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Earthquake performance of the two approach viaducts of the bosphorus suspension bridge

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Abstract. The main purpose of this paper is to determine the dynamic characteristics and the structural stability of the two approach viaducts of the Bosphorus Suspension Bridge under the expected stresses that would be caused during earthquake conditions. The Ortakoy and the Beylerbeyi approach viaducts constitute the side spans of the bridge at two locations. The bridge's main span over the Bosphorus is suspended, whereas they are supported at the base at either end. For the numerical investigation of the viaducts, 3-D computational structural finite element-FE models were developed. Their natural frequencies and the corresponding mode shapes were obtained, analyzed, presented and compared. The performances of the viaducts, under earthquake conditions, were studied considering the P-Delta effects implementing the push-over (POA) and the non-linear time-history analyses (NTHA). For the NTHA, three earthquake ground motions were generated depending on the location of the bridge. Seismic performances of the viaducts were determined in accordance with the requirements of the Turkish Seismic Code for the Earthquake Design of Railways Bridges (TSC-R/2008) and those of Caltrans (CALTRANS-2001) given for Seismic Design of Steel Bridges, separately. Furthermore, the investigation was extended for evaluating the possible need for retrofitting in the future. After the analysis of the resultant data, a retrofit recommendation for the viaducts was presented.

Keywords: suspension bridge; approach viaducts; seismic performance; non-linear time-history; push-over; seismic retrofit

1. Introduction

In developing countries, the need for transportation structures has continuously increased. The most critical component in a transportation system can be considered to be the bridge structures.

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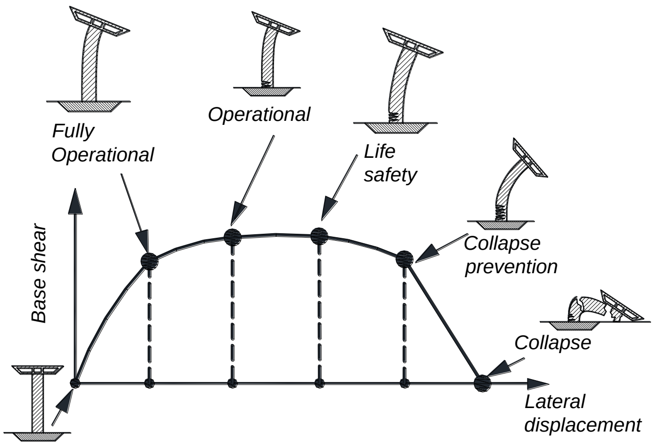


Fig. 1 Capacity curve for a bridge

Numerous bridge failures in the catastrophic earthquakes, such as Loma Prieta (US, 1989), Northridge (US, 1994) and Kocaeli (Turkey, 1999) earthquakes have resulted in growing interest in the seismic safety evaluation of the existing bridges along with their performance-based assessment. Performance-based earthquake engineering aims to determine the seismic behavior of structures and to check whether the seismic performance levels are acceptable TRB (2013). Various structural performance levels of a bridge are presented in Fig. 1 where the capacity curve is plotted as the base shear versus the displacement at a monitoring point.

The non-linear behavior of bridges results from non-linear deformations of cross sections and structural elements due to non-linearity in material stress-strain relations and geometry, called P-Δ effects (Aviram *et al*. 2008). Various methods have been developed for the application of non-linear analyses. One of them is the non-linear static analysis, often called the push-over analysis. Many studies related to this analysis have been conducted by Freeman *et al*. (1975), Krawinkler (1996), Krawinkler and Seneviretna (1998), Chopra and Goel (2000), Chopra and Goel (2001) and Fajfar (2000). Basically, the analysis aims at obtaining the non-linear behavior of structural systems under predetermined lateral load variation, which is assumed to represent earthquake effects with an inherent gradual increase. ATC (1996), FEMA (2000), SEAOC (1995), EC8 (2005) and TSC (2007) also recommend the push-over analysis for seismic evaluation of the structures. However, these studies and the guidelines focus on the seismic assessment of buildings rather than bridges. Similar concepts and procedures can also be employed in the performance-based seismic evaluation of bridges. Aydinoglu (2003) suggested a comprehensive push-over method called Incremental Response Spectrum Analysis (IRSA), which additionally considers the effect of the multi-mode response. Aydinoglu and Onem (2007) applied the IRSA method to a set of bridges in order to evaluate their non-linear behavior and compared their results with those obtained from the non-linear time-history analysis. Isakovic *et al.* (2008), and Isakovicet and Fischinger (2011) applied various versions of the push-over analysis to the bridges, tested on shaking tables. All performed procedures of the push-over analysis, based on the adaptive multi-mode procedure, were found to be very effective in regard to the estimation of the displacement of the bridge deck when subjected to severe earthquakes. Casarottiet and Pinho (2007) proposed an innovative assessment procedure for bridges with flexible and rigid superstructures, which could accurately estimate the inelastic quantities, such as displacement and moment demands. Pinho *et al*. (2009)

investigated the accuracy of the pushover-based methods in literatures utilizing a set of continuous bridges subjected to highly intensive earthquake motion. The study concluded that displacement was estimated well in all methods. Barron (1999) and Shinozuka *et al*. (2000) implemented the non-linear static method to determine the fragility curves for bridges, where the capacity spectrum method was often used to estimate the displacement demand of the bridges. Casarottiet *et al*. (2009) proposed practice-oriented procedure for the non-linear static analysis of multiple-degree-of-freedom (MDOF) structural system of bridges. This aim was achieved by using the existing expressions of single-degree-of-freedom structural system for MDOF system. They utilized a large set of bridges and compared the results from the proposed approach with those from the time-history analysis to verify the procedure. Thus, they presented the most suitable spectral reduction plan to be easily utilized in the performance assessment of bridge structures. Recently new approach of the probabilistic-based earthquake performance assessment of R/C bridges was proposed by Monteiro *et al*. (2016b). For this objective, considering two uncertainty models, which are local and global uncertainty models, push-over-based probabilities of the case study of seven R/C bridges were calculated and compared to those from non-liner dynamic analysis that was conducted with a number of real earthquake records. They indicated that non-linear static analysis has the ability to conservatively predict probability of seismic demand of reinforced concrete bridges compared to the non-linear dynamic analysis. Even though the proposed non-linear static methods facilitate the determination of the non-linear earthquake demands in structures, the non-linear time-history analysis has still been the most rigorous approach, which in turn is considered as a reference method as it takes both the inelastic response properties and the dynamic effects in the structures into account. (Chopra 2007, Kappos *et al*. 2012) stated that the major challenge in this procedure was the selection and scaling of earthquake ground motion records to be used. Related to computational model considerations for the push-over analysis, Tang *et al*. (2014) implemented the method to multi-scale and fiber model of steel bridge.

