Steel and Composite Structures, *Vol. 26, No. 2 (2018) 197-211*
DOI: https://doi.org/10.12989/scs.2018.26.2.197

Multi-point earthquake response of the Bosphorus Bridge to site-specific ground motions

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(*Received July 6, 2017, Revised October 12, 2017, Accepted December 10, 2017*)

Abstract. The study presents the earthquake performance of the Bosphorus Bridge under multi-point earthquake excitation considering the spatially varying site-specific earthquake motions. The elaborate FE model of the bridge is firstly established depending on the new considerations of the used FEM software specifications, such as cable-sag effect, rigid link and gap elements. The modal analysis showed that singular modes of the deck and the tower were relatively effective in the dynamic behavior of the bridge due to higher total mass participation mass ratio of 80%. The parameters and requirements to be considered in simulation process are determined to generate the spatially varying site-specific ground motions. Total number of twelve simulated ground motions are defined for the multi-support earthquake analysis (*Mp-sup*). In order to easily implement multi-point earthquake excitation to the bridge, the practice-oriented procedure is summarized. The results demonstrated that the *Mp-sup* led to high increase in sectional forces of the critical components of the bridge, especially tower base section and tensile force of the main and back stay cables. A close relationship between the dynamic response and the behavior of the bridge under the *Mp-sup* was also obtained. Consequently, the outcomes from this study underscored the importance of the utilization of the multi-point earthquake analysis and the necessity of considering specifically generated earthquake motions for suspension bridges.

Keywords: suspension bridge; multi-point earthquake analysis; spatially varying site-specific earthquake motions; finite element model

1. Introduction

1.1 Literature survey

Long-span bridges are special structures compared to other structures. They are not only a component of transportation system of a country/state but also are kept in mind as the symbol of their located region. Hence, many transportation departments pay special attention to this type of bridges to continue their service without any interruption. Of many costs of long-span bridge, such as maintenance, management i.e., the rehabilitation cost for structural safety constitutes of the major part of the allocated budget. Due to higher complexity and vulnerability of long-span bridges to unpredictable extreme events than other type of structures, such as seismic, strong wind and marathon etc., better understanding the structural behavior of them under these events become inevitable for reliable structural rehabilitation.

One of the most significant events for all structures is earthquake excitation because of the various uncertainties in terms of seismology and response of structures to this event. Seismological properties of earthquakes, such as fault mechanism, seismic hazard and risk assessment etc. have been identified well utilizing the probabilistic and deterministic approaches. The response of structures to earthquake event; however, has still been studied by researchers in the field of structural engineering although many significant provisions are proposed by seismic codes, experimentally and theoretically conducted studies. With the help of field-reconnaissance and post-earthquake assessment after from destructive earthquakes, various methods and philosophies for earthquake analysis and earthquake-resistant design have been accurately developed, especially for the building structures. For seismic structural analysis of large-scale structures, such as long-span suspension or cable-stayed bridges, such advances are partially acceptable due to the complexity of these structures, which means that there is no general codes or standards for large-scale bridges. Therefore, special efforts for seismic analysis and design of long-span bridges need to be made by identifying its current structural properties and site soil conditions properly.

In the seismic analysis of structures, it is general assumption that earthquake induces structures uniformly, which means that the effects of site soil conditions are not

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http://www.techno-press.org/?journal=scs&subpage=6
ISSN: 1229-9367 (Print), 1598-6233 (Online)

