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PERFORMANCE ASSESSMENT OF AN EXISTING REINFORCED CEMENT CONCRETE T-BEAM AND SLAB BRIDGE USING PUSHOVER ANALYSIS

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Abstract. The objective of this paper is to evaluate the structural condition of an existing reinforced cement concrete T-beam and slab bridge and to check its survivability under two imposed ground motions. A three dimensional finite element model of the bridge was developed using Structural Analysis and Program Software SAP2000. The finite element model of the bridge was subjected to ground motions along the transverse and longitudinal directions. A nonlinear static pushover analysis was carried out and the capacity curves were obtained. The seismic demands and the capacities were established. The performance of the bridge was assessed and the results are reported.

Keywords: T-beam and slab bridge, pushover analysis, capacity spectrum method, performance assessment, structural ductility.

1. Introduction

Road bridges are the most critical means of transportation systems which are susceptible to failure if their structural deficiencies are unidentified. In seismic design of bridges, the structural ductility (performance ductility and displacement ductility (Roy *et al.* 2010)) acts as a crucial element to check the survivability of the bridges under severe earthquakes. Many reinforced concrete bridges suffered severe damages leading to loss of lives and property during the following earthquakes: 1971 San Fernando earthquake of magnitude 6.6, 1989 Loma Prieta earthquake of magnitude 7.1, 1995 Hanshin-Awaji Kobe earthquake of magnitude 7.2, 1994 Northridge earthquake of magnitude 6.7 and the most recent 2011 Tohoku earthquake (Japan) of magnitude 8.9. In India, many existing bridges were constructed before seismic actions were adequately understood. The 2001 Bhuj earthquake of magnitude 7.6 that shook the Indian Province of Gujarat was the most deadly in India's recorded history. This disaster has created awareness among the engineers to determine the structural vulnerability of the bridges which were built before 2001 to develop the required retrofit measures. Therefore, it is the need of the hour to study the performance and check the survivability of the existing bridges. Thus, the objective of this paper is to study the inelastic behaviour of an existing T-beam cum slab bridge using nonlinear static pushover analysis in which the structural performance levels are well understood in a wider range than only at the first yield or near collapse.

The inelastic performance of the bridge when subjected to earthquake shaking can be evaluated with the help of any of the nonlinear analytical methods presented in the guideline documents such as *Applied Technology Council – 40* (ATC-40) (1996), *Federal Emergency Management Agency – 356* (FEMA-356) (1997) and *Federal Emergency Management Agency – 440* (FEMA-440) (2005). Among them, Capacity Spectrum Method (CSM) as specified by ATC-40 is viewed as a powerful tool as it readily gives the graphical depiction of the capacity of the structure (Yu *et al.* 1999) for the imposed ground motions. The performance condition and survivability of the structure can be readily obtained from the capacity spectrum graph which makes this method as advantageous over other methods. In this paper, the performance of an existing reinforced cement concrete (R.C.C.) T-beam and slab road bridge located in Chennai City, Tamilnadu, India was assessed using the nonlinear analysis software package SAP2000 and the results are reported.

2. Capacity Spectrum Method

The CSM, originally developed by Freeman *et al.* (1975), is a graphical procedure for estimating structural load–deformation characteristics and for predicting

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Table 1. Characteristics of parking systems in the main residential districts of Vilnius

Description	Size, in mm
Longitudinal Girder	
Top flange	2500 × 220
Bottom flange	500 × 300
Web	250 × 1400
Cross Girder	200 × 1400
Bent Cap	
Cross Section	1400 × 600
Length	8800
Bent Column	
Diameter	800
Height	5476
Bearing Pad	500 × 320 × 33.5