Limited studies have been carried out for the seismic performance evaluation of the critical bridges in Turkey, the Bosphorus Bridge and the Fatih Sultan Mehmet Bridge, since 1999 Kocaeli earthquake. Apaydin (2002) performed a detailed research for the evaluation of the dynamic characteristics of the Fatih Sultan Mehmet Bridge using experimental and analytical methods. Kosar (2003) evaluated the dynamic characteristics of the Bosphorus Suspension Bridge using Ambient Vibration Test (AVT). Experimentally obtained modal properties of the bridge were compared with those from the numerical analysis adopting detailed 3-D finite element model of the bridge without the approach viaducts. Furthermore, recently new comprehensive investigation on the earthquake performance of the existing suspension bridges in Turkey, the Bosphorus Bridge and the Fatih Sultan Mehmet Bridge, was carried out by Apaydin (2010). Utilizing sophisticated 3-D finite element-FE model of the bridge with the approach viaducts, geometric non-linear earthquake analysis and free vibration analysis of the Bosphorus Bridge were performed in that study. Based on the outcomes from these studies, the dynamic behavior of the Bosphorus Bridge was determined to be considerably affected by including and excluding the approach viaducts in the 3-D FE model of the bridge. To illustrate, the bridge with the approach viaducts has higher modal properties of mode shapes amplitudes and periods as well as displacements at the critical points than the absence of the viaduct. In addition, Kosar (2003) and Apaydin (2010) showed that plastic range was not taken into consideration for suspended or hanged components of the Bosphorus Bridge. They indirectly recommended that the approach viaducts with no suspenders should be investigated separately to reliably determine the earthquake performance level of them.

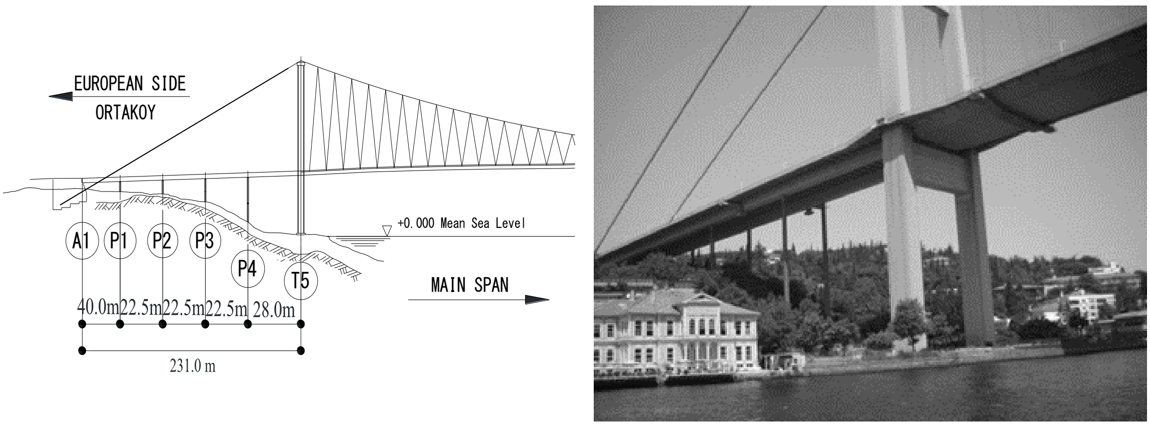
Considering the differences between the FE model of the bridge with and without the approach viaducts as significant indicator to make elaborate investigation on the viaducts, the present study aims at determining the dynamic response characteristics and the structural earthquake performance of the approach viaducts of the Bosphorus Bridge in Istanbul. For this objective, free vibration analysis of the approach viaducts was carried out, and the corresponding mode shapes were obtained. In the consideration of these dynamic modal parameters, seismic performance assessments of the viaducts were determined using the push-over (POA) and the non-linear time-history analyses (NTHA). Performance evaluation of the approach viaducts was separately performed in compliance with the requirements of the Turkish Seismic Code for Railways Bridges (2008) and those of Caltrans Seismic Design of Steel Bridges (2001). The outcomes from the non-linear analyses demonstrated that the approach viaducts of the bridge could be damageable under earthquake loading and that the damage only concentrated in the columns of the viaducts, which means no damage on the superstructure of the viaducts. Such behavior of them was based on supporting of the viaducts at the base providing high rigidity when compared to suspended system. Considering these consequences, approach spans of long-span bridges which were supported at the base should be considered as a separated structure from bridge in order to reliably estimate damage level. Besides, the rigidity of approach span should be specified and should be adjusted according to that of the main span of long-span bridges. Based on the results of the seismic assessment, the most suitable recommendation was also given for retrofitting of the viaducts by taking the current operational condition of the bridge and the bridge's traffic into account.

2. Description of the approach viaducts

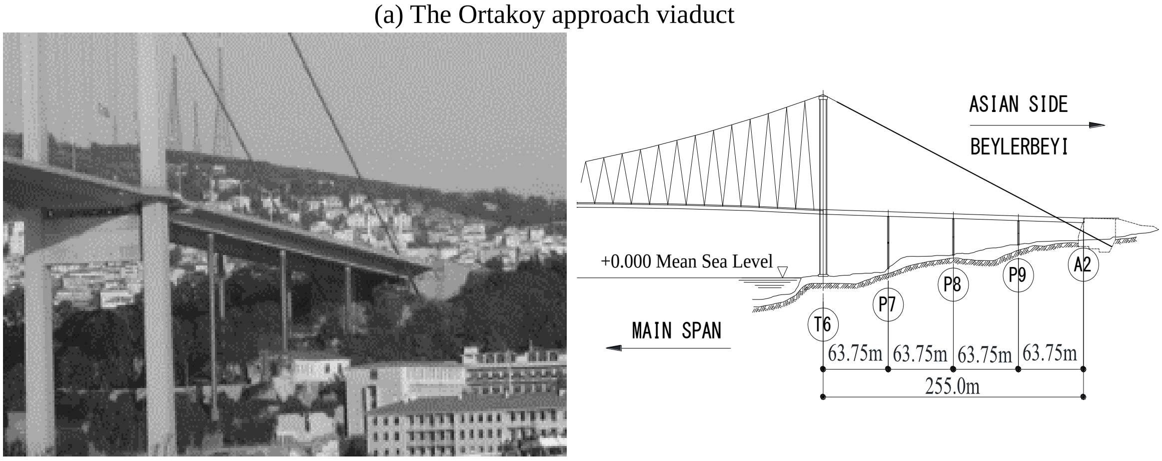
The Bosphorus Suspension Bridge has two approach viaducts. As shown in Figs. 2(a)-2(b), the one referred to as Ortakoy is located at the European side. It has a length of 231 m and five spans. The other one, called Beylerbeyi, is located at the Asian side. It has a length of 255 m and four spans. The viaducts have no hanger elements, and they have columns of various heights and were supported at the base. The approach viaducts were separated by the simple support connection at the lower portal beam level instead of the expansion joint. The viaducts were also simply supported on the land side close to the anchorages. Thus, the bridge features a deck, displaying discontinuity at the tower supports, which separates the approach viaducts from the main span. Consequently, no coupling between the main part and the approach viaducts of the bridge can be considered; accordingly, the FE model of each approach viaduct was developed separately.

A live load of H30-S24 was considered on the truck lane, which is the equivalent distributed lane load, 9.0 kN/m, according to the Technical Specifications for Highway Bridges (1982), whereas for the pedestrian lane, 3.6 kN/m equivalent distributed load was regarded. In the dynamic analysis, 9.0 kN/m live load was assumed as a uniformly distributed load for the six lanes of the viaducts. The one-thirds of H30-S24, 3.0 kN/m, was regarded as a uniformly distributed car lane load for the other two lanes (Freeman 1968). The deck of the viaducts was made up two steel box-girders spanning along the axis of the viaducts. The box-girders were connected to each other by the steel cross-beams having cantilever lengths of 3.00 m in lateral direction. The cross-section of the viaducts' deck is shown in Fig. 3(a). The two steel box-girders, supported by the columns, were considered to be the main structural elements, i.e. the backbone of the viaducts. Cross sectional dimensions of the girders, stiffened with the steel plate elements in the longitudinal direction, were 3.00 m×3.87 m as shown in Fig. 3(b). The cross-beams, connected to the girders and stretching out laterally, are also the other main elements of the structural system. They were also included in the model. These beams have a cross section of I-profile as shown in Fig. 3(c). They also comprised of moment-resisting connections to the girders (Bas 2011). The column of the viaducts has circular sections with varying wall thickness and heights depending on ground level. The support conditions of the columns of the Ortakoy and the Beylerbeyi approach viaducts at the bottom and the top ends are given in Figs. 4(a)-(b), respectively. In addition to the support conditions of the columns for each viaduct, the corresponding support conditions of the viaducts at their two end supports, anchorage and tower (expansion joint), were also taken into account as shown in Fig. 4.

