural design as overall economy and safety of structure is most of the time governed by the load cases involving seismic forces. Many studies associated with this have been performed over last few decades. However, number of researches on dynamic behavior of Extradosed Bridge is very limited.

To gauge the exact dynamic response of structure, time history analysis is used. But this method is found to be time consuming and involves considerable cost. Further it is sometimes required to know the approximate results beforehand prior to proceeding to extensive analysis such as time history analysis.

The Proposed "Earthquake- Displacement Ratio" (EDR) is defined as ratio of maximum seismic displacement to maximum static displacement. These both displacements measured at the same point. Earthquake-displacement ratio can be evaluated as seismic damage index.

The aim of this paper is to present work relating to dynamic behavior of Extradosed bridge and the proposed PGA – EDR relationship.

II. FORMULATION OF PROBLEM

STIFFNESS AND MASS MATRIX FORMULATION FOR EXTRADOSED BRIDGE

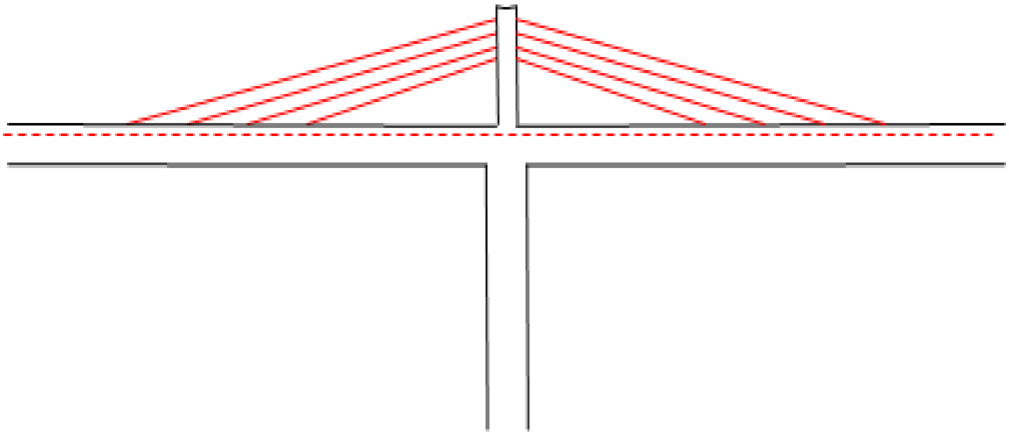

Fig.2 Elevation – Typical Extradosed bridge

Consider a typical Extradosed bridge as shown in figure 2, let us take a small section as shown in the figure 3 below. The boundary conditions for this element can be considered as that of beam on elastic foundation to relate effect of elastic support provided by cable. Further this beam will be subjected to prestressing force due to horizontal component of cable forces, as shown in figure 3

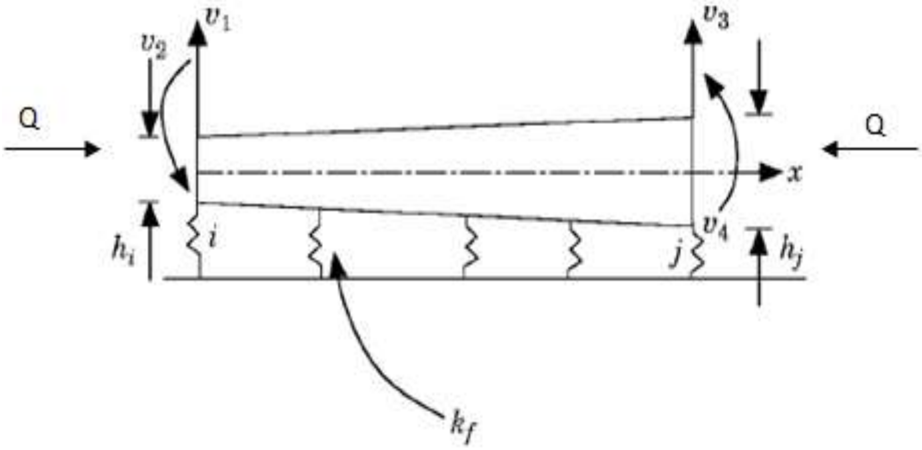

Fig.3 Beam on elastic foundation

Now, consider an element *ij* of length *L* of a beam on an elastic foundation as shown in Figure.3 having a uniform width *b* and a linearly varying thickness *h(x)* It will be a simple matter to consider an element having a linearly varying width if the need arises. Neglecting axial deformations this beam on an elastic foundation element has two degrees of freedom per node a lateral translation and a rotation about an axis normal to the plane of the paper and thus possesses a total of four degrees of freedom. The (4x4) stiffness matrix *k* of the element is obtained by adding the (4x4) stiffness matrices *kB*, *kF* and *kQ* pertaining to the usual beam bending stiffness and foundation stiffness and stiffness due to prestressing force (Q) respectively