Fig. 1. Longitudinal view of the bridge

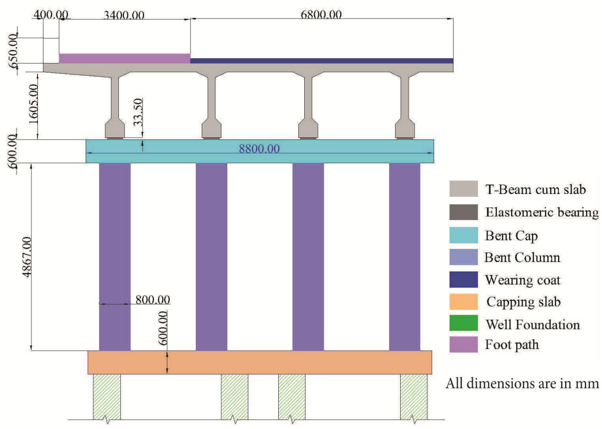

Fig. 2. Cross sectional elevation of the bridge

earthquake damage and structure survivability. The procedure compares the capacity of a structure (in the form of a pushover curve) with the demands on the structure (in the form of response spectra). With this technique, the ability of the structure to resist seismic forces and deformations is depicted graphically by constructing two curves, one representing the capacity of the structure to resist lateral forces and displacements (pushover curve) and the other representing the demand associated with the ground motion. The demand associated with the ground motion is represented by a combined elastic Acceleration and Displacement-Response Spectrum (ADRS) curve for various levels of equivalent viscous damping ratios. The capacity curve is converted to an acceleration-displacement curve by dividing the force by the mass and is then overlaid on the ADRS curve to assess the seismic response of the structure. The seismic performance of the bridge is assessed by overlaying the capacity and demand curve in ADRS format using dynamic properties of the system. The resulting curve is called a capacity spectrum.

In the capacity spectrum, the point at which both the curves interact with the same equivalent viscous damping ratio is the performance point which defines the demand imposed on the structure. The trial and error procedure is used to find the performance point. The CSM assumes that elastic response spectra can be used together with the inelastic capacity curve of a structure to determine the seismic response (Roy *et al.* 2010).

3. Description of the study bridge

The bridge is built over Coovam River in Koyembedu and connects Guindy and Thirumangalam. It is a simply supported R.C.C. T-beam and slab bridge having the total span of 129.7 m with eight equal spans of 16.21 m. The cross sectional details of the bridge components are given in Table 1.

Each span of the superstructure consists of four longitudinal T-beam girders and five cross beams. It is supported on multi-column bents over plain elastomeric bearing pads. Each multi-column bent has four columns which are transversely connected by the bent cap. The bridge piers and abutments are supported on well foundations.

The longitudinal view, the cross sectional elevation and the longitudinal elevation of the bridge are shown in Figs 1–3 respectively.

4. Modeling of the bridge

A three dimensional (3D) finite element model (FEM) of the bridge was created using Structural Analysis and Program Software SAP2000. Spine model (a type of superstructure model) was employed for modeling the superstructure (Priestly *et al.* 1996; Ryan, Richins 2011). The deck edges in each simply supported span were considered rigid. Due to the large in-plane rigidity, the superstructure was assumed as a rigid body for lateral loadings (Nielson *et al.* 2005; Priestly *et al.* 1996; Shatarat, Assaf 2009). The bridge consists of seven multi-column bents and every bent was modeled as a plane frame. The framing action and coupling between columns