According to the project drawings of the viaducts, the abutment at the land side was provided with additional part to the stiff anchorage block. Thus, the weight of anchorage block was directly increased with this arrangement. This abutment with less height compared to usual one was also supporter for the precast concrete viaducts enabling to link the bridge to the center of the city. In the project of the approach viaducts, all details were given only for the anchorage. Certain specifications and the dimensions of the abutment were not given in detail. Therefore, the abutments could not be considered in the FE model of the viaducts. However, the integration of the abutment with the anchorage block was recommended to be studied particularly.



(a) The Ortakoy approach viaduct



(b) The Beylerbeyi approach viaduct

Fig. 2 General arrangement and view from the viaducts

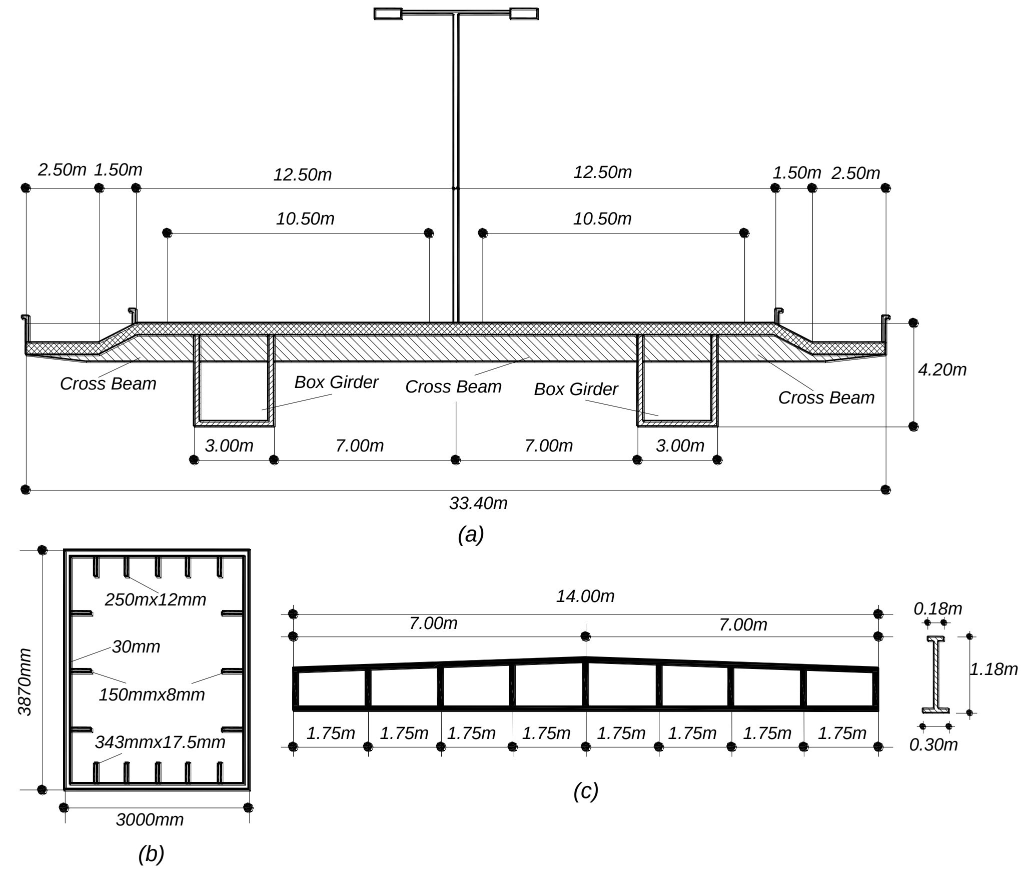


Fig. 3 General layout of (a) the deck section, (b) the box-girders and (c) the cross-beam

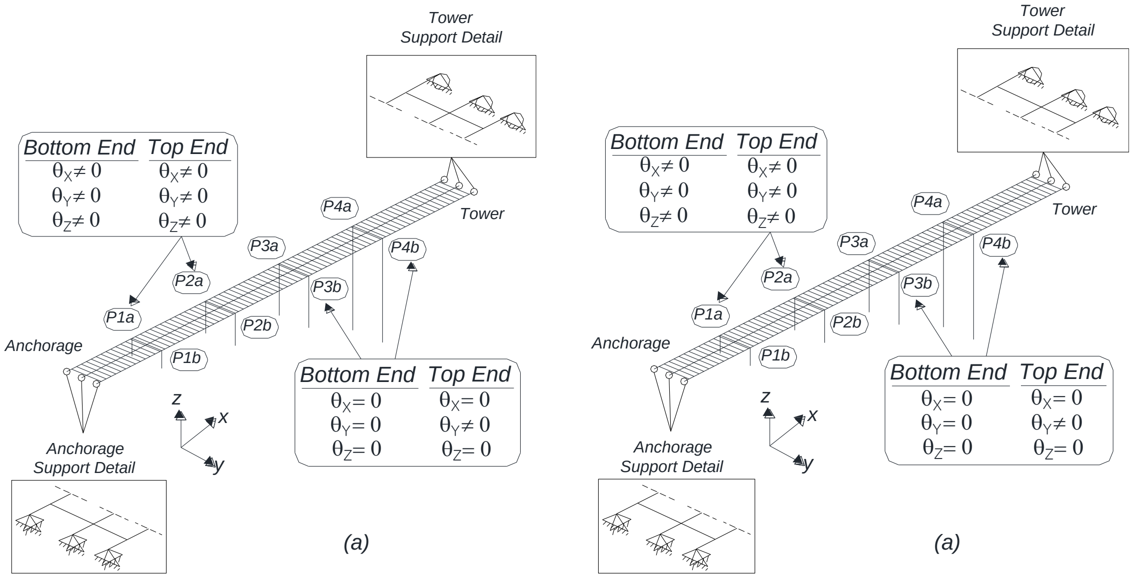


Fig. 4 Support conditions of (a) the Ortakoy and (b) the Beylerbeyi approach viaducts

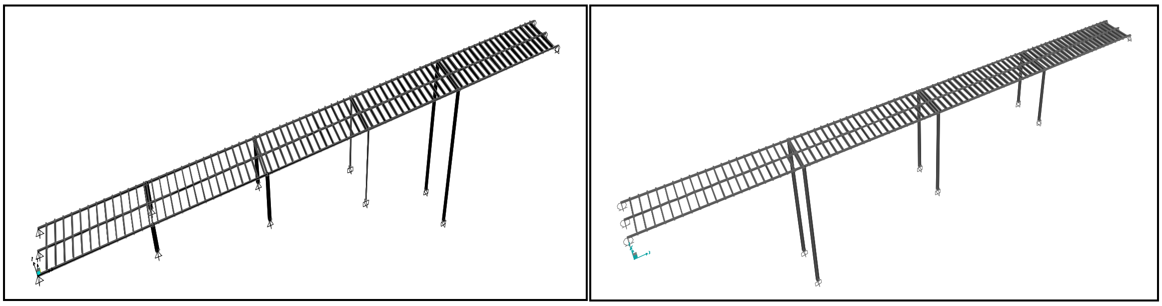
3. Free vibration analysis of the viaducts

3.1 Finite element model-FE

For performing a more accurate dynamic analysis, the finite element models of the viaducts were developed using the technical specifications of the viaducts. The analysis was accomplished by SAP2000 (CSI, 2011). All structural members of the viaducts such as the box-girders, the cross-beams and the circular steel columns were considered as frame element in the model. Considering these assumptions, the finite element models are presented in Fig. 5 for the Ortakoy and the Beylerbeyi approach viaducts, respectively.

