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# Nonlinear behavior of concrete end diaphragms in straight slab-girder bridges

**H. Hojat Jalali and S. Maleki***

*Department of Civil Engineering, Sharif University of Technology, Tehran, P.O. Box 11155-9313, Iran.*

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**Abstract.** The seismic behavior of concrete end diaphragms of bridges has not been studied before and there are no significant design provisions available. According to the American Association of State Highway and Transportation Officials (AASHTO), the end diaphragms being part of the seismic load path have to remain elastic under the prescribed seismic design forces, regardless of the type of bearings used. In this paper, using a three-dimensional finite element model and nonlinear time history analyses, the behavior of reinforced concrete end diaphragms in straight single-span slab-girder bridges has been investigated. The results are compared to AASHTO's design provisions. It is concluded that for slab-girder concrete bridges, the concrete diaphragms remain elastic under design earthquake loading. It is also concluded that AASHTO's recommended seismic design force for end diaphragms could be unsafe in most cases.



## 1. Introduction

Diaphragms, as transverse members between girders, play an important role in the stability of bridge girders and distribution of vertical and lateral loads. They prevent the girders from twisting during the process of construction and help in distributing vertical live loads among girders, and transfer lateral loads (wind or earthquake) to the supports. They also provide restraint for the lateral-torsional buckling of the girders. Diaphragms at the ends of a bridge and over the supports are called End Diaphragms (EDs) while the diaphragms between the supports and across the span are called Intermediate Diaphragms (IDs). These members are usually made of steel in cross-frame form for steel bridges and in the form of a Reinforced Concrete (RC) beam for concrete girder bridges. Based on their behavior, the cross-frame diaphragms can be classified as axial, and the beam diaphragms as flexural diaphragms.

EDs provide a load path for the seismically induced loads, since seismic forces at the deck would have to pass through the diaphragms to arrive at the top of bearings. Despite their important role in transferring seismic loads to the bearings and substructure, no extensive experimental and/or numerical researches have been conducted concerning their seismic behavior. Consequently, detailed seismic design of end diaphragms have not been included in bridge design codes.

A literature survey indicates that many studies have been performed on IDs and their effects on structural response, but not on EDs. Sithichaikasem and Gamble (1972) [1] performed a parametric study for several simply supported straight bridges to investigate the effect of IDs on the overall load distribution in Prestressed Concrete (PC) girder and slab bridges. Some parameters considered were the location, number and stiffness of IDs, girder spacing to span length ratio, girder to slab stiffness ratio, and diaphragm to girder

*. *Corresponding author. Tel.: +98 21 66164294; Fax: +98 21 66014828
E-mail address: smaleki@sharif.edu (S. Maleki)*

flexural stiffness ratio. It was found that in order for the diaphragms to be effective, the flexural stiffness of the IDs should be selected carefully, and diaphragms with flexural stiffness, beyond a certain limit, would prove to be detrimental. It was also found that the number of diaphragms did not have a significant effect on the load distribution, and recommended to eliminate IDs for PC girder bridges. Wong and Gamble (1973) [2] continued the theoretical study of Sithichaikasem and Gamble (1972) [1] for continuous bridges and found that the IDs were mostly effective for bridges with large girder spacing to span length ratio. The study suggested that in most cases the IDs were harmful, and recommended that the IDs be eliminated for highway bridges. Sengupta and Breen (1973) [3] performed an extensive experimental and numerical study on the influence of EDs and IDs on the load distribution in simple-span PC-girder and slab bridges under both static and dynamic (cyclic and impact) loads. The experimental part included four 1 to 5.5 scale bridges with and without diaphragms, with variables being span length, number, location, and stiffness of the diaphragms. The experimental results were then used to validate the numerical models for further investigation. Based on this study, it was found that diaphragms did not have any effect on the dynamic properties (natural frequency and damping) of the bridge, when subjected to cyclic loading. Under impact loads, the diaphragms reduced the energy absorption of the girders and hence increased the vulnerability of the girder. EDs were found to increase both cracking and ultimate load of the concrete girders. Therefore, it was emphasized to use EDs in PC-girder and slab bridges. Kostem and deCastro (1977) [4] studied the effect of IDs on the lateral load distribution of vertical live loads in straight simple-span PC-girder and slab bridges using finite element analyses of two existing field-tested bridges. It was found that only 20 to 30 percent of stiffness of IDs contributes to lateral load distribution. This study also suggested that increasing the number of IDs does not necessarily result in a more uniform load distribution at critical sections. Cheung et al. (1986) [5] found that there was general disagreement between researchers on the effectiveness of IDs in lateral distribution of live loads and suggested that a thorough study needs to be performed to better understand the structural behavior of IDs in girder and slab bridges.

Abendroth et al. (1995) [6] incorporated a wide-ranging literature review and survey of design agencies. They tested a full scale, simple span, prestressed concrete girder bridge model and conducted finite element analyses of the bridge model considering both pinned and fixed end conditions. They concluded that the vertical load distribution is independent of the type and location of IDs; the horizontal load distribution is a function of the type and location of IDs; constructional details at the girder supports form substantial rotational end restraint for both vertical and horizontal loading; and the steel diaphragms can essentially replace reinforced concrete IDs for vertical load distribution. Later, Abendroth et al. (2003) [7] analyzed bridge models for a lateral impact load both at and away from the location of the diaphragms. They found that the reinforced concrete IDs provided largest degree of impact protection when the impact load was applied at the diaphragm location.

The seismic behavior of steel diaphragms has been investigated by Zahrai and Bruneau (1999) [8], Maleki (2001) [9] and more recently by Carden et al. (2006, 2007) [10-12], Bahrami et al. (2009) [13] and Celik and Bruneau (2009) [14]. The findings of these researches show that EDs can be used as energy dissipating devices and will enhance the overall seismic performance of slab-girder bridges. However, no major publication can be found for seismic behavior of reinforced concrete diaphragms. Therefore, these members require further attention, and more research have to be carried out in this field.