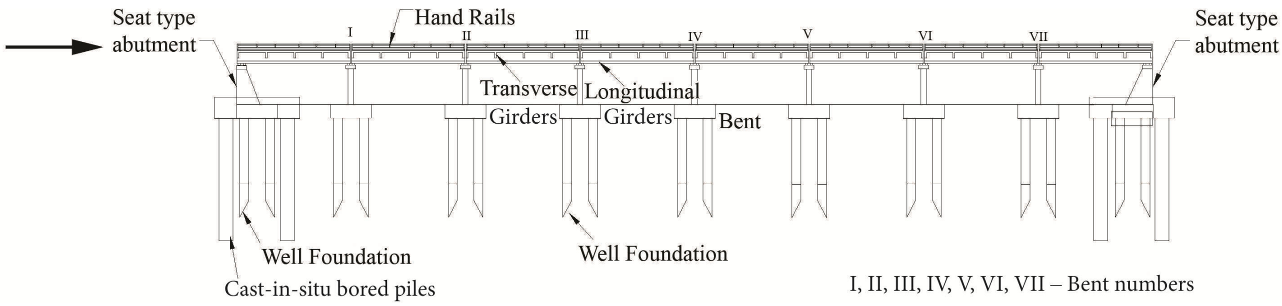

Fig. 3. Longitudinal elevation of the bridge

in the multi-column bent provides seismic resistance in terms of strength and stiffness. A planar frame FEM was developed incorporating all these effects. The bent cap and the columns were modeled as beam-column elements. Effective moment of inertia was taken as $0.7I_g$ (Priestley *et al.* 1996) for reinforced concrete columns which were modeled using Section Designer (Sub programme in SAP2000). The interface between each column and the corresponding geometric centre of the bent cap was considered rigid. The default hinge properties (PMM – P stands for axial force, M stands for M2 moment, and M stands for M3 moment in SAP2000) were assigned to each end of the columns. The base of the column was assumed as fixed. The 3D FEM of the bridge structure was developed using SAP2000 and is shown in Fig. 4.

The girders of the bridge are simply supported over plain elastomeric bearing pads. The horizontal sliding behaviour of the interface between the bearing and girder or cap beam was modeled using linear spring element (El-Gawady *et al.* 2009) and is shown in Fig. 5. The initial stiffness of the spring was calculated from the geometric properties of the pad (Akogul, Celik 2008) using the Eqs (1)–(3):

$$\text{Translational stiffness:} \quad K_H = \frac{GA}{h}, \tag{1}$$

$$\text{Translational stiffness:} \quad K_V = \frac{EA}{h}, \tag{2}$$

$$\text{Translational stiffness:} \quad K_\theta = \frac{EI}{h}, \tag{3}$$

where K_H – lateral stiffness, N/mm; K_V – vertical stiffness, N/mm; K_θ – rotational stiffness, Nmm/mm; G – the rigidity modulus, N/mm^2; E – the Young's Modulus, N/mm^2; A – the cross sectional area of the bearing pad, mm^2; h – the height of the bearing pad, mm; I – the moment of inertia, mm^4.

The expansion joints between the deck slabs, the abutment and deck slab were modeled as gap elements (Fig. 6). Gap element is a compression only element (ElGawady *et al.* 2009; Muthukumar 2003; Nielson *et al.* 2005) to contribute resistance when the relative distance between the adjacent structures is more than the initial gap of 25.40 mm. When the gap closes, pounding occurs and the gap element offers infinite stiffness. The effective stiffness, Keff (Muthukumar 2003) was calculated using the Eq (4).

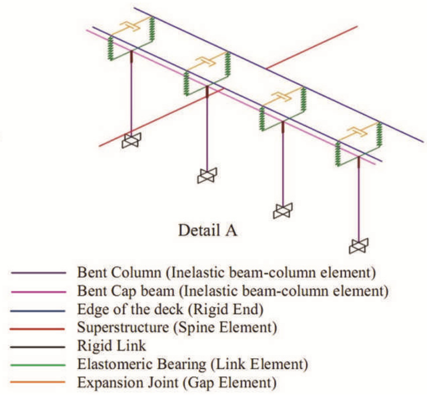

Fig. 4. SAP2000 FEM of the study bridge

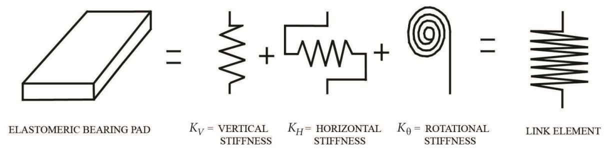

Fig. 5. Elastomeric Bearing Pad Model

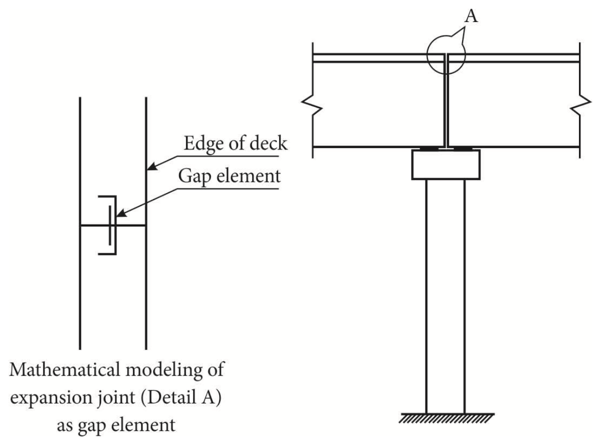

Fig. 6. Expansion Joint Model

$$\text{Effective stiffness:} \quad K_{eff} = k_h \sqrt{\delta_m}, \tag{4}$$

where k_h and δ_m – the impact stiffness parameter and max penetration, mm.