3.2 Structural dynamic characteristics of the approach viaducts

Free vibration and mode shape analysis of the viaducts were performed using SAP2000, and the effective mode shapes of the viaducts were determined in the lateral direction. In spite of the high lateral stiffness value of the deck, vibration occurs mainly due to bending and deformation of the columns. Results of the dynamic parameters and amplitudes of the fundamental mode shapes of the viaducts are given in Table 1. Additionally, the fundamental mode shapes of the viaducts are shown schematically in Fig. 6. For the push-over analysis (POA) of the viaducts, only the lateral mode shapes were taken into account. As seen in Fig. 6, the monitoring points of the viaducts,



(a) (b)

Fig. 5 Finite element model of (a) the Ortakoy approach viaduct and (b) the Beylerbeyi approach viaduct

Table 1 Results of the free vibration analysis of the viaducts

Viaducts	Amplitudes of the Effective Mode Shapes		Period T_{1y}, sec	Freq. cyc/sec	Circular Freq. rad/sec	Effective (My1) / Total mass (M)	Modal Participation Factor Γy1
Ortakoy	$\Phi_{11}=$	0.26	1.27	0.79	4.95	0.87	1.27
	$\Phi_{21}=$	0.75					
	$\Phi_{31}=$	1.00					
	$\Phi_{41}=$	0.79					
Beylerbeyi	$\Phi_{71}=$	0.84	1.55	0.65	4.05	0.98	1.23
	$\Phi_{81}=$	1.00					
	$\Phi_{91}=$	0.46					

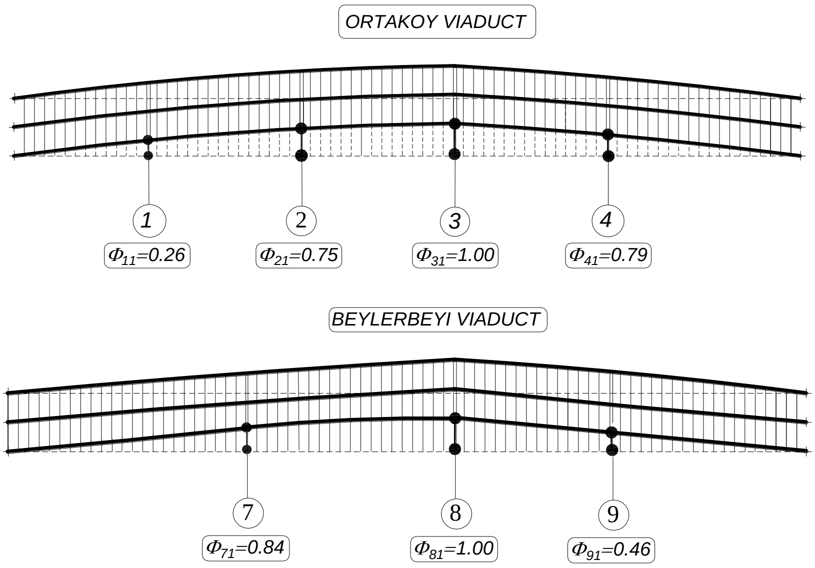


Fig. 6 The fundamental mode shape of the viaducts

especially the ones necessary for the POA, were selected as the support 3 and as the support 8 for the Ortakoy and the Beylerbeyi viaducts, respectively. These supports constitute the mass center of the deck of the viaducts. Geometric non-linearity (P-Δ), with its significant effect on the behavior of the slender columns, resulting in a lateral displacement due to the size of axial columns, was also taken into account in all analyses.

4. Earthquake performance of the approach viaducts

4.1 Simulated earthquake ground motion records

For the NTHA, the TSC-R/2008 proposed that three or seven earthquake records should be used in compliance with the requirements given in the code. Seven or more number of earthquakes were proposed by Casarottiet *et al*. (2009) to be used for reliable seismic evaluation of bridge structures. For sampling- and probabilistic-based earthquake performance evaluation of bridge structures, Monteiro (2016) and Monteiro *et al*. (2016a) considered a set of ten records obtained from real historical earthquakes for the non-linear dynamic analysis and these records were used as input variable for failure probability of bridge structures. Thus, they exhibited that record-to-record variability has significant effect on the increase in failure probability of bridge structures. However, three simulated earthquake records were decided to be used in the NTHA of the viaducts so that the results of the NTHA and how to generate spectrum-compatible earthquake records could be easily understood especially from practicing engineers/practitioners. Original earthquake records of Loma Prieta (NW, 1989), Erzincan (NE, 1992) and Kocaeli (NE, 1999), obtained from the Pacific Earthquake Engineering (PEER) strong motion data base, were considered and their simulated versions were generated making sure that their spectra were compatible with those of the TSC-R/2008. The requirements were fulfilled through Seismo-Match (2010) software that