American Association of State Highway and Transportation Officials (AASHTO, 2007) [15] does not require any seismic analysis for single-span bridges regardless of seismic zone. However, it recommends a minimum seismic design force equal to the products of site coefficient ($S$), acceleration coefficient ($A$) and tributary permanent load, for the design of superstructure supporting elements. Note that, this method does not have an $R$ factor, or $R$ is assumed to be one, implicitly. Herein, this design method is called the $S \times A$ method and is described in articles 3.10.9.1 and 4.7.4.2 of AASHTO. It is also mandatory to have end diaphragms to transfer lateral loads to the substructure. It is implicitly understood that the code intends to use the above force for the design of end diaphragms and bearings of single-span bridges, as well.

In this paper, the behavior of reinforced concrete EDs in single-span slab-girder bridges has been investigated using a three-dimensional finite element model. In order to achieve this goal, linear and nonlinear time history analyses of straight slab-girder bridges under earthquakes with different intensity and frequency contents have been performed. The effect of diaphragm to girder connection has been studied. Both pinned and elastomeric bearings are considered. Span lengths of 10 m, 20 m and 30 m are included. Based on these analyses, the seismic force distribution in diaphragms is determined and results are compared to forces introduced in EDs using the $S \times A$ method. The cyclic moment-rotation curves of diaphragm beams are plotted and the absorbed seismic energies are calculated. Whether or not the EDs remain elastic under the design basis earthquake has also been investigated.

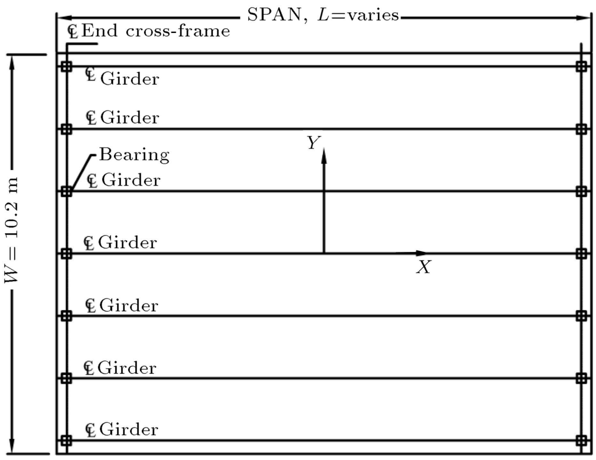


**Figure 1.** Slab-girder bridge plan view.

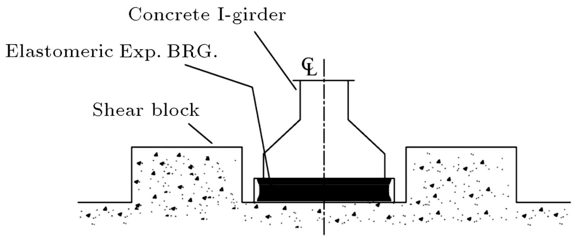


**Figure 2.** Elastomeric bearing detail.

## 2. Bridge description

Slab-girder bridges are the most common type of bridge construction used for short- to medium-range spans. They consist of a concrete deck spanning over concrete or steel longitudinal beams. For longer spans, shear connectors are provided at the top of the beams to ensure composite action with the concrete deck for gravity loading. A typical plan view of a slab-beam bridge is shown in Figure 1.

A typical elastomeric bearing detail is shown in Figure 2. It is assumed that the bearing is flexible along the longitudinal direction, but is restrained with shear blocks in the perpendicular direction to prevent roll-out failure. Lateral load resistance in the transverse direction is provided by means of RC end diaphragms, located between each girder.

## 3. Analysis model

Seismic behavior of reinforced concrete EDs in single-span slab-girder bridges has been investigated using a three-dimensional Finite Element (FE) model. The three-dimensional FE model utilizes quadrilateral shell elements for the deck slab with eccentrically stiffened beam elements for the girders. The eccentricity of the girders is taken into account by using body restraint between the centroid of the concrete slab and the centroid of the concrete girders. This will capture the contribution of girders' weak-axis moment of inertia to the superstructure stiffness for transverse loading. The diaphragm beams are modeled with frame elements connected rigidly to the girders. A rigid link is used to fill the offset between the diaphragm and the girder centerlines. The ends of the girders are attached to two springs representing the elastomer's vertical and lateral stiffness. For the pinned end condition these stiffnesses are set to very high values. The modeling of superstructure is consistent with recommendations of Mabsout et al. (1997) [16] and Maleki (2002) [17].

For single-span bridges, the abutment stiffness is ignored. The justification for this assumption is described herein. In the transverse direction, abutments are, in general, much stiffer than concrete end diaphragms. Since the two are modeled as springs in series, the abutment will not contribute overall, and only the effect of EDs is considered. In the longitudinal direction, abutment stiffness is much higher than the elastomer's shear stiffness. Being connected in series, an equivalent spring will have only the effect of elastomer's stiffness. Note that, in case the seismic motion is towards the abutment, the added stiffness of the soil would make the assumption even truer.

The parameters involved in this study are the span length and bearing type. For the latter parameter, two cases are considered: elastomeric bearings with stiffness as shown in Table 1, and an extreme case of infinite stiffness in the transverse direction, identified as a pinned support condition. Two cases (rigid and simple) of diaphragm to girder connection have been considered. The girder types, lateral and vertical stiffness of elastomers and other dimensions and properties of bridges used in the analyses are given in Table 1.

Material nonlinearity is considered for the diaphragm beams. Material properties are assumed to be 29 MPa for the concrete compressive strength and 295 MPa for the yield strength of both longitudinal and transverse steel reinforcements. For the nonlinear time history analyses, material stress-strain curves are required for both concrete and reinforcing steel. The

**Table 1.** Bridge properties used in analyses.

| Span (m) | Width (m) | Slab thickness (m) | AASHTO girder type | Number of girders | Elastomer lateral stiffness (kN/m) | Elastomer vertical stiffness (kN/m) |
|---|---|---|---|---|---|---|
| 10 | 10.2 | 0.2 | II | 7 | 1000 | 200000 |
| 20 | 10.2 | 0.2 | IV | 7 | 1000 | 200000 |
| 30 | 10.2 | 0.2 | VI | 7 | 1000 | 200000 |

Mander et al. (1988) [18] model is used to represent the uniaxial stress-strain behavior for concrete. Due to existence of a close-spaced transverse reinforcement, the Mander confined stress-strain curve is applied. It is to be noted that the confined compressive strength is a function of confinement and is somewhat higher than the concrete compressive strength. Since the concrete tensile strength affects the initial stiffness of RC members, the concrete tensile strength for confined concrete is also included.