Since, there are four end displacements or degrees of freedom a cubic variation in displacement is assumed in the form

$$v = Aa \qquad \text{Eq. (1)}$$

Where, $A = (1\ x\ x^2\ x^3)$ and $a^T = (a_1\ a_2\ a_3\ a_4)$ (Displacement variation within element)

The four degrees of freedom corresponding to the displacements v_1, v_3 and the rotations v_2 v_4 at the longitudinal nodes are given by

$q=Ca$ (Nodal displacements) Eq. (2)

Where $q^T = (v_0\ v_1\ v_2\ v_3)$ and C is the connectivity matrix for an element *ij* between $x=0$ and $x=L$ *as given in* Figure 3

From equations (Eq.1) and (Eq.2)

$V=AC^{-1}q$ Eq. (3)

If E is the Young|s modulus and $I=bh(x)^3/12$ is the second moment of area of the beam Cross-section about an axis normal to the plane of the paper\ the bending moment M in the element is given by

$$M = D\frac{\partial^2 v}{\partial x^2} = DBC^{-1}q \qquad \text{Eq. (4)}$$

Where $D=EI(x)$ and $B=d^2A/dx^2=(0, 0, 2, 6x)$

A. Stiffness due to bending

The potential energy U_B due to bending is

$$U_B = \frac{1}{2}\int_0^l \frac{d^2v}{dx^2} M dx \qquad \text{Eq. (5)}$$

And the stiffness is given by

$$kb = \frac{\partial^2 U_B}{\partial d^2} \qquad \text{Eq. (6)}$$

From equations (Eq.5) and (Eq.6) we get,

$$k\bar{b} = \int_0^l B^T DBdx \text{ (Elemental)} \qquad \text{Eq. (7)}$$

$$Kb = (C^{-1})^T k\bar{b} C^{-1} \text{ (Assmebled)} \qquad \text{Eq. (8)}$$

B. Stiffness due to elastic foundation

The potential energy of foundation stiffness is given by,

$$U_f = \frac{1}{2}\int_0^l V^T kfVdx \qquad \text{Eq. (9)}$$

and then the stiffness is given by,

$$kf = \frac{\partial^2 U_f}{\partial d^2} \qquad \text{Eq. (10)}$$

From equations (Eq.9) in (Eq.10) we get,

$$\bar{k}f = \int_0^l A^T kfAdx \text{ (Elemental)} \qquad \text{Eq. (11)}$$

$$Kf = (C^{-1})^T k\bar{f}C^{-1} \text{ (Assembled)} \qquad \text{Eq. (12)}$$

C. Stiffness due to Prestressing force

The potential energy of prestressing force is given by,

$$U_Q = \frac{1}{2}\int_0^l Q\left(\frac{\partial v}{\partial x}\right)^2 dx \qquad \text{Eq. (13)}$$

Then the stiffness is given by,

$$k_Q = \frac{\partial^2 U_Q}{\partial d^2} \qquad \text{Eq. (14)}$$

Substituting equation (13) in (14) we get,

$$\bar{k}_Q = \int_0^l A^T k_Q Adx \qquad \text{Eq. (15)}$$

$$K_Q = (C^{-1})^T k_Q C^{-1} \qquad \text{Eq. (16)}$$

Finally complete stiffness is given by,

$$K = K_B + K_f + K_Q \qquad \text{Eq. (17)}$$

Element mass matrix is the equivalent nodal mass that dynamically represents the actual distributed mass of the element. This is kinetic energy of the element.

$$T = \frac{1}{2}\int_0^l (\dot{v})^T \rho dV\dot{v} \qquad \text{Eq. (18)}$$

Where, $\dot{v}$ = Lateral velocity and ρ = mass density

$$T = \frac{\rho}{2}(\dot{q})^T (C^{-1})^T \left\{\int_0^l A^T hxAdx\right\}(C)^{-1} q \qquad \text{Eq. (19)}$$