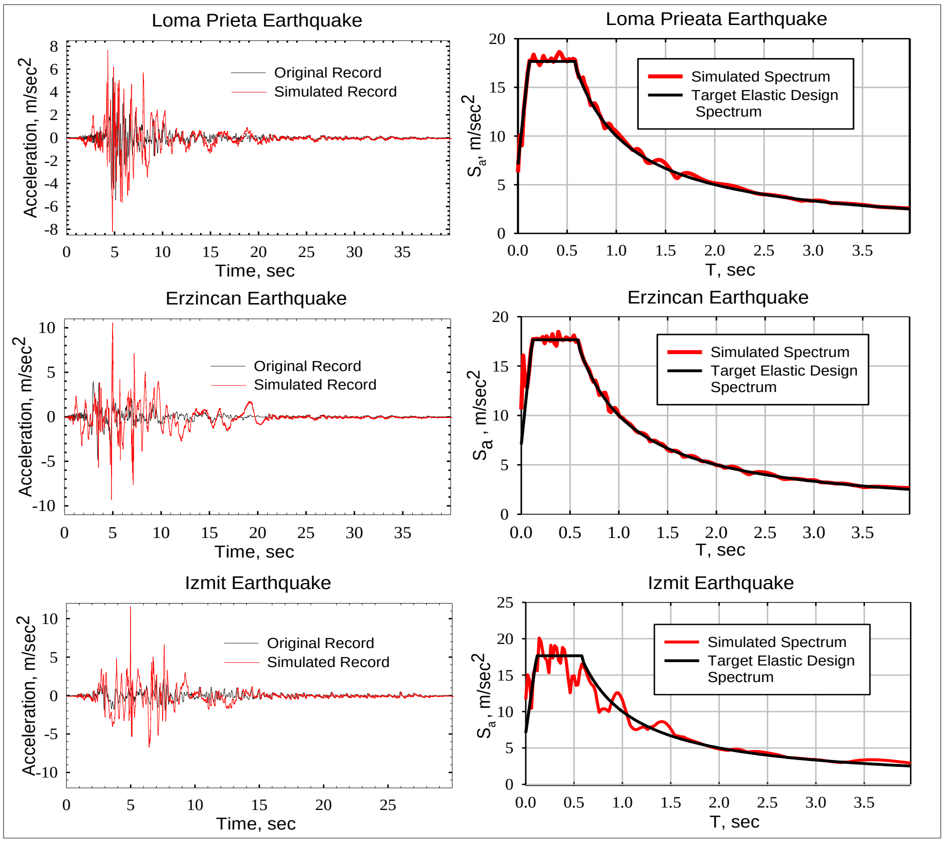


Fig. 7 Simulated earthquake strong ground motion records and their spectra

considers the proposed wavelets algorithm (Abrahamson 1992, Hancock *et al*. 2006). After simulation of the earthquake records, the fundamental data processing of baseline correction and detrending was also employed to obtain precise results from the analyses since the raw earthquake records could contain unwanted problems related to noise and accelerometer deployment etc. Besides, any specific provisions for data processing were not given in the TSC-R/2008. In Fig. 7, the original and the simulated earthquake records are shown with their elastic design spectra.

4.2 Push-over analysis (POA)

The earthquake performances of the viaducts were determined through the fundamental mode-based POA. The push-over analysis was performed in accordance with the requirements described in the TSC-R/2008. The TSC-R/2008 has certain similar provisions of the N2 method proposed by Fajfar (2000). Therefore, the specifications of the N2 method were considered in the push-over analysis. The POA was implemented based on a plastic hinge assumption where plastic deformations are assumed to be concentrated. The possible plastic hinges were assumed to be located within the columns only. Plastic hinges at the end of the columns were defined taking axial force into account. Since the deck has a high bending stiffness in a lateral direction, no plastic

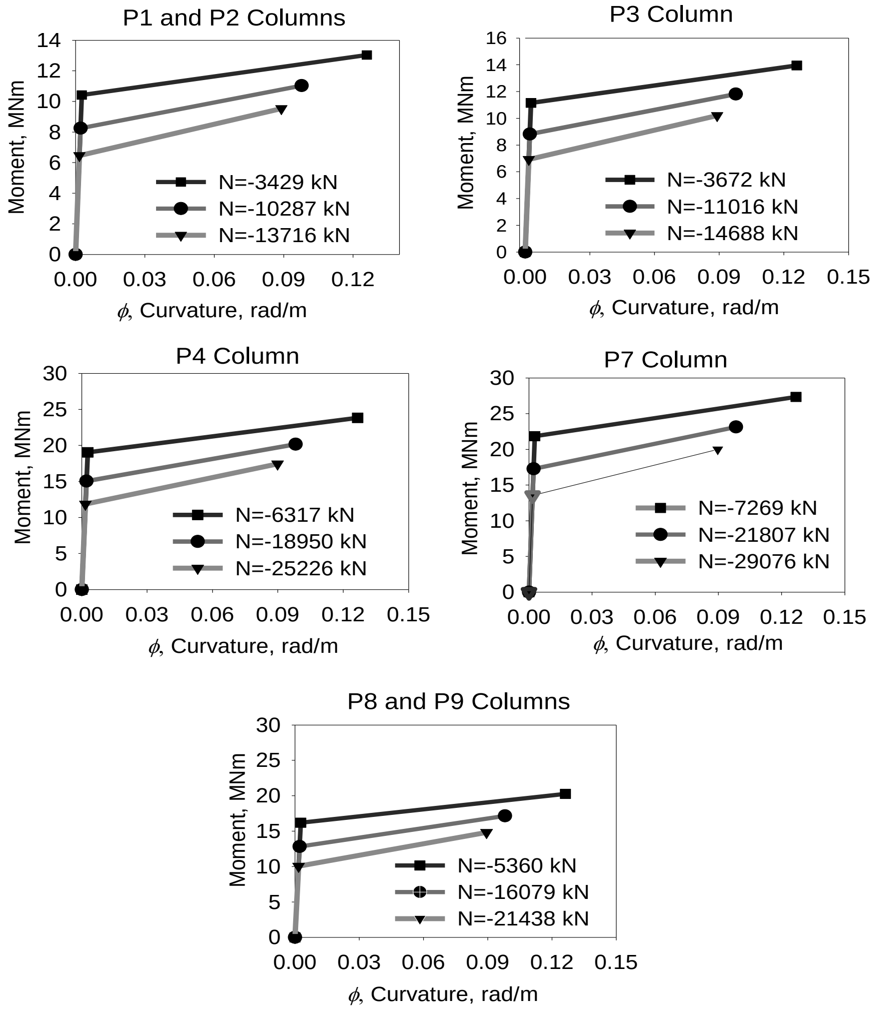


Fig. 8 Moment-curvature relations of the columns

deformation is expected in the box-girders and in the cross-beams; therefore, no plastic hinges were assigned to them. As for the top and bottom sections of the columns, where inelastic deformations were expected and plastic hinges were assigned. The length of the plastic hinge (L_p) was assumed to be half of the cross-sectional diameter of the columns as defined in the TSC (2007). In Fig. 8, the idealized moment-curvature relations of the columns are shown. Gravity load of the viaducts was considered as the initial condition before performing the fundamental mode-based push-over analysis. The push-over curves are shown for each viaduct in Fig. 9. In order to determine the softening part of the push-over curves, the displacement of the monitoring points of the viaducts was selected as relatively high. After from this investigation, a softening branch of the push-over curves was not obtained; however, almost linear sharp decreasing branch was obtained. Since their decreasing branches started at considerably large displacement point not to be displayed in the meaningful range for performance prediction, they were not shown in the push-curves of the viaducts as indicated in Fig. 9. Although these curves seem to be linear, they exhibit

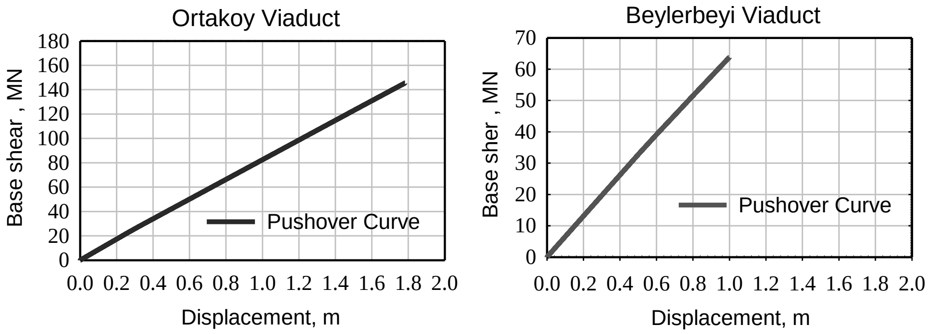


Fig. 9 Push-over curves of the viaducts

a slight non-linearity which corresponds to inelastic deformations. The reason why almost linear behaviors were determined was based on the high degree of freedom of the viaducts resulting from the boundary conditions of the columns shown in Fig. 4. Due to this type of somewhat non-linear behavior of the viaducts, almost same performance points were obtained although some iteration providing to reduce the demand spectrum proposed in the N2 method was made. Therefore, the reduced demand spectrum was not given.