This study includes two sets of analyses: linear and nonlinear time history analyses using real ground motions for the parametric study, and nonlinear time history analyses using scaled ground motions for determining elastic limits of the EDs. For the parametric study, $0.25\times0.45$ m diaphragm beams with $6\varphi20$ as longitudinal reinforcements and $\varphi10$@10 cm as transverse reinforcements are used.

According to AASHTO, article 4.6.2.8.2, the end diaphragms, being part of the seismic load path, have to remain elastic under the prescribed design seismic forces, regardless of the type of the bearings used. To verify whether the diaphragms remain elastic under the design earthquake forces, nonlinear time history analyses were carried out using scaled ground motions along with diaphragm sections shown in Table 2. These are assumed to be minimum sections used in practice for the girder heights used. For this part of the analyses the ground motions were scaled to a PGA of 0.4 g.

The earthquake inertial force is mainly because of the mass of the deck. Therefore, it is usually assumed that a great proportion of the earthquake force is concentrated at the bridge deck center of mass. This force has to pass through the end diaphragms to reach the bearings. When the bridge is subjected to a horizontal load at the deck level, moment and axial forces are induced in the EDs. It is widely assumed that this horizontal force is equally divided among the diaphragm beams as an axial force. To the contrary, analyses show that this assumption is not true and the axial forces vary across the bridge width as shown in Figure 3. In fact, the ED beams at the ends of the bridge absorb more axial forces than the middle ones. In addition, because of the offset between the deck center of mass and that of the EDs, flexural

**Table 2.** Diaphragm properties used in analyses with scaled ground motions.

| Span (m) | Diaphragm section $b\times h$ (m×m) | Longitudinal reinforcement | Transverse reinforcement |
|---|---|---|---|
| 10 | $0.25\times0.45$ | $6\phi20$ | $\phi10$@10 cm |
| 20 | $0.35\times0.70$ | $10\phi20$ | $\phi10$@10 cm |
| 30 | $0.35\times0.80$ | $12\phi20$ | $\phi10$@10 cm |

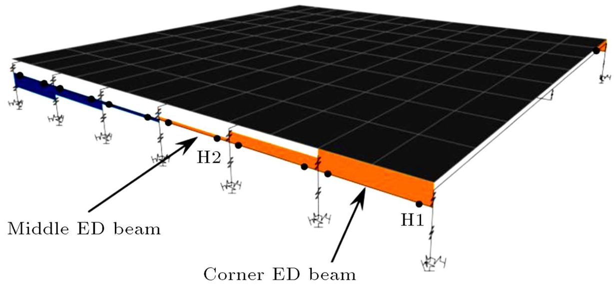


**Figure 3.** Variation of seismic axial force across the bridge width.

moments are also introduced in the diaphragm beams. Therefore, the corner diaphragms are subjected to both axial forces and flexural moments, while the middle diaphragms are mainly subjected to flexural moments. This, in effect, alters the hysteretic behavior of the diaphragm beams.

The nonlinear behavior of diaphragms is examined using lumped plastic fiber hinges at the two ends of each ED beam in the nonlinear analyses (Figure 3). The fiber hinge computes a moment-curvature relation in the bending direction for varying levels of axial load. This interaction between moment and axial force, and the distribution of inelastic action throughout the section is obtained automatically by assigning particular stress-strain relationships to individual discretized fibers in the cross section. The stress-strain relationships correspond to confined concrete and longitudinal steel reinforcement. The definition of each fiber in the cross section of the diaphragm includes the area, centroidal coordinates and material type for which a stress-strain relationship has been defined previously. The fiber model used can represent the loss of stiffness caused by concrete cracking, yielding and strain hardening of reinforcing steel. Therefore, it is successful in representing degradation and softening after yielding; however pinching and bond slip are not considered. Shear and torsion behaviors of the cross section are represented elastically.

Modeling the diaphragms and their connections to the main girders is an important part of this study. The connection between a cast-in-place diaphragm and precast girders is of great importance and whether it is a moment-resistant or a simple (pinned) connection, greatly affects the structural behavior of the diaphragms. A moment (rigid) connection is a connection with adequate top and bottom reinforcements in the diaphragm beam flanges crossing the bridge girders. Any connection that does not satisfy these conditions can be considered as a simple connection. It is observed that in practice this connection detail is usually made with rebars in the diaphragm beam flanges continuously crossing the girders. Therefore, the majority of the EDs, if designed properly, can be assumed to have a moment connection. However, in this study, both moment and simple connections are considered.

## 4. Seismic loading

As explained earlier, according to AASHTO, articles 3.10.9.1 and 4.7.4.2, single-span bridges require no seismic analysis and only the connection of the superstructure to substructure is designed for an acceleration of $S \times A$; where, $A$ is the peak ground acceleration for the seismic zone and $S$ is the site coefficient. For acceleration of 0.4 g and site coefficient of 1.2, this force equals to 0.48 times the tributary weight of the structure. In order to investigate the adequacy of this design force, linear and nonlinear time history analyses were performed. To consider earthquakes with different frequency contents and magnitudes, the 1940 El Centro ground motion with peak ground acceleration of 0.313 g, the 1994 Northridge (Sylmar station) time history with peak ground acceleration of 0.84 g, and the 1971 San Fernando (Pacoima Dam station) time history with peak ground acceleration of 1.226 g are used in this study.

The response spectra of all ground motions are shown in Figure 4. Note that the Sylmar and San Fernando ground motions, unlike El Centro, are more impulsive and have their greatest effects on structures with periods in the range between 0.3 and 0.6 sec. Most seismic codes define the Maximum Considered Earthquake (MCE) to be at least 1.5 times stronger than the design earthquake. Therefore, these two ground motions can be considered to be an MCE for illustrative purposes, whereas El Centro is a Design Basis Earthquake (DBE).