Then, the mass matrix is given by,

$$m = (C^{-1})^T \overline{m} C^{-1} \qquad \text{Eq. (20)}$$

and $\overline{m} = \rho\int_0^l A^T hxAdx$ Eq. (21)

for free vibration of this beam,

$$[M]\{\ddot{q}\} + [C]\{\dot{q}\} + [K]\{q\} = 0 \qquad \text{Eq. (22)}$$

and for forced vibration,

$$[M]\{\ddot{q}\} + [C]\{\dot{q}\} + [K]\{q\} = \{f\} = [N]^{-1} f_0 \qquad \text{Eq. (23)}$$

For Extradosed bridge, since the cable are shallower and the effect of prestressing force is more the effecting of prestress shall be taken in to account as shown in the equation above.

III. ANALITICAL STUDY – FORCED VIBRATION BEHAVIOUR

To study static as well as dynamic behavior of Extradosed Bridge, 3 numbers of models with variable parameters are prepared. Basic span configuration as applicable for Extradosed span is selected to be 120, 200 and 260m main span, the side span is about 0.45 of main span. The pylon height is varied from 8 to 12 to account for the effect of varying cable inclinations. The cable inclination varies from 17 to 30 degrees. The requirement of cable area and prestressing is as per preliminary design. Box beam superstructure is adopted with solid rectangular pylon designed by working stress method. For details of model refer table-1

Since, Extradosed bridges take part in an intermediate zone between prestressed bridges and cablestayed bridges, their structural behavior may be similar to these kinds of typologies, depending on design criteria adopted during the project stage. Generally a rigid deck Extradosed bridge shall have a similar behavior to the prestressed bridges, thus avoiding high stress oscillations of stay cables and, consequently, avoiding fatigue conditions associated with anchorages and tendons present in a slender deck extradosed bridge, which behavior is quite close to the cable-stayed bridge. Its construction demands the acquaintance of technologies currently applied on straight course-prestressed concrete bridges and cable-stayed bridges, which is generally developed by means of the consecutive cantilever method but counting with the assistance of tension rods that are not placed on temporary, but on permanent basis.

In order to analytically study structural behavior of Extradosed bridges with respect to ground acceleration numerical studies were conducted for forced vibration on typical Extradosed bridge. The parameters were non-dimentionalised using equivalent factors. The earthquake time histories used are having Koyna (PGA-0.49g), Bhuj (PGA-0.3g) and El-Centro (PGA-1.61g).

Table-1 Details of Earthquake time histories used for this study;

Sr No	Span (L) configuration	Pylon ht (H)	Remarks
1	48+100+48	10	On pile foundation, 12.5 wide deck
2	90+200+90	20	On pile foundation, 14 wide deck
3	110+260+110	26	On pile foundation, 18 wide deck

Non-Dimensionalizing of parameters

Forced vibration analysis for three earthquake time histories having different characteristics are undertaken. To compare the results all parameter have been non dimensionalised using equivalent factors as mentioned below;

- $V = \rho * g * A * L$
- $M = \rho * g * A * L^2$

Where, V & M are non dimensioning factors for shear force, bending moment. Where, ρ = Mass Density, g= Gravitational acceleration, A= Cross section area of component and L= Half span length of the component.
Span and pylon height are non-dimensionlalized by using parametric length. The results obtained from the time history analysis in terms of bending moment and shear forced in the structure are non-dimensionalised and superimposed and presented in Fig 4 to 8