The push-over curves were then transformed to the capacity curves that correspond to the equivalent single-degree-of-freedom (SDOF) system. For this purpose, the base shear (V_{x1}) was transformed into the fundamental mode spectral acceleration (a_1), and the monitoring point displacement (u_{xN1}) was transformed into the fundamental mode spectral displacement (d_1) using the following the equation (Eq. (1))

$$a_1^{(\mathrm{i})} = \frac{V_{x1}^{(i)}}{M_{x1}}, \quad d_1^{(\mathrm{i})} = \frac{u_{x\mathrm{N1}}^{(i)}}{\Phi_{x\mathrm{N1}}\Gamma_{x1}} \tag{1}$$

where M_{x1} is the corresponding effective modal mass for the fundamental natural mode, $\Phi_{x\mathrm{N1}}$ is the modal amplitude of the monitoring mode, and Γ_{x1} is the modal participation factor for the effective natural mode of the viaducts. Similarly, the elastic design spectrum, corresponding to the maximum earthquake level E3 with 2% probability of exceeding in 50 years and a return period of 2475 years as shown in Fig. 10, was converted to the elastic acceleration-displacement response spectrum by the use of the following relation (Eq. (2))

$$S_{di} = S_{ai} \left| \frac{T_i}{2\pi} \right|^2 \tag{2}$$

where S_d is the spectral displacement, $S_a$is the spectral acceleration and T is the corresponding period. As shown in Fig. 11, the capacity curve and the elastic acceleration-displacement response spectrum (ADRS) were depicted graphically in the same coordinate of the spectral acceleration (S_a) versus the spectral displacement (S_d), and the seismic performance point of the viaducts, presenting the spectral displacement demand, was obtained through the application of the equal displacement rule, since the capacity curve intersected with ADRS in the long period range. The corresponding displacements of the viaducts were determined using Eq. (1). The displacement results, obtained for each viaduct, are as follows:

$$u_{xN1}^{(p)} = \Phi_{xN1}\Gamma_{x1}d_1^{(p)} = 1.0\times 1.27\times 0.32 = 0.406m \qquad \text{(Ortakoy viaduct)}$$

$$u_{xN1}^{(p)} = \Phi_{xN1}\Gamma_{x1}d_1^{(p)} = 1.0\times 1.23\times 0.395 = 0.485m \qquad \text{(Beylerbeyi viaduct)}$$

As seen in the Table 2, the plastic deformations occurred only at the top and the bottom sections in the column 3 of the Ortakoy viaduct while the other columns remained elastic, indicating that they did not undergo plastic deformation. Inelastic strains in the sections of the column 3 were obtained through the evaluation of plastic rotations in the plastic hinges.

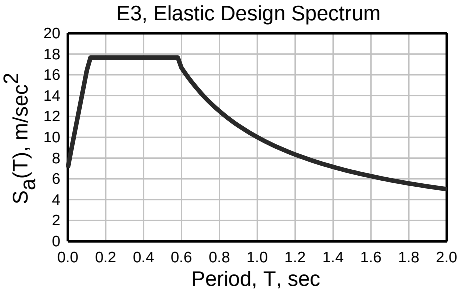
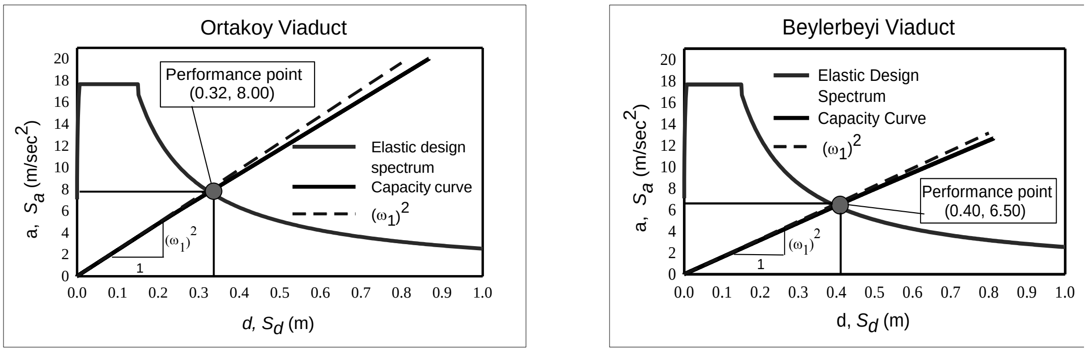


Fig. 10 Elastic design spectrum (EDS) for E3 earthquake level



(a) (b)

Fig.11 Earthquake performance of (a) the Ortakoy and (b) the Beylerbeyi approach viaducts

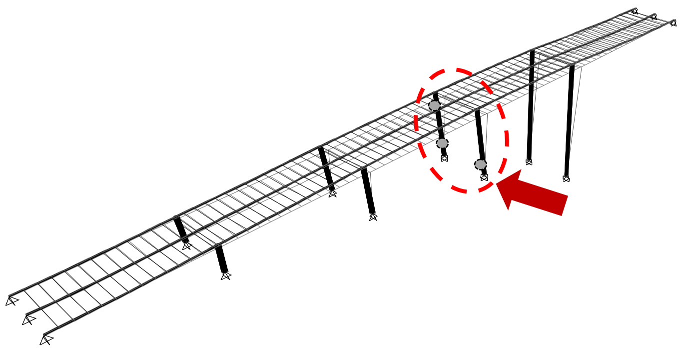


Fig. 12 Plastic hinges: the Ortakoy viaduct according to the POA

Table 2 Push-over analysis results of the viaducts

The Results of Push-over Analysis									
Ortakoy Viaduct					Beylerbeyi Viaduct				
Support No	Column	Column Section	Plastic Rotation (rad)	Plastic Curvature (1/m)	Support No	Column	Column Section	Plastic Rotation (rad)	Plastic Curvature (1/m)
1	P1a	Top	No plastic deformation	Elastic	7	P1a	Top	No plastic deformation	Elastic
		Bottom					Bottom		
	P1b	Top				P1b	Top		
		Bottom					Bottom		
2	P2a	Top			8	P2a	Top		
		Bottom					Bottom		
	P2b	Top				P2b	Top		
		Bottom					Bottom		
3	P3a	Top	0.0031	0.0059	9	P3a	Top		
		Bottom	0.0034	0.0063			Bottom		
	P3b	Top	0.0033	0.0063		P3b	Top		
		Bottom	No plastic deformation	Elastic			Bottom		
4	P4a	Top							
		Bottom							
	P4b	Top							
		Bottom							

Table 3 Sectional and structural earthquake performance of the Ortakoy viaduct

Non-linear Static Push-over Analysis Sectional and Structural Earthquake Performance								
Ortakoy Viaduct								
TSC-R/2008								
Support No	Column	Column Section	Plastic Curvature (1/m)	Total Strain	Strain Limit: Minimum (MN)	Strain Limit: Repairable (RP)	Sectional Performance	Structural Performance
3	P3a	Top	0.0059	0.0061	<0.008	<0.025	MN	Fully Operational
		Bottom	0.0063	0.0063			MN	
	P3b	Top	0.0062	0.0062			MN	
CALTRANS-2001								
Support No	Column	Column Section	Plastic Curvature (1/m)	Total Strain	Strain Limit: Minimum (MN)	Strain Limit: Repairable (RP)	Sectional Performance	Structural Performance
3	P3a	Top	0.0059	0.0061	<0.003	<0.008	RP	Operational
		Bottom	0.0063	0.0063			RP	
	P3b	Top	0.0062	0.0062			RP	

The corresponding results of the calculation are given in Table 2. Damage levels of each section of the columns were calculated by employing the plastic rotation and the total curvature. For the Beylerbeyi viaduct, no plastic deformation was observed at any section of the columns. Accordingly, all columns of the Beylerbeyi viaduct remained elastic. The locations of the inelastic deformation at the columns of the Ortakoy viaduct are illustrated in Fig. 12. Evaluating the results given in Table 2, the performance level of the Ortakoy viaduct was obtained and presented in Table 3 according to the requirements of the TSC-R/2008 and the CALTRANS-2001 separately.