The support provided by the abutment was assumed as fixed against vertical translation and the stiffness properties of the translational spring in the longitudinal and transverse directions were given as per Caltrans design aid (Caltrans 1996) using Eqs (5) and (6). The active and passive soil earth pressures were not considered in the abutment modeling.

$$K_L = 47000\,Wh\,, \tag{5}$$

where W and h – the width and height of the back wall, mm.

$$K_T = 102000\,b\,, \tag{6}$$

where b – the width of wing wall, mm.

5. Analysis

5.1. Modal analysis

The study bridge (Koyembedu Bridge) is symmetric, however both the longitudinal and transverse responses are significant because the lateral load may lead to stability problem in the transverse direction and unseating of deck is most common in the longitudinal direction. Modal analysis predicts the dynamic characteristics of structures under vibrational excitation. Hence, the modal analysis of the bridge was carried out to find the dynamic characteristics of the bridge such as modal participation, mode shapes etc. Table 2 shows the modal period and mass participation of the bridge in both the longitudinal and transverse directions and the corresponding mode shapes are presented in Fig. 7 and Fig. 8 below.

Table 2. Modal periods and mass participation of the bridge

Mode number	Period, s	Mass excited in direction	
		longitudinal, %	transverse,%
1	0.422	0	84.56
2	0.279	93.36	0

In the fundamental mode (Mode 1), 84.56% of the total mass of the bridge structure participated in the vibration of the structure in the transverse direction. In the second mode (Mode 2), 93.36% of the total mass participated in vibrating the bridge structure in the longitudinal direction. The modal load pattern was chosen for conducting the pushover analysis (Modal Pushover Analysis, MPA) of the bridge.

5.2. Pushover analysis

Pushover analysis is an inelastic analysis (Chiorean 2003; Gu, Zhuo 2012; Kappos *et al.* 2005; Kim, D'Amore 1999; Ryan, Richins 2011; Sharma *et al.* 2013) which gives nonlinear response of the structure in global Force – Displacement format (capacity curve). This analysis is used to determine displacement capacity of a structure and it demonstrates the sequential formation of plastic hinges at every push step. Following the initial conditions obtained due to gravitational forces, the pushover analysis was performed in both the directions considering P–Δ effects. Only the fundamental vibration mode is considered for pushover analysis. Effect of higher modes may also be considered for better understanding of the structural performance of the bridge. The displacement pattern was configured from the mode shapes obtained. The bridge was subjected to lateral forces distributed proportionally over the span of the bridge in accordance to the product of mass and displaced shape. The capacity curves of the bridge in transverse and longitudinal directions are shown in Figs 9–10. It indicates the fact that the capacity remains almost same in both directions because of the circular piers.

The pushover curve for Mode 1 is shown in Fig. 9. The figure indicates that the first yielding occurred at a base shear of 7961.26 kN. Beyond the first yield, the control node displacement increases with the increase in base shear. The softening of the pushover curves associated with the progressive formation of plastic hinges was noticed in the multi-column bents of the bridge structure, with increasing lateral forces. The first mode caused a global plastic mechanism and increasing force intensity leading to the rotation of the bridge structure about its base (bottom local plastic mechanism). The control node continued to move in the direction of the application of lateral force. The pushover curve displayed normal behaviour without any reversal. The formation of mechanism reduced the stiffness and caused an incremental displacement.