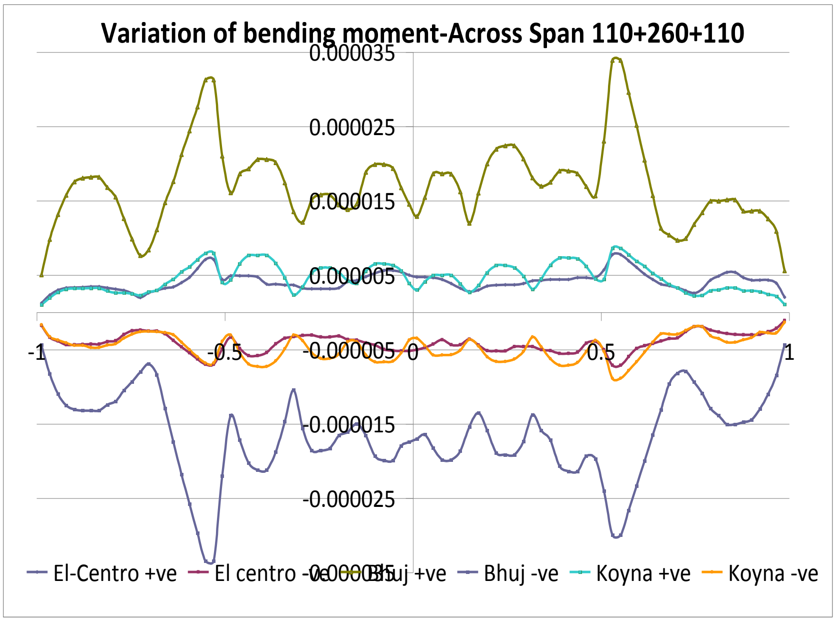

Fig.4 Variation of bending moment across span for 110+260+110m span (Non-dimentionalized)

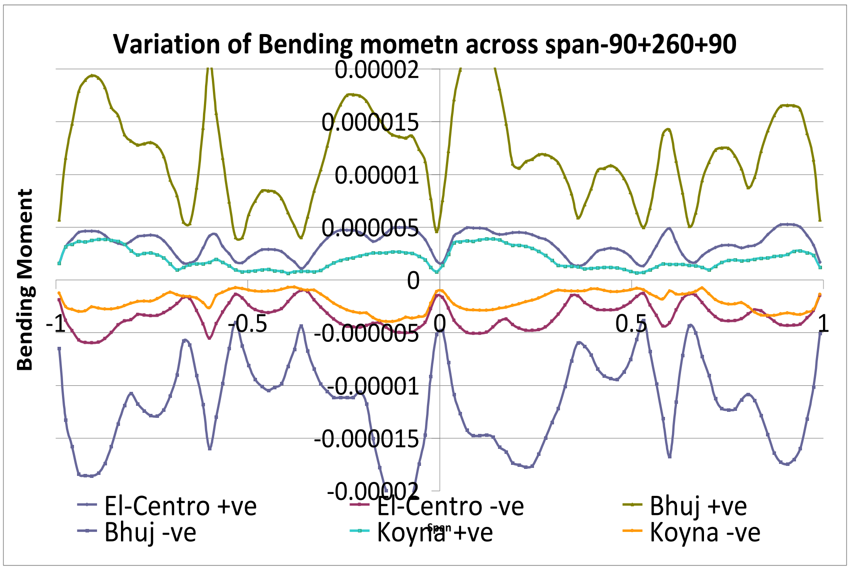

Fig.5 Variation of bending moment across span for 90+200+90m span (Non-dimentionalized)

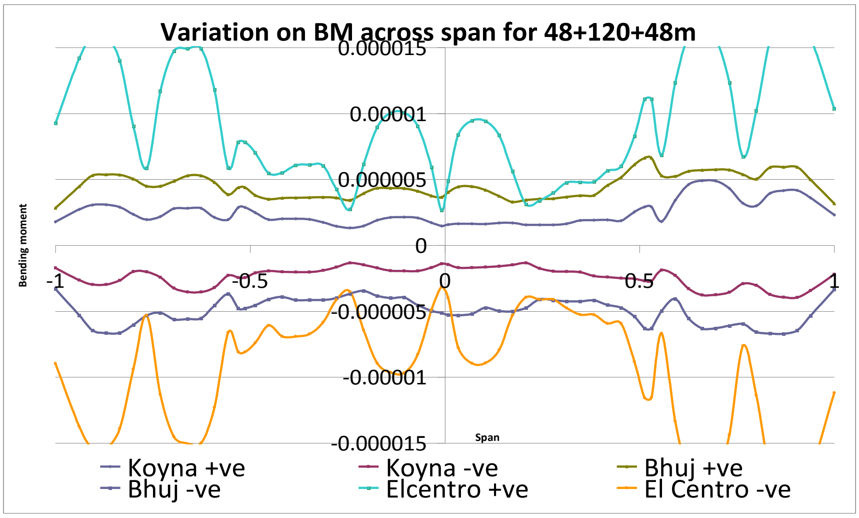

Fig.6 Variation of bending moment across span for 48+120+48m span (Non-dimentionalized)