considered. This idealization can be taken into account for building and short-span structures; however, for large-scale structures whose supports are likely to have different soil conditions, a special attention to these bridges needs to be paid. Therefore, various investigations and studies have been conducted in literature to identify the earthquake behavior of suspension bridges and cable-stayed bridges. The first prominent scientific studies in literature are conducted from Abdel-Ghaffar. The vertical and lateral response of the Golden Gate Suspension Bridge to multi-support earthquake motions was also studied by Abdel-Ghaffar and Rubin (1983a, b). They exhibited that more realistic response of suspension bridges could be obtained considering a number of modes and that uniform-support excitation did not yield to reliable results for the extended structures due to not having the ability to find out the most unfavorable case. In order to identify the nonlinear behavior of suspension bridges, an approach was proposed by Abdel-Ghaffar and Rubin (1983c), and they indicated the applicability of this approach considering the Golden Gate Bridge and the Vincent Thomas Bridge. A new method was proposed by Kiureghian and Neuenhofer (1992) for the multi-support earthquake analysis of structures under spatially varying earthquake excitations accounting for wave passage, coherency and local soil condition effects. Dumanoglu and Soyluk (2003) conducted a study on stochastic response of cable-stayed bridge under spatially varying seismic motions. In that study, dynamic behavior of long-span bridges was demonstrated to be relatively affected from spatially varying ground motions. Therefore, they concluded that the varying earthquake motions should be considered in the analysis of large-scale structures. Site-specific ground motions were utilized for stochastic earthquake analysis of a cable-supported bridge. From the analysis, the supports of the bridge were relatively affected under spatially varying earthquake motions (Dumanoglu and Severn 1990, Dumanoglu and Soyluk 2003). Recently conducted studies from (Alexander 2008, Wang *et al*. 2009, Karmakar *et al*. 2012, Soyluk and Sicacik 2012, Zhao *et al*. 2015, Adanur *et al*. 2016a, Altunisik and Kalkan 2016, Adanur *et al*. 2016b) were also focused on the investigation of the effects of multi-support earthquake excitation on existing long-span bridges. The multi-support seismic excitation was also considered for concreted filled steel tubular (CFST) arch bridge by Bi *et al*. (2013). They showed the importance of consideration of the multi-support earthquake excitation for bridge structures. In theoretical aspect, Roudsari and Hosseini (2013) carried out a study to determine a relation between multi-component and multi-support excitation analyses. They proposed a matrix transformation and showed that an earthquake influence factor could not be defined for all structures under multi support earthquake excitation.

After from the destructive earthquakes in last two decades in Turkey, Izmit (1999) and Duzce (1999) earthquakes, the public awareness of structural earthquake safety and performance of the existing structures in Turkey has increased progressively. General Directorate of Turkish State Highways (KGM) conducted a number of rehabilitation projects (JBSI 2004) for the most critical long-span bridges in Turkey, Fatih Sultan Mehmet Bridges and the Bosphorus. Besides, researchers in bridge engineering carried out important studies for these bridges. The first informative and prominent studies were conducted by (Brownjohn *et al*. 1989, 1992, Erdik and Uckan 1989, Dumanoglu and Severn 1990, Dumanoglu *et al*. 1992) on system identification of the bridges based on the experimental results. Ambient vibration test was performed by Brownjohn *et al*. (1989) and Erdik and Uckan (1989) to extract the vibration properties of the Bosphorus Bridge using monitoring data. They resulted in these studies with a closure agreement between experimental outcomes and those from numerical analysis. Brownjohn *et al*. (1992) conducted a much more comprehensive study on full-scale dynamic testing of the Fatih Sultan Mehmet Bridge to identify experimental and theoretical dynamic characteristics of the bridge. Utilizing data recorded from the reference accelerometers installed on the critical points at the deck and the towers of the bridge, lateral, vertical, torsional and longitudinal mode shapes and associated frequencies were extracted. In other studies from (Brownjohn *et al*. 1992, Dumanoglu *et al*. 1992), dynamic properties of the Fatih Sultan Mehmet Bridge and earthquake-induced behavior of the bridge were also investigated. Recently new studies were also conducted for these bridges by Apaydin (2010), Kosar (2003) and Erdik and Apaydın (2007). In order to determine natural vibration characteristics of the Bosphorus Bridge and to verify experimental results with those from the other studies in literature, ambient vibration survey was utilized and finite element model-FE of the bridge was established by Kosar (2003). Seismic performance evaluation of two approach viaducts of the Bosphorus Bridge was investigated by (Bas *et al*. 2015, 2016) to indicate the efficiency of the performance-based assessment and design code of TSC-R (2008). The outcomes obtained from the study showed good relationship between the provisions of TSC-R (2008) and CalTrans (2001). Depending on the performance of the viaducts, retrofitting investigation was made for their columns. Consequently, this study has revealed that performance evaluation of the side-span including damage investigation of potential structural elements is necessary for reliable retrofitting of the bridge although no plastic deformation need to be considered for the main span of the bridge. Detailed study on the earthquake behavior and retrofit needs for the Fatih Sultan Mehmet Bridge and the Bosphorus Bridge was carried out by Apaydin (2010). For this purpose, realistic uniform earthquake motion records generated according to the seismic demand of the bridges given in KGM (2004) were utilized considering site-soil properties of the located region of the bridges and the earthquake scenario Mw = 7.5 for active Marmara Fault. This study also presented the considerations for FE model of the bridges. Based on the results from the uniform-support earthquake analysis (U-sup), the needs for considering the multi-support earthquake analysis (Mp-sup) were stated in that for long-span bridges. Moreover, dampers replaced to the tower-deck points close to the rocker bearings was recommended so as to reduce longitudinal translation of the deck. Apaydin *et al*. (2016)

conducted recently new study on the performance prediction of the Fatih Sultan Mehmet Bridge under site-specific spatially varying multi-point earthquake excitation. They utilized specifically generated strong ground motions for the earthquake analysis. The comparative outcomes from the study indicated that the *Mp-sup* analysis should be considered for long-span suspension bridge to reliably identify structural behavior of the bridge and thus to determine the most suitable retrofit project for the bridge.