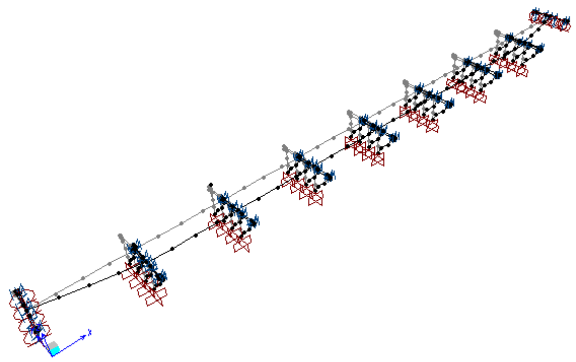

Fig. 7. Mode shape in the transverse direction (Mode 1)

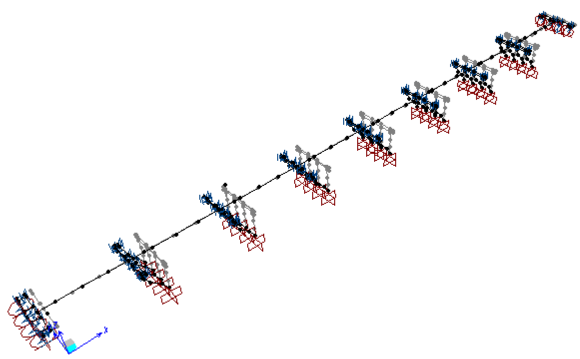

Fig. 8. Mode shape in the longitudinal direction (Mode 2)

The pushover curve for Mode 2 is shown in Fig. 10. From the pushover curve it was found that the overall strength of the system appeared to be higher (i.e. yielding occurred at a higher level of base shear). The shear force at the base of the structure in the longitudinal direction was much larger than the base shear in the transverse direction. In the longitudinal pushover analysis, when the push load was applied in the longitudinal direction, the expansion joints which are provided between the adjacent sides of a deck joint, permitted relative translations and rotations at both sides of the bridge decks.

5.3. Hinge status of the bridge in transverse and longitudinal directions

The performance level of the bridge was studied by monitoring the sequential formation of the hinges at each push step.

The hinge status at the ultimate step of the pushover analysis in transverse and longitudinal directions is shown in Figs 11 and 12 and the details are summarized in Table 3.

In the transverse direction, at the initial step of the pushover analysis the bridge displaced under its own weight. The behaviour of the hinges under self-weight was in the linear range. The structure was pushed further up to the last step (step 7) and it was found that the hinges at the bottom of all the columns at IV bent failed, as their rotation capacity had been exceeded, and four hinges were over their collapse prevention (CP) performance level, and the hinges at the bottom of the III bent and V bent at the Life Safety (LS) performance level.

In the longitudinal directions, at the initial step the bridge displaced under its self-weight. In the longitudinal push (Mode 2), it was observed that the performance of the hinges in all the columns was at the initial yield level in the first step. The structure was pushed further up to the last step (step 8) and it was found that the hinges at the bottom of all the columns of all the bents excluding the exterior bents at either end were at the Life Safety (LS) performance level, and the rotation capacity was within the acceptable limit. The hinges assigned to the bent cap beams were in the elastic state.

6. Results and discussions

6.1. Transverse pushover analysis

At the initial step the bridge displaced under its own weight. The behaviour of the hinges under self-weight was in the linear range. By observing the behaviour of the structure in the transverse push (Mode 1), it was found that the performance of the hinges was at the initial yield level till the first four steps. At step 5, the plastic hinges at the bottom of the III, IV and V bents were at the Immediate Occupancy (IO) performance level. At step 6, the hinges at the bottom of the III and IV bents reached the