Fig 7 to 9 gives results for BM across pylon height

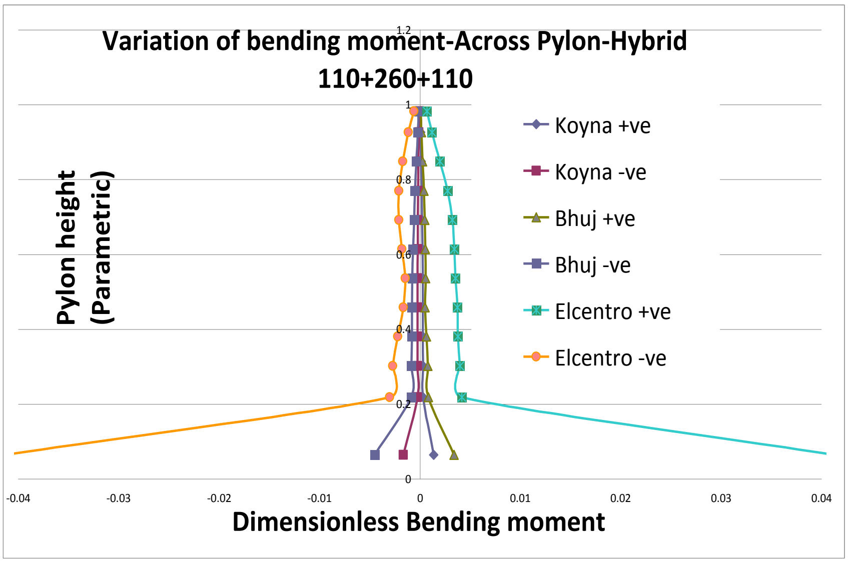

Fig.7 Variation of bending moment across pylon height for 110+260+110m span (Non-dimentionalized)

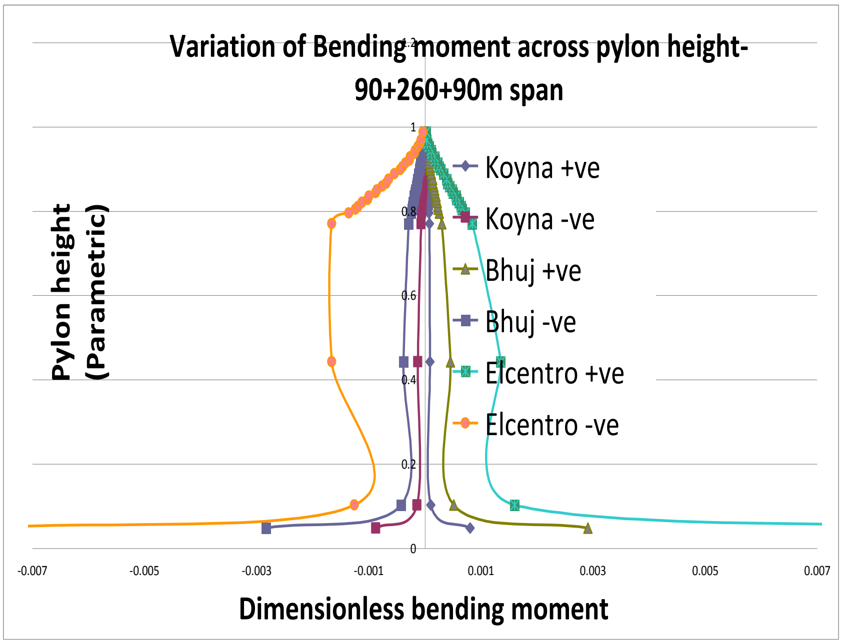

Fig.8 Variation of bending moment across pylon height for 90+200+90m span (Non-dimentionalized)