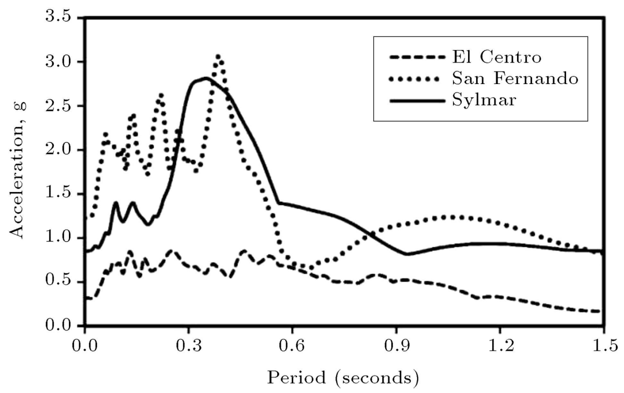


**Figure 4.** Response spectra for considered earthquakes.

## 5. Analyses results and discussion

The main objective of this section is to conduct a parametric study to evaluate the plastic behavior of EDs using nonlinear time history analyses under real ground motions. Two hinges are used for the evaluation of seismic behavior of EDs in the nonlinear analyses: Hinge H1 for the corner diaphragm beams and H2 for the middle diaphragm beams (Figure 3).

As stated before, material and geometrical nonlinearities are considered in these analyses and the bridge span and bearing type and diaphragm/girder connection are the varying parameters.

Figures 5 and 6 compare the maximum axial forces generated in the corner ED beams for both bearing support types. In the legend of these figures letters "M" and "S" denote moment and simple connections for the diaphragm beam, respectively; while "E" and "P" denote elastomeric and pinned bearing types, respectively. As observed from Figures 5 and 6, both tensile and compressive forces are present. This is an important finding since these concrete members are not normally designed to accommodate tensile forces. It can be observed that in most cases the compressive axial forces are greater for moment connections, while for simple connections the tensile forces are larger in magnitude.

Figure 7 shows the variation of base shear versus span length for both bearing and connection types under different earthquake loadings. As observed from Figure 7, base shear is smaller in magnitude when moment connections are used. This reduction is related to hysteretic energy dissipation due to nonlinear flexural behavior. Therefore, for the remaining part of the paper, and, for the sake of brevity, only the results of EDs with moment connections are presented.

Comparing moment-rotation diagrams for both

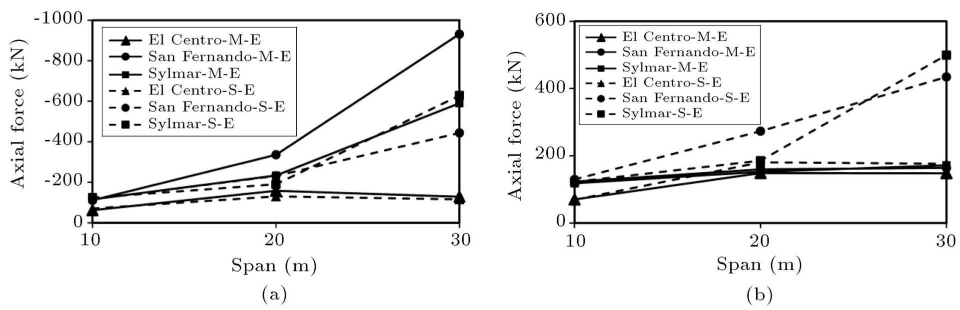


**Figure 5.** Axial force vs. span for bridges supported on elastomeric bearings: (a) and (b) Compressive, and tensile axial forces, respectively.

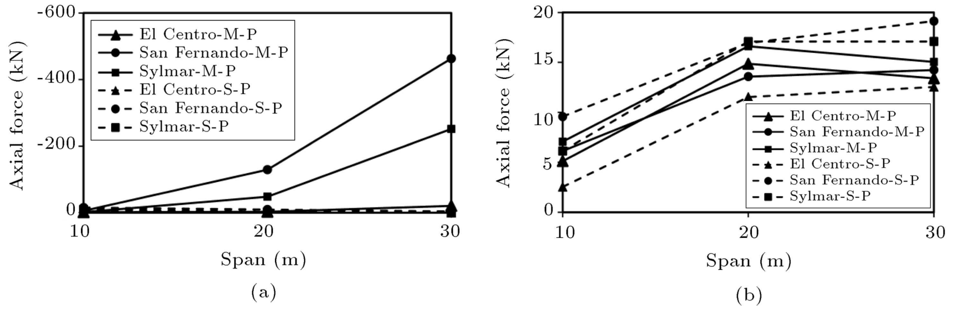


**Figure 6.** Axial force vs. span for bridges supported on pinned bearings: (a) and (b) Compressive, and tensile axial forces, respectively.