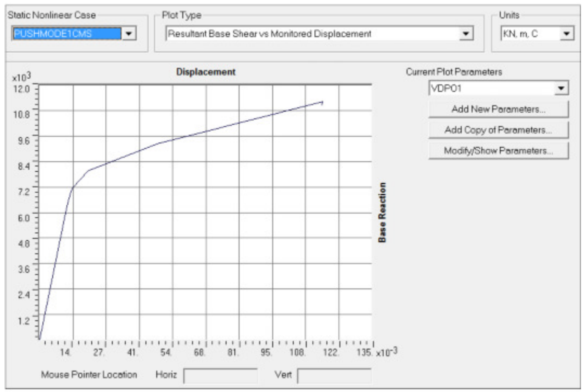

Fig. 9. Base shear vs. displacement in the transverse direction

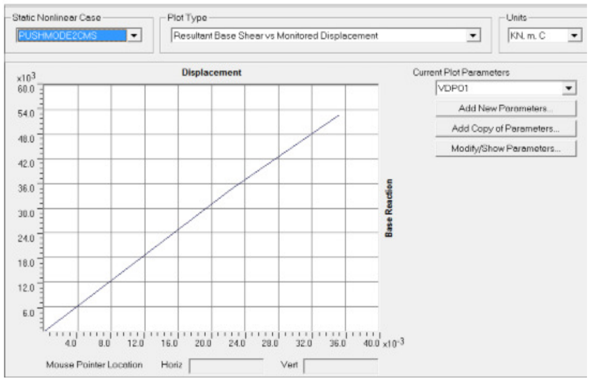

Fig. 10. Base shear vs. displacement in the longitudinal direction

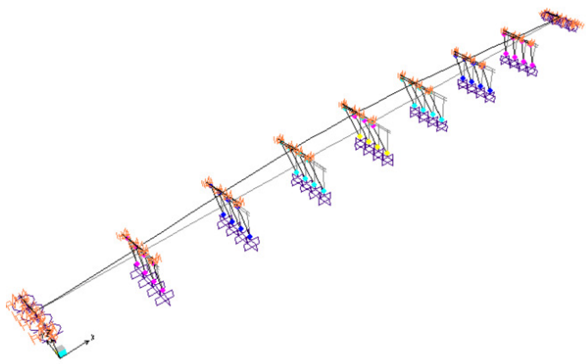

Fig. 11. Plastic hinge formation in the last transverse pushover step

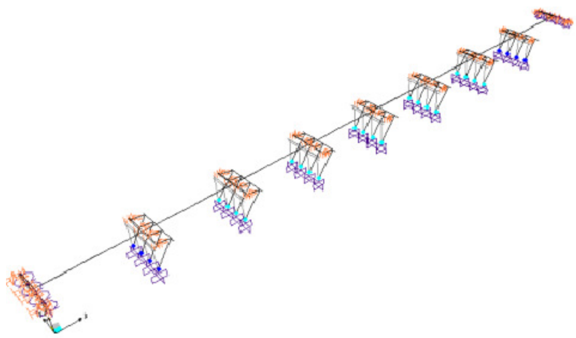

Fig. 12. Plastic hinge formation in the last longitudinal pushover step

Table 3. Hinge status of the bridge at different steps of pushover analysis

Steps	Displacement, m	Number of hinges						
		A–B	B–IO	IO–LS	CP–C–D	D–E	> E	Total
Pushover analysis in transverse direction								
Initial	0	56	–	–	–	–	–	56
Yield	0.045	18	38	–	–	–	–	56
Ultimate	0.110	0	4	28	22	–	2	56
Pushover analysis in longitudinal direction								
Initial	0	28	–	–	–	–	–	28
Yield	0.043	–	28	–	–	–	–	28
Ultimate	0.116	–	–	4	24	–	–	28

Note: IO – Immediate Occupancy; LS – Life Safety; CP – Collapse Prevention, C – Collapse.

Life Safety (LS) performance level. The structure was pushed further up to the last step (step 7), and it was found that the hinges at the bottom of all the columns at bent IV failed, as their rotation capacity had been exceeded, and four hinges were over their Collapse Prevention (CP) performance level, and the hinges at the bottom of the III and V bents at the Life Safety (LS) performance level. The hinges assigned to the bent cap beams were in the elastic state. As a result, the structure failed due to global instability. When the performance levels of the hinges exceed the Collapse Prevention (CP) performance level, it indicated that significant damages have occurred in the structure. The damages may be concrete cracking, reinforcement yielding and major spalling of concrete which require either closure of the bridge structure for repair or partial or permanent replacement of the structure. In the study bridge, as the hinges at the bottom of the mid-bent have exceeded the CP performance level, the repair of the IV bent will effectively enhance the performance of the structure.