As seen in the table, the plastic deformations of the column at the support 3 were highly effective on the performance level of the system since no damage took place at any of the other columns. As seen from Table 3, the sections of the column at the support 3 were at the minimum (MN) damage level according to the TSC-R/2008; however, these sections remained in the repairable (RP) damage level according to the CALTRANS-2001. The TSC-R/2008 required operational the (OP) performance level for the viaducts, which were considered as critical parts of the bridge. The analysis revealed that the Ortakoy viaduct satisfied the fully operational (FOP) performance level according to the TSC-R/2008 and the operational (OP) performance level according to the requirements of the CALTRANS-2001. However, the Beylerbeyi viaduct satisfied the fully operational (FOP) performance level according to both codes since all members of the viaduct remained elastic.

Table 4 Non-linear time-history analysis results of the viaducts

The results of Non-linear Time-History Analysis								
Ortakoy Viaduct								
Support No	Column	Column Section	Loma Prieta EQ		Erzincan EQ		Kocaeli EQ	
			Plastic Curvature (1/m)	Total Strain (1/m)	Plastic Curvature (1/m)	Total Strain (1/m)	Plastic Curvature (1/m)	Total Strain (1/m)
1	P1a	Top	Elastic	Elastic	Elastic	Elastic	Elastic	Elastic
		Bottom						
	P1b	Top						
		Bottom						
2	P2a	Top						
		Bottom						
	P2b	Top						
		Bottom						
3	P3a	Top	0.0066	0.0063	0.0065	0.0063	0.0072	0.0065
		Bottom	0.0051	0.0051	0.0044	0.0045	0.0051	0.0051
	P3b	Top	0.0061	0.0057	0.0047	0.0046	0.0047	0.0046
		Bottom	0.0051	0.0056	0.0057	0.0057	0.0054	0.0056
4	P4a	Top	Elastic	Elastic	Elastic	Elastic	Elastic	Elastic
		Bottom						
	P4b	Top						
		Bottom						

Table 5 Sectional and structural earthquake performance of the Ortakoy viaduct

Non-linear Time-History Analysis Sectional and Structural Earthquake Performance								
Ortakoy Viaduct								
TSC-R/2008								
Support No	Column	Column Section	Plastic Curvature (1/m)	Total Strain	Limit Strain		Sectional Performance	Structural Performance
					Minimum (MN)	Repairable (RP)		
3	P3a	Top	0.0072	0.0065	<0.008	<0.025	MN	Fully
		Bottom	0.0051	0.0051			MN	
	P3b	Top	0.0061	0.0057			MN	
		Bottom	0.0057	0.0057			MN	
CALTRANS-2001								
Support No	Column	Col. Section	Plastic Curvature (1/m)	Total Strain	Limit Strain		Sectional Performance	Structural Performance
					Minimum (MN)	Repairable (RP)		
3	P3a	Top	0.0072	0.0065	<0.003	<0.008	RP	Operational
		Bottom	0.0051	0.0051			RP	
	P3b	Top	0.0061	0.0057			RP	
		Bottom	0.0057	0.0057			RP	

4.3 Non-linear time-history analysis

For this analysis, the original Loma Prieta (NW, 1989), Erzincan (NE, 1992) and Kocaeli (NE, 1999) earthquake ground motion records were simulated adopting the elastic design spectrum of the maximum earthquake E3, which was required by the TSC-R/2008 due to the significance of the approach viaducts. The results of the NTHA are presented in Table 4, along with the plastic deformations at the column hinges. Since the Beylerbeyi viaduct remained elastic, no result was given in Table 4. As seen in Table 4, inelastic deformation concentrated only at the sections of the columns P3a and P3b of the Ortakoy viaduct. As in the push-over analysis, the other columns remained elastic.

Using the sectional deformation results of the column P3, the sectional and the structural earthquake performance of the Ortakoy viaduct were summarized in Table 5. The sections of the columns P3a and P3b of the Ortakoy viaduct displayed the minimum (MN) damage level according to the TSC-R/2008 and the repairable (RP) damage level according to the CALTRANS-2001.

As it is the case with the push-over analysis, the structural earthquake performance of the Ortakoy viaduct was determined as the fully operational (FOP) and the operational (OP) according to the requirements of the TSC-R/2008 and the CALTRANS-2001, respectively. On the other hand, the Beylerbeyi viaduct directly displayed the fully operational (FOP) earthquake performance level with no plastic deformation.

5. Seismic retrofit investigation for the approach viaducts

The earthquake performance study of the approach viaducts showed that only the sections of the column P3 of the Ortakoy viaduct were subject to plastic deformations higher than the limited strain values given in the codes. In order to improve the performance level of the Ortakoy viaduct, some recommendations for retrofitting of the viaduct were given, and all analyses were repeated based on these retrofit recommendations.

Due to its critical function in the highway networks in Istanbul, no interruption is desired in the operation of the Bosphorus Bridge. For this reason, strengthening of the column P3 adding the steel stiffener elements to the existing circular-box section was recommended. In Fig. 13, general properties and dimensions of the recommended section are given. As shown in Fig. 13, the stiffener elements, T-shaped ribs and the stiffening plate, were used for structural retrofitting. These elements were continuous along the height of the columns including the top and the bottom sections. Bending moment capacity of the retrofitted section was obtained to be almost 5 times higher than that of the current section.

After repeated analyses for determining the earthquake performance of the viaduct with a retrofitted section, the column P3 did not display any inelastic deformation and remain in the elastic range. Thus, it satisfied the minimum (MN) sectional damage level corresponding to the fully operational (FOP) structural performance level according to both codes. In the push-over analysis (POA), the performance point of the retrofitted viaduct was found to be higher than that of the viaduct having the circular-box sections due to the higher displacement stiffness of the retrofitted column P3. Similar results were obtained in the non-linear time-history analysis (NTHA). The fundamental modal period of the retrofitted viaduct was found as T_1=1.15 s whereas the original period was T_1=1.27 s, indicating a 10% decrease in the period. Based on these results, the retrofit recommendation, where T-shaped ribs and the stiffening plates would be added to the current section, was given in detail in Fig. 14.