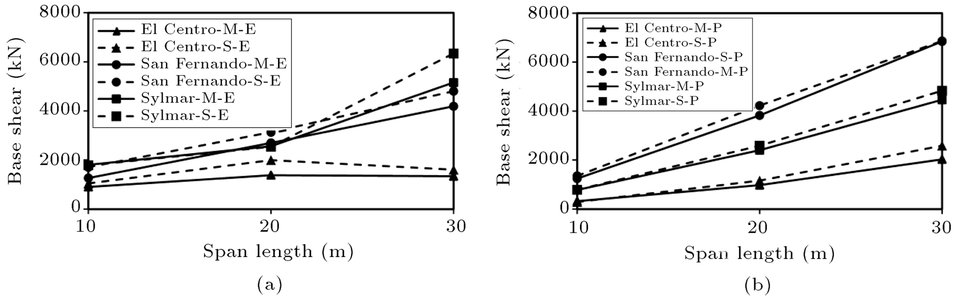


**Figure 7.** Base shear vs. span: (a) Elastomeric bearing; and (b) pinned bearing.

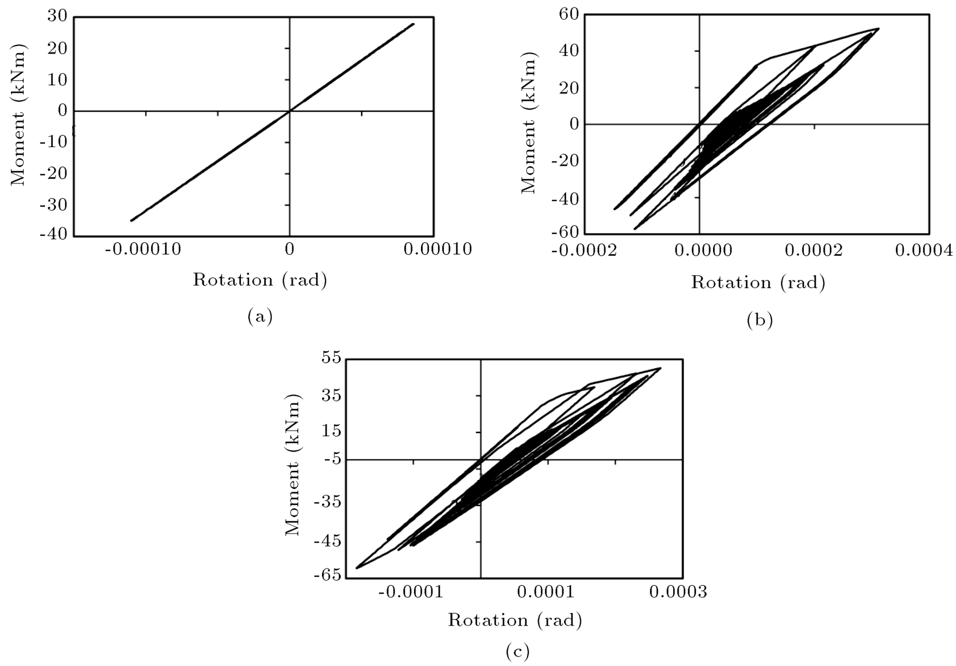


**Figure 8.** Moment rotation of hinge H1, 10 m span bridge with elastomeric bearings: (a) El Centro; (b) San Fernando; and (c) Sylmar ground motions.

bearing support conditions, it can be concluded that the forces induced in the corner EDs are greater when they are supported on elastomeric bearings. Therefore, for brevity, the results for pinned bearings and middle diaphragms are not shown. The moment-rotation curves of the corner diaphragm beams subjected to real ground motions are shown in Figures 8-10 for the three span lengths. It is seen that for the 10 m span length, the amount of hinge plastification is negligible. However, the inelastic deformations increase with increasing span lengths, and therefore the amount of energy that needs to be dissipated by these members increases. Also, stable hysteresis loops due to close-spaced stirrups are observed. It should be noted that since shear and torsion is considered elastically in the fiber hinges, no shear failure is monitored in current

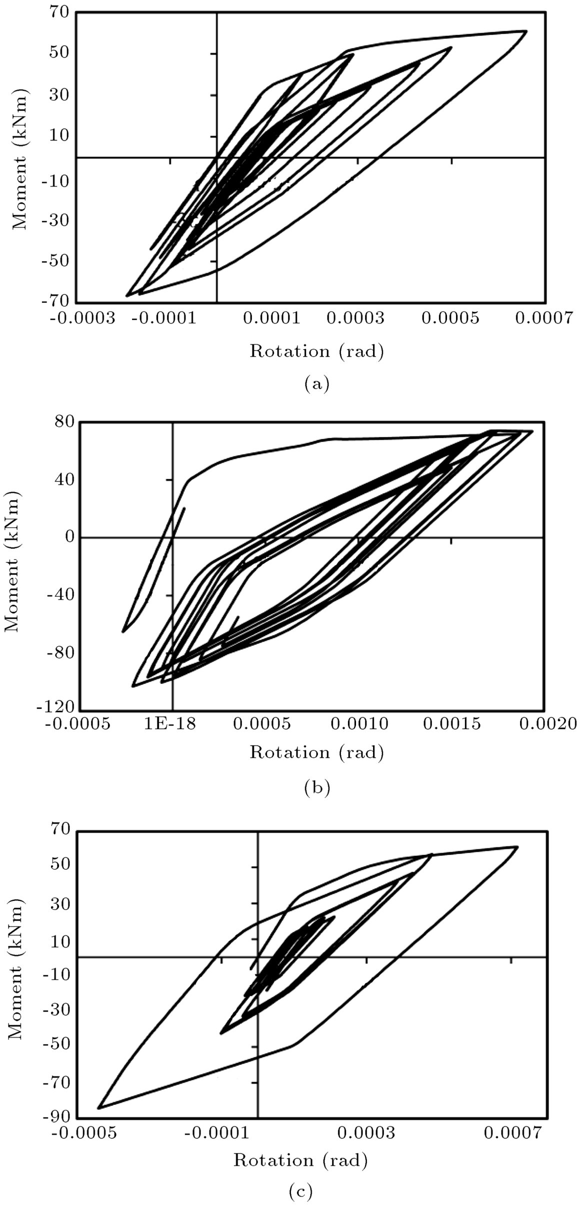


**Figure 9.** Moment rotation of hinge H1, 20 m span bridge with elastomeric bearings: (a) El Centro; (b) San Fernando; and (c) Sylmar ground motions.