6.2. Longitudinal pushover analysis

At the initial step the bridge displaced under its self-weight. In the longitudinal push (Mode 2), it was observed that the performance of the hinges in all the columns was at the initial yield level in the first step. At step 2, all the plastic hinges yielded, with all the hinges at the bottom of the I, II and VII bents at the Immediate Occupancy (IO) performance level, and the hinges in the III, IV, V and VI bents at the Life safety (LS) performance level. The structure was pushed further up to the last step (step 8) and it was found that the hinges at the bottom of all the columns of all the bents excluding the exterior bents at either end were at the Life Safety (LS) performance level, and the rotation capacity was within the acceptable limit. The hinges assigned to the bent cap beams were in the elastic state.

6.3. Capacity Spectrum Method

The capacity spectrum of the bridge in transverse and longitudinal directions for El Centro and Kobe earth quakes were developed and are shown in Figs 13–16. In the capacity spectrum, the blue line represents capacity of the bridge, red line represents the demand from the earthquake (for various damping ratios) and the green line represents single demand spectra with variable damping.

Referring Figs 13–14, for an earthquake similar to the El Centro earthquake, it was found that the bridge capacity curves extend through the envelope of the demand curves for both transverse and longitudinal directions indicating that the bridge would survive. In Mode 1, the response of the bridge was governed by the transverse demand with an effective damping of 25.8% which means that the structure would experience 86 mm displacement in the transverse direction with 25.8% of the energy dissipated by damping. It is about 5 times that of inherent damping indicating the ability of the structure to undergo large amplitude cyclic deformations in the inelastic range, without a substantial reduction in the strength. In the longitudinal direction (Mode 2), the energy dissipated by damping was little above 5% inherent viscous damping.

For an earthquake similar to Kobe earthquake, the capacity curves of the bridge did not extend through the envelope of the demand curves for both transverse (Fig. 15) and longitudinal (Fig. 16) directions indicating that the demand was greater than the capacity and proving that the bridge would not survive. The structural ductility of the bridge in the transverse and longitudinal directions for the imposed ground motions are summarized in Table 4.

In the transverse direction the displacement ductility (ratio of ultimate displacement to yield displacement) for Mode 1 was 2.444. In the transverse mode the structure indicates large energy absorption capacities in the inelastic range, without a significant loss of strength and stiffness. In the longitudinal direction the displacement ductility (Mode 2) was 2.704. When the bridge is subjected to an earthquake similar to El Centro earthquake its performance displacement in the transverse direction is 78.18% of its ultimate displacement, while in the longitudinal direction the performance displacement is 29.31% of its ultimate displacement. This means that the bridge has more displacement capacity reserved in longitudinal direction than in transverse direction. The longitudinal ductility is better than the transverse ductility.