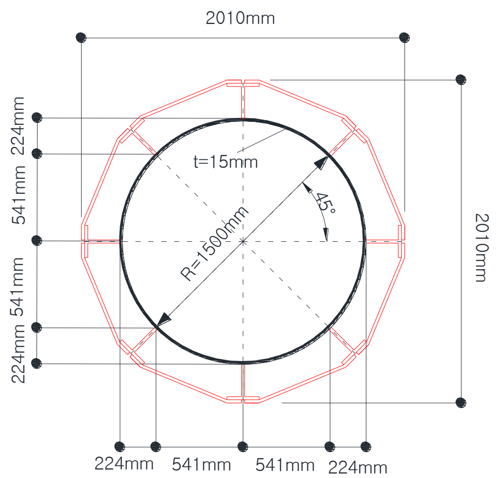


Fig. 13 Recommended section for the column P3

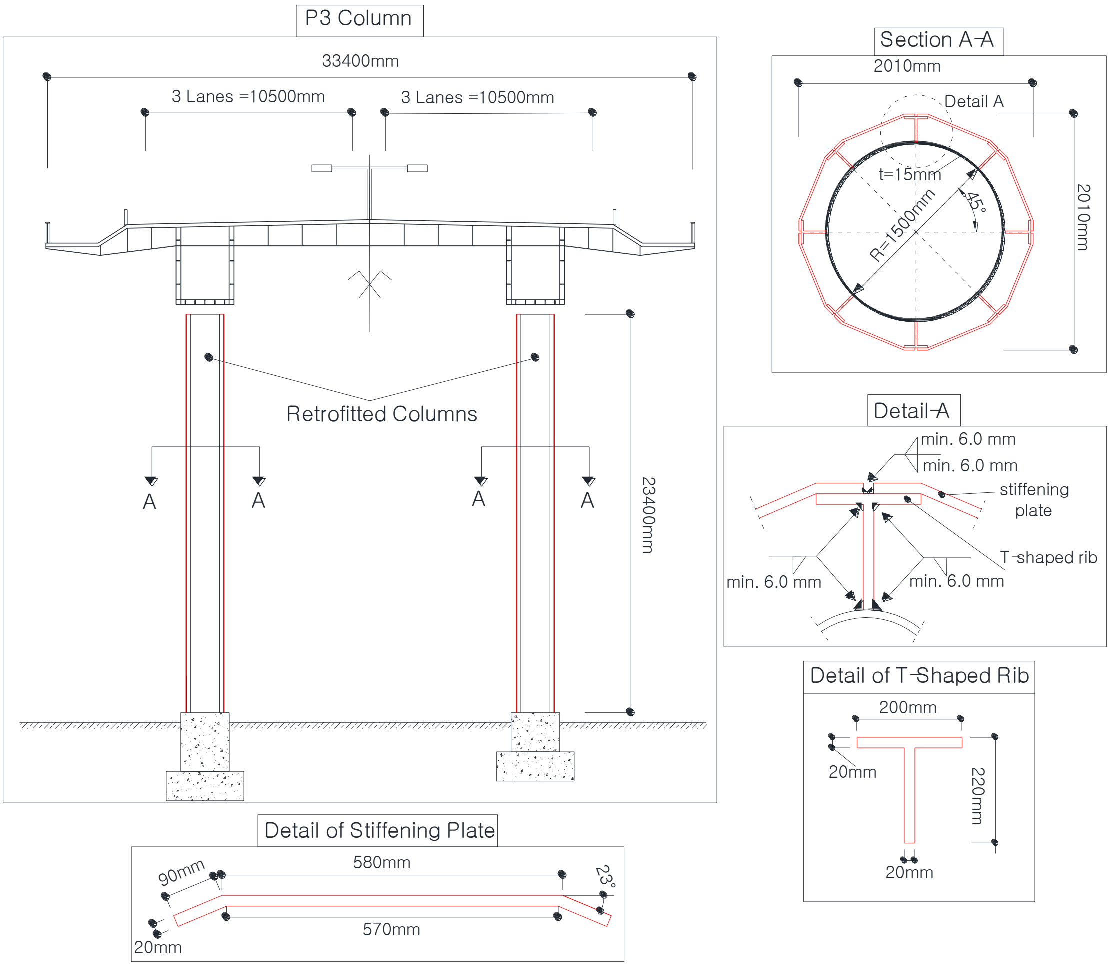


Fig. 14 Structural retrofitting details of the columns P3

The dimensions and thickness values of the elements were determined according to the minimum thickness given in AASTHO/NSBA Steel Bridge Collaboration (2003). Additionally, a fillet weld connection was recommended for these elements considering the minimum fillet weld size given in AASHTO/LRFD Bridge Design Specifications (2007). The residual stress on the welded element will be reduced by symmetrically locating the T-shaped ribs and the stiffening plate on the current circular-box section. Based on the retrofitting recommendation, the buckling capacity of the columns was also provided to increase.

6. Conclusions

The Ortakoy and the Beylerbeyi approach viaducts have not any suspender element and comprise of almost identical deck systems, consisting of the continuous box-girders and the cross-beams. The main part of the bridge, i.e., the deck, stiffened by panels, has an aerodynamic cross-section which is continuous along the viaduct axis in the two cases. However, the viaducts have different support conditions at the tower (expansion joints) and at the anchorage points. Therefore the Ortakoy and the Beylerbeyi approach viaducts have separate structural systems. In the present study, these different structural properties were considered, and accordingly, the analysis was carried out separately.

Seismic performances of the viaducts were evaluated employing the push-over (POA) and non-linear time-history analyses (NTHA). The push-over analysis was performed adopting the effective lateral mode shape. The push-over and the non-linear time-history analyses revealed that the behavior of the viaducts was affected by the support conditions of the columns and support conditions at the expansion joints. The push-over analysis showed that the Ortakoy viaduct met the requirements of the TSC-R/2008 with the fully operational (FOP) structural performance level while the operational (OP) structural performance level was provided according to the provisions of the CALTRANS-2001. However, the Beylerbeyi viaduct provided the directly minimum (MN) sectional damage level and the fully operational (FOP) structural performance level according to both codes since they did not involve any inelastic deformed section. From the non-linear time-history analysis, the Ortakoy viaduct provided the fully operational (FOP) structural performance level according to the TSC-R/2008 and the operational (OP) structural performance level according to the CALTRANS-2001. Consequently, the viaducts satisfied the same earthquake performance level according to the results of the POA and the NTHA. On the other hand, the damage on the column P3, detected via the NTHA was higher than those detected via the POA. The non-linear analyses of the approach viaducts indicated that the damage was concentrated on the columns of the support 3, and no damage was found on the other columns. These results indicated that the Ortakoy approach viaduct reached to the limit in the case of the support 3. Due to large lateral displacements, inelastic deformations were obtained at the top of the column. As expected, the behavior of the approach viaducts and the main span of the bridge were different under earthquake motion owing to their different structural systems. The study also showed that the single mode POA led to very accurate results in case of the viaducts, where the first mode was very effective during earthquake motion. After performance evaluation of the viaducts, the retrofit recommendation for the column P3 was investigated. Since no interruption was desired in the critical function of the Ortakoy approach viaduct in the highway networks in Istanbul, it was decided that the P3 columns of the viaduct were to be retrofitted adding the stiffener steel elements, T-shaped ribs and the stiffening plate, to the existing section as shown in Fig.13. In consideration of the sectional specifications of the recommended section for retrofitting, the performance analysis of the Ortakoy viaduct was repeated. According to the results, no inelastic deformation was obtained in all sections. Thus, the retrofitted sections of the column P3 met the minimum (MN) sectional damage level and the fully operational structural performance level (FOP) according to both codes. Based on these results, the retrofitting study was detailed for implementation. In Fig. 14, all specifications of the retrofit recommendation are given. As seen in the Fig. 14, the T-shaped ribs and the stiffening plate were continuous along the height of the column. While deciding on this retrofitting recommendation, reducing the residual stresses in the welded elements, and increasing the buckling capacity of the column were also considered.

Hopefully, all the results obtained from this study are useful for the authority responsible for these critical structures on the basis of identification of the behavior of the viaduct and their structural rehabilitation in the case of a possible earthquake in Istanbul.

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