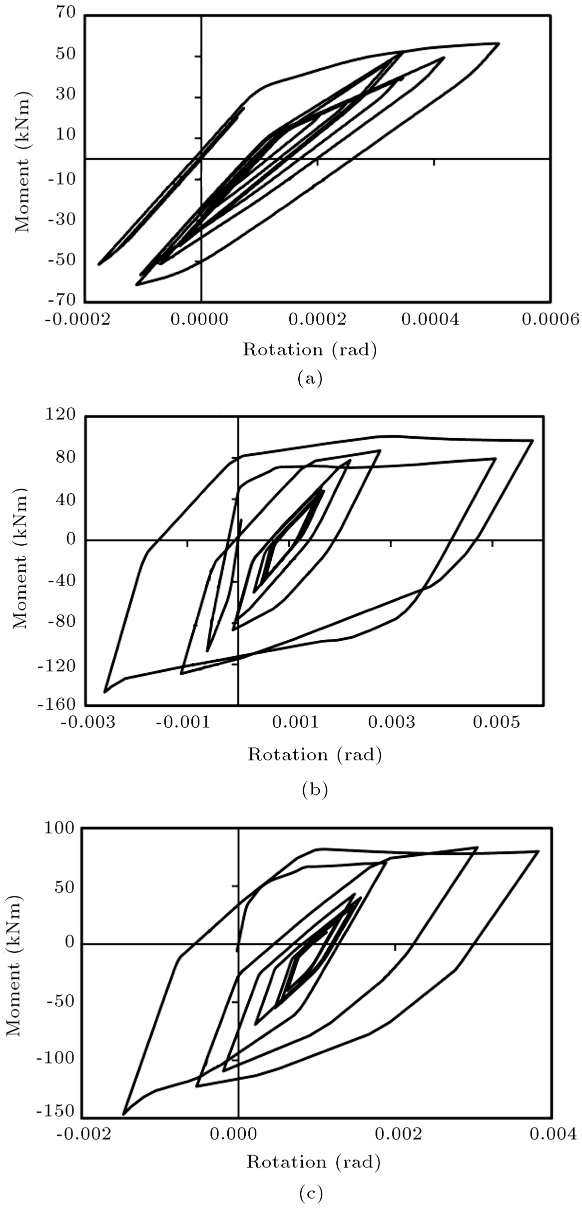


**Figure 10.** Moment rotation of hinge H1, 30 m span bridge with elastomeric bearings: (a) El Centro; (b) San Fernando; and (c) Sylmar ground motions.

analyses. This assumption is valid since in current analyses the seismic shear demands are less than the ultimate shear capacities of the EDs. However in some cases where the EDs act as deep beams (such as in the case of full-depth diaphragms) shear failure is more likely to happen prior to flexural failure, if not designed properly.

Figure 11 shows the variation of maximum hysteretic energy dissipation versus span length in corner and middle ED beams for both types of supports. The results for elastomeric bearings are shown in full lines, while the pinned bearing results are in dashed lines. Except for San Fernando ground motion, the amount of hysteretic energy dissipation by the EDs is more significant in bridges with elastomeric bearings than in the case of pinned bearings. It is also concluded that for both support conditions the contribution of corner diaphragm beams to energy dissipation is greater than the middle diaphragm beams. Furthermore, the corner diaphragm beams, due to the presence of higher axial forces, are more critical from design perspective than the middle ones.

Next, the results for corner ED beams will be compared to forces introduced by AASHTO's recommended design force ($S \times A$ ). Figures 12 and 13 show the variation of maximum moment and axial force induced in the corner ED beams, versus span length under different earthquake loadings and the $S \times A$

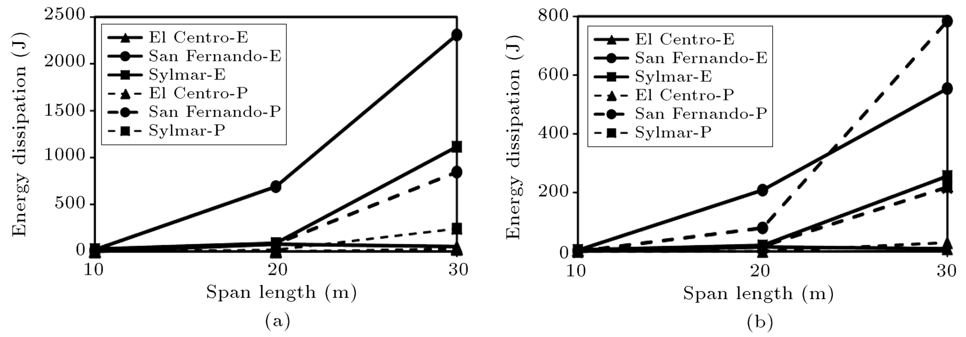


**Figure 11.** Dissipated energy vs. span length: (a) and (b) Corner, and middle diaphragm beams, respectively.

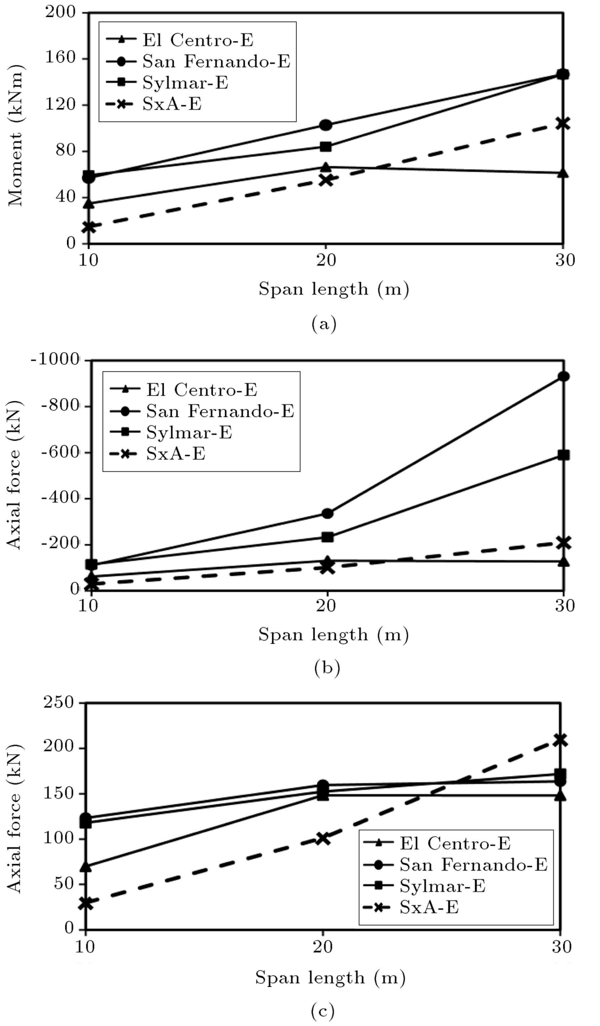


**Figure 12.** Comparison between maximum actions for different earthquake loadings, and $S \times A$ method for bridge supported on elastomeric bearing: (a) Flexural moments; (b) and (c) compressive, and tensile axial forces, respectively.