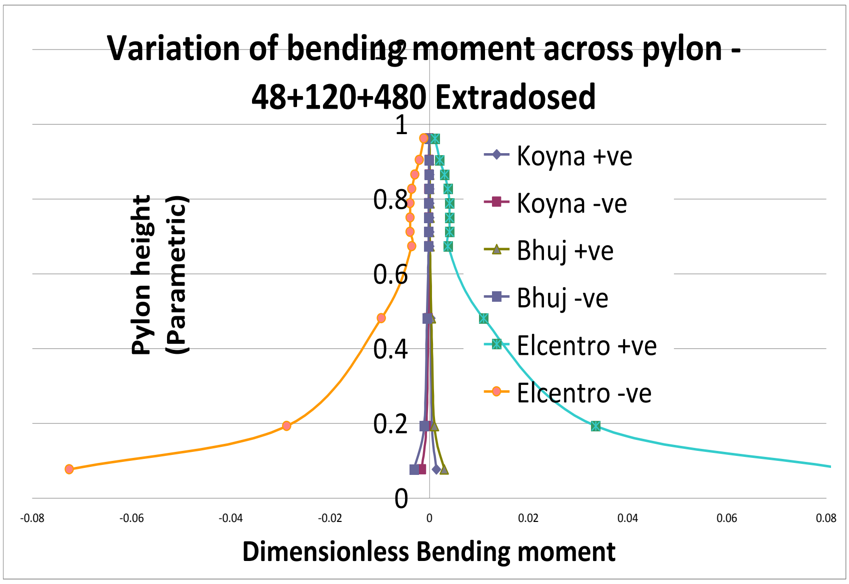

Fig.9 Variation of bending moment across pylon height for 48+120+48m span (Non-dimentionalized)

Forced vibration studies of deck and pylon of three types of bridges reaffirms following facts for extradosed bridge,

- Magnitude of bending moment / shear force is directly proportional to the magnitude of forcing function / Peak Ground Acceleration (PGA).
- With increase in the distance between cables supports the shear force in deck also increases.
- Pylon stiffness does not have any effect on the deck moments/shear.
- It is observed that only for cable stayed bridge with harp shape cable arrangement the shear force reduces at the junction of deck

IV. ANALITICAL STUDY – PGA-EDR RELATIONSHIPS

It is seen in above sections that for time history analysis generally the magnitude of structural actions are directly proportional to magnitude of forcing function ie. Its Peak ground acceleration. Hence it will be possible to relate these quantities as a function of forcing frequency. The term EDR as explained below is used for this purpose.

All data of earthquake records are now days processed using “Seismosignal” software. Seismosignal processes strong motion seismic data. It will generate strong motion parameter such as Fourier and Power spectra, Elastic response spectra and pseudo spectra, over damped and constant ductility in elastic response spectra. Spatial earthquake represents ie. Time history components or all the three direction viz. longitudinal, transverse and vertical are specified. This is data can be obtained from various agencies and can be classified by the Peak Ground Accelerations.

Earthquake Deformation ratio (EDR) Every structure has its stiffness characteristics which controls the behaviour of structure. To account for this variable, the ratio EDR is defined as ratio of maximum seismic displacement to maximum static displacement. These both displacements measured at the same point.
The values of PGA and EDR can be plotted against each other to get correlation by curve fitting.

To obtain the values of EDR for various PGA it is required to simulate earthquake Ground accelerations on analytical model in computer analysis using Time histories which is the time variation of ground acceleration. It is the most useful way of defining the shaking of ground during earthquake. This ground acceleration is descriptive by numerical values at discrete time intervals. Integration of this time acceleration history gives velocity history, integration of which intern gives displacement history.

V. DISCUSSIONS & CONCLUSIONS

Forced vibration studies of three different Extradosed bridges are done. It is noted that;

1. Forced vibration is governed by peak accelerations
2. Design of structures is most often governed by seismic cases and combinations thereof.
3. For cable stayed structures it is difficult to predict dynamic response using usual method
4. Accurate analysis viz. Time history analysis involves time, cost and skill requirements
5. Effect of seismic transmission units or isolators can not be considered in codal methods.
6. The commonly used simplified methods used for analysis are based of theory of dynamics pertaining to SDOF systems. Rules of modal combinations viz SRSS, CQC are used for MDOF systems.
7. It is observed that there is some relationship between Peak Ground Accelerations and the response of structure, relationship between these needs to be established by further study.

Forced vibration studies of deck and pylon of three types of bridges need to be done to find out correlation between peak ground acceleration and Earthquake displacement ratio.

The correlations can be used to quickly predict the expected dynamic response of the highly indeterminate structures like Extradosed cable stayed bridge during preliminary design as exact dynamic analysis at this stage is time and cost consuming and also not very much desired. Exact time histories at all locations are not available and design can be done for expected PGA. Using these relations displacement based design can be done for structure ie. By applying obtained deflections on static structural model. Apart from this the EDR values can be used as a tool for seismic damage index for earthquakes beyond designed values and provides less conservative approach than response spectrum analysis.

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