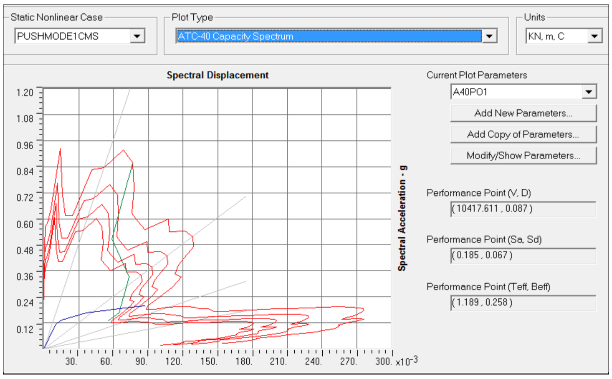

Fig. 13. Capacity spectrum in the transverse direction for El Centro earthquake

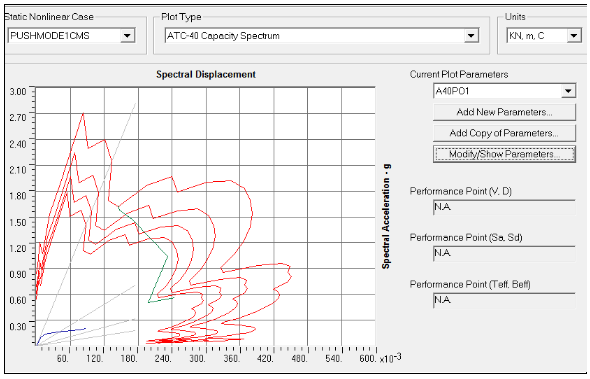

Fig. 15. Capacity spectrum in the transverse direction for Kobe earthquake

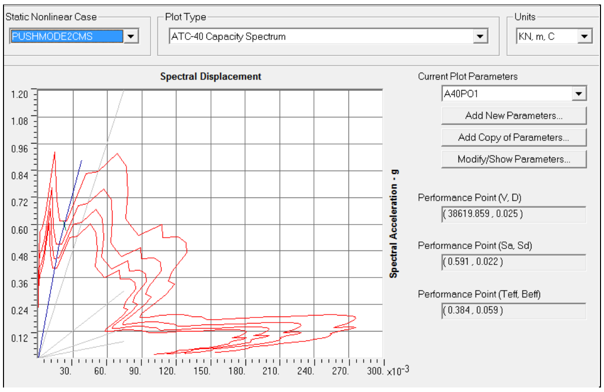

Fig. 14. Capacity spectrum in the longitudinal direction for El Centro earthquake

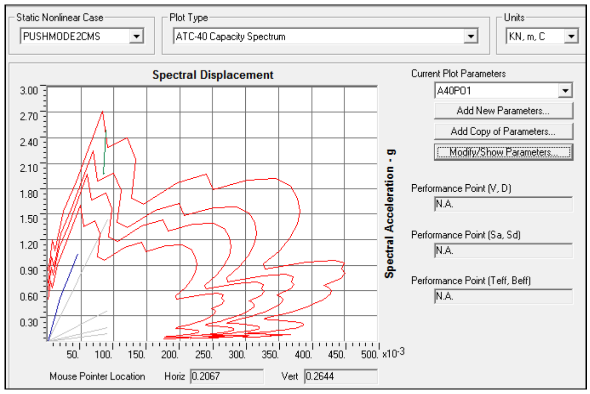

Fig. 16. Capacity spectrum in the longitudinal direction for Kobe earthquake

Table 4. Structural ductility of Koyembedu Bridge in transverse and longitudinal direction

Earthquake	Yield displacement, mm	Ultimate displacement, mm	Performance displacement, mm	Displacement ductility	Performance ductility
Transverse direction					
El Centro	45	110	86	2.444	1.279
Kobe	45	110	N.A.	N.A.	N.A.
Longitudinal direction					
El Centro	42.9	116	34	2.704	3.412
Kobe	42.9	116	N.A.	N.A.	N.A.

Note: N.A. – not applicable.

7. Conclusions

The main category of bridge performances is the structural condition, structural integrity and functionality. In this paper the structural condition of an existing R.C.C. T-beam and slab bridge was assessed and its survivability under two imposed ground motions was studied. A 3D FEM of the bridge was developed using SAP2000. A non-linear static pushover analysis was carried out and the capacity curves were obtained. The seismic demands and the capacities were established and the conclusions are the following.

1. The bridge consists of 28 columns, four columns at each bent. 56 hinges were pre-assigned in the 3D FEM. The formation of plastic hinges was in sequence on a step-to-step basis. From the pushover analysis results, in the transverse direction, at the ultimate pushover step the plastic hinges in all the bents were in safe performance levels in the range from immediate occupancy to life safety performance level except the columns in bent IV. In bent IV the plastic hinges in all the columns reached the structural stability performance level according to *ATC-40* document beyond which the bent would collapse leading to jeopardy. In the longitudinal direction hinges

were formed at the bottom of the columns. The range of performance level was from life safety to structural stability damage levels and a specific check against collapse is probably warranted.

2. When the bridge is subjected to an earthquake similar to El Centro earthquake its performance displacement in the transverse direction is 78.18% of its ultimate displacement, while in the longitudinal direction the performance displacement is 29.31% of its ultimate displacement. This means that the bridge has more displacement capacity reserved in longitudinal direction than in transverse direction. The longitudinal ductility is better than the transverse ductility. Thus, retrofitting plans for enhancing effective transverse performance are favourable. When subjected to an earthquake similar to Kobe the bridge neither presents a good displacement capacity in transverse nor longitudinal direction as the capacity of the bridge does not meet the higher demand.

3. CSM is a powerful tool to visualize the relationship between demand and capacity of a bridge.

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