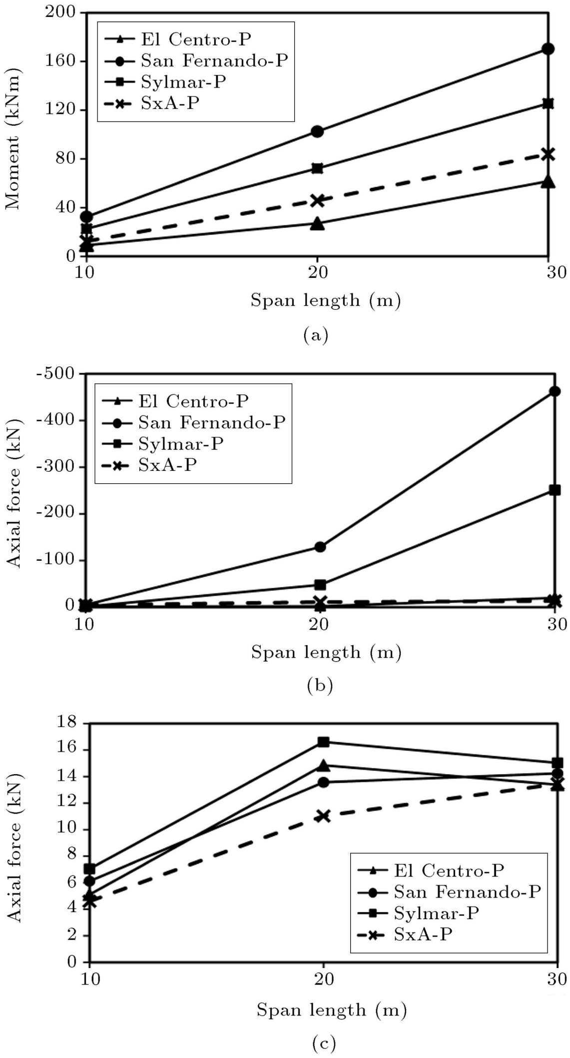


**Figure 13.** Comparison between maximum actions for different earthquake loadings, and $S \times A$ method for bridge supported on pinned bearing: (a) Flexural moments; (b) and (c) compressive, and tensile axial forces, respectively.

method. It can be concluded that the flexural moments and compressive axial forces increase with increasing span lengths for both bearing types, except for the El Centro ground motion with elastomeric bearings in which it decreases due to the frequency content of El Centro ground motion. The tensile axial forces increase with increasing span length for spans up to 20 m and decrease for span of 30 m. This decrease is expected since the compressive axial forces increase. Figure 12 shows that for bridges supported on elastomeric bear-

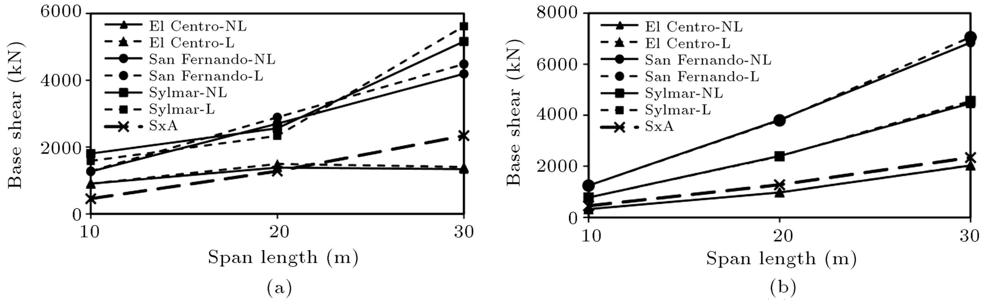


**Figure 14.** Maximum base shear vs. span length: (a) Elastomeric bearing; and (b) pinned bearing.

ings, flexural moments and compressive axial forces generated by El Centro ground motion are in good agreement with those generated by the $S \times A$ method for spans up to 20 m. For San Fernando and Sylmar ground motions, the introduced forces are higher. Tensile axial forces generated by the $S \times A$ method are generally lower than those generated by considered earthquakes, except for span of 30 m. Figure 13 shows that for bridges supported on pinned bearings, generated forces by El Centro ground motion are in good agreement with those generated by the $S \times A$ method for span of 10 m. For all other spans, flexural moments generated by the $S \times A$ method are higher compared to those generated by El Centro ground motion and lower compared to those generated by San Fernando and Sylmar ground motions. Comparing the results of different earthquakes with the $S \times A$ method, it can be concluded that the results of the $S \times A$ method are unsafe in most cases.

Figure 14 shows that for both support conditions, base shear increases with increasing span. However, for the El Centro ground motion the base shear has decreased when span changes from 20 to 30 m. This decrease again is related to the frequency content of the El Centro ground motion. Therefore the frequency content of the earthquake has to be taken into account before any general conclusion is made. In Figure 14, the base shear in both linear and nonlinear time history analyses and for both support conditions are compared to evaluate the effect of nonlinear behavior of end diaphragm beams on total structural response. In the legend of figures, "L" and "NL" denote linear and nonlinear time history analysis, respectively. The base shear has decreased in the nonlinear analyses for larger spans when subjected to MCE ground motions (San Fernando and Sylmar). For instance, the base shear for the 30 m span in the nonlinear time history analyses, due to Sylmar ground motion, has decreased about 9% compared to the linear case. This reduction is about 7% for San Fernando ground motion. It should be noted that for the pinned bearings the effect of nonlinearity is negligible, even for larger spans. The reduction in base shear reduces the forces transmitted to the substructure and yields a more economical design. Furthermore, for larger spans, the base shear is less when the bridge is supported on elastomeric bearings.

From a design perspective, the base shear generated by El Centro ground motion agrees well with the $S \times A$ method for bridges supported on pinned bearings. However, the $S \times A$ method yields unsafe results for San Fernando and Sylmar ground motions for both support conditions and all spans. For bridges supported on elastomeric bearings the $S \times A$ method yields unsafe results compared to El Centro for span of 10 m, and agrees well for span of 20 m and is conservative for span of 30 m. Based on the average results presented herein, a magnification factor of 2.05 and 1.85 is obtained for elastomeric and pinned bearings, respectively. Thus, for practical purposes, it is recommended that AASHTO's prescribed design force for the end diaphragms be increased to $2.05 \times S \times A$ times the tributary weight for all single-span bridges and for all support types. This recommendation agrees well with previously obtained results for steel bridges [19].

Furthermore, to investigate AASHTO article 4.6.2.8.2, the moment-rotation curves of the hinges in corner ED under scaled ground motions were obtained. Only the 30 m span results are shown in Figure 15. The figure shows that the end diaphragm beams remain elastic and behave linearly. Since dimensions considered in this study for end diaphragm sections are the minimum dimensions used in practice, it can be concluded that for single span bridges AASHTO article 4.6.2.8.2 is satisfied and the end diaphragms remain elastic under design earthquakes with various frequency contents.

## 6. Conclusions

A parametric study was conducted on a three-dimensional finite element model of single-span slab-girder bridges to explore the behavior of concrete end diaphragms under transverse earthquake forces. Two different support conditions, namely, elastomeric and pinned bearings were considered. Span lengths of 10, 20 and 30 m were considered to include the usual range

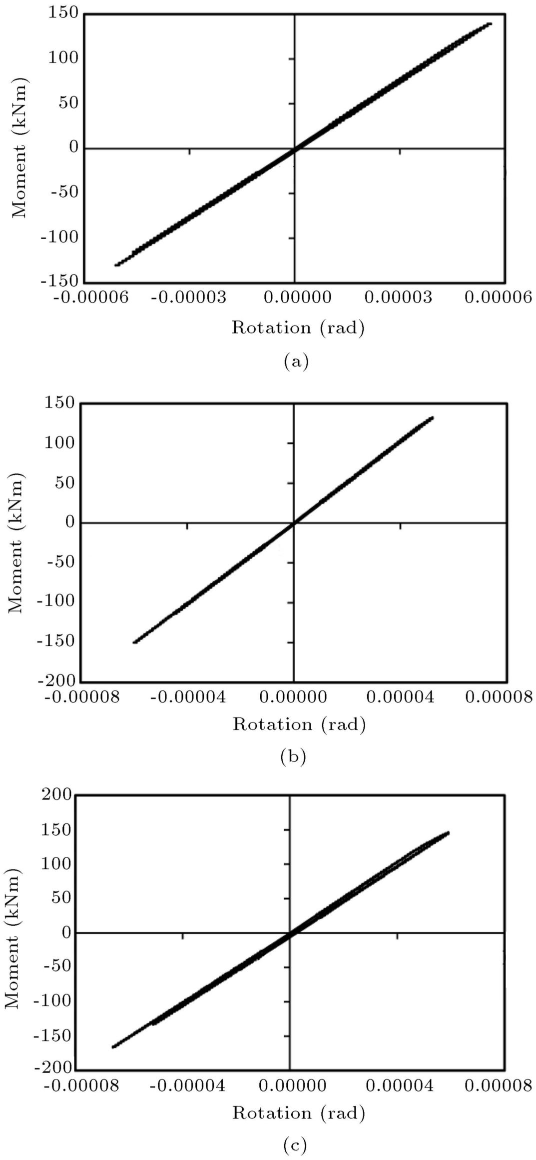


**Figure 15.** Moment rotation of hinge H1, 30 m span bridge with elastomeric bearings: (a) El Centro; (b) San Fernando; and (c) Sylmar ground motions.

of slab-girder bridges. AASHTO design provisions were reviewed and compared to numerical results obtained from analyses. The following conclusions can be drawn from this study:

1. It was found that the end connections of diaphragm beams (being simple or moment connection) are of great importance in the transverse seismic behavior of the bridge. In the simply connected ED beams, only axial forces exist, whereas for the moment connection flexural moments are also present. Therefore, in the latter case, the seismic energy is absorbed via the hysteresis of the beam elements and in the former elastic behavior is expected.
2. Both compressive and tensile axial forces can be present in the ED beams. These members are not commonly designed for tensile forces and the design strategy needs further investigation. The axial forces in the corner ED beams are higher than the middle beams.
3. For bridges supported on elastomeric bearings, forces induced in the ED beams are larger compared to those supported on pinned bearings. It is also concluded that with increasing span length, the flexural moments increase, resulting in an increased hysteretic energy dissipation and base shear reduction. Consequently, the forces transmitted to the substructure are reduced.
4. The results of time history analyses were compared to prescribed seismic design force by AASHTO articles 3.10.9.1 and 4.7.4.2 ($S \times A$ method). It was concluded that in general the $S \times A$ method yields unsafe results for strong earthquakes. Currently, the $R$ factor in seismic design of EDs is equal to one, meaning no major hysteretic energy dissipation is expected. The nonlinear time history analyses prove that this appears to be true. Hence, for single-span bridges with all types of bearing supports, it is recommended that the prescribed design force be increased to $2.05 \times S \times A$ times the tributary weight to salvage the superstructure in major earthquakes.
5. According to AASHTO article 4.6.2.8.2, the end diaphragms being part of the seismic load path, have to remain elastic under the prescribed design seismic forces, regardless of the type of bearings used. Nonlinear time history analyses using scaled ground motions with different frequency contents proved that for single-span slab-girder bridges with common ED beam sizes this article is satisfied and the end diaphragms remain elastic.

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## Biographies

**Himan Hojat Jalali** obtained his Bachelor Degree in Civil Engineering from the Iran University of Science and Technology, Tehran, Iran, in 2005, with honor. He pursued his higher education at Sharif University of Technology, Tehran, Iran, where he received his MSc degree in Structural Engineering in 2008. He is currently pursuing his studies as a PhD Candidate in Structural and Earthquake Engineering at Sharif University of Technology, Tehran, Iran. His research focuses on the seismic design and nonlinear analysis of structures.

**Shervin Maleki** obtained his Bachelor and Master's Degrees from the University of Texas at Arlington with honor. He then pursued his PhD degree in New Mexico State University at Las Cruces and finished it in 1988. He has many years of experience in structural design both in the US and Iran. He has been a faculty member at Bradley University in Illinois and Sharif University of Technology. He has authored and coauthored over forty technical papers and has authored two books and a chapter in Handbook of International Bridge Engineering. His research area is mainly focused on seismic design of bridges and buildings. He has invented several metallic dampers for buildings and bridges.