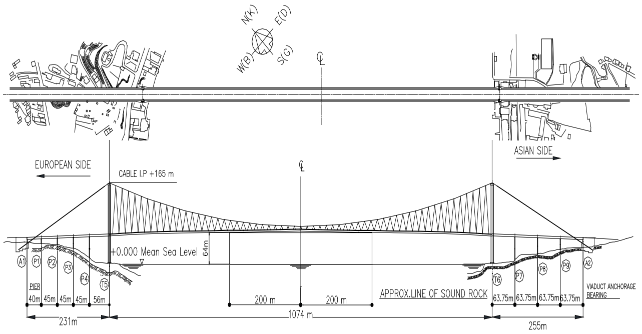
1.2 Aims and scope of the study

As mentioned in the references, either no or limited elaborate investigations on structural performance of the Bosphorus Bridge were made. Particularly, there is no specific study for the multi-point earthquake analysis of the bridge in the literature. The related studies in literature were basically focused on the uniform support analysis (U-sup) of the bridge. Therefore, the *Mp-sup* analysis is required to better understand the seismic behavior of the Bosphorus Bridge. Considering these recommendations, this study aims at determining the effects of spatially varying earthquake motion on the Bosphorus Bridge, at presenting the considerations for FE modeling of the bridge and the practice-oriented multi-point earthquake analysis procedure for the bridge, and at offering more reliable results from the multi-point earthquake analysis to recommend the most suitable retrofit strategy for the bridge. For this objective, spatially varying site-specific earthquake motion records are generated taking the geographic coordinates and site soil condition of the bridge’s supports of the anchorage and tower at each side into account. In order to make earthquake analysis of the bridge, a powerful tool of finite element-FE modeling is used and detailed FE modeling considerations were specified for the bridge. Accordingly, the detailed 3-D FE model of the bridge is established. In addition, practice-oriented multi-point earthquake analysis procedure easily implementable for long-span structures is proposed. Utilizing the sophisticated FE model of the bridge and proposed analysis method, the results obtained from the *Mp-sup* are presented and are compared with those from the retrofit project of (JBSI 2004) and the *U-sup* study of Apaydin (2010) to demonstrate the effect of the *Mp-sup* on the seismic performance of the bridge. Thus, the most critical bridge’s components in terms of structural rehabilitation are determined. Certain recommendations are given for these critical elements of the bridge.

2. The Bosphorus Bridge and general properties

The Bosphorus Bridge as shown in Fig. 1(a), also called the 1st Bosphorus Bridge, is one of the two long-span bridges in Turkey, providing a connection between two continents, the Asian and the European. When opened to traffic in 1973, the bridge was classified as the 4th longest suspension bridge in the world according to its main span. The bridge serves as vital link on the Motorway-1 (O1) connecting the city center of Istanbul. Significant part of heavy traffic of Istanbul has been carried from the bridge along with the Fatih Sultan Mehmet Bridge named the 2nd Bosphorus Bridge located on the northern side of the Bosphorus Bridge.

(a)



(b)

Fig. 1 (a) General view; and (b) arrangement of the Bosphorus Bridge (Freeman *et al*. 1968)

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3. Finite Element Modeling (FE) of the bridge

Based on the project specifications and general properties of the bridge in the previous section, elaborate FE model of the bridge are developed utilizing the spine-beam modeling approach. For this objective, the structural analysis software SAP2000 (CSI 2016) is adopted. Due to the considerations of cable-sag effect, gap, rigid link and large-displacement option properties, SAP2000 is selected to develop 3-D FE model and to perform *Mp-sup* time-history analysis of the bridge. The bridge’s structural components of the tower, the main deck, the portal beams, and approach span are modeled as equivalent frame element corresponding their mechanical and sectional properties. According to the technical calculations of the bridge, the high yield structural steel of BS-968 (BS968 1962) is originally used for the bridge. However, no detailed specifications for this code are available in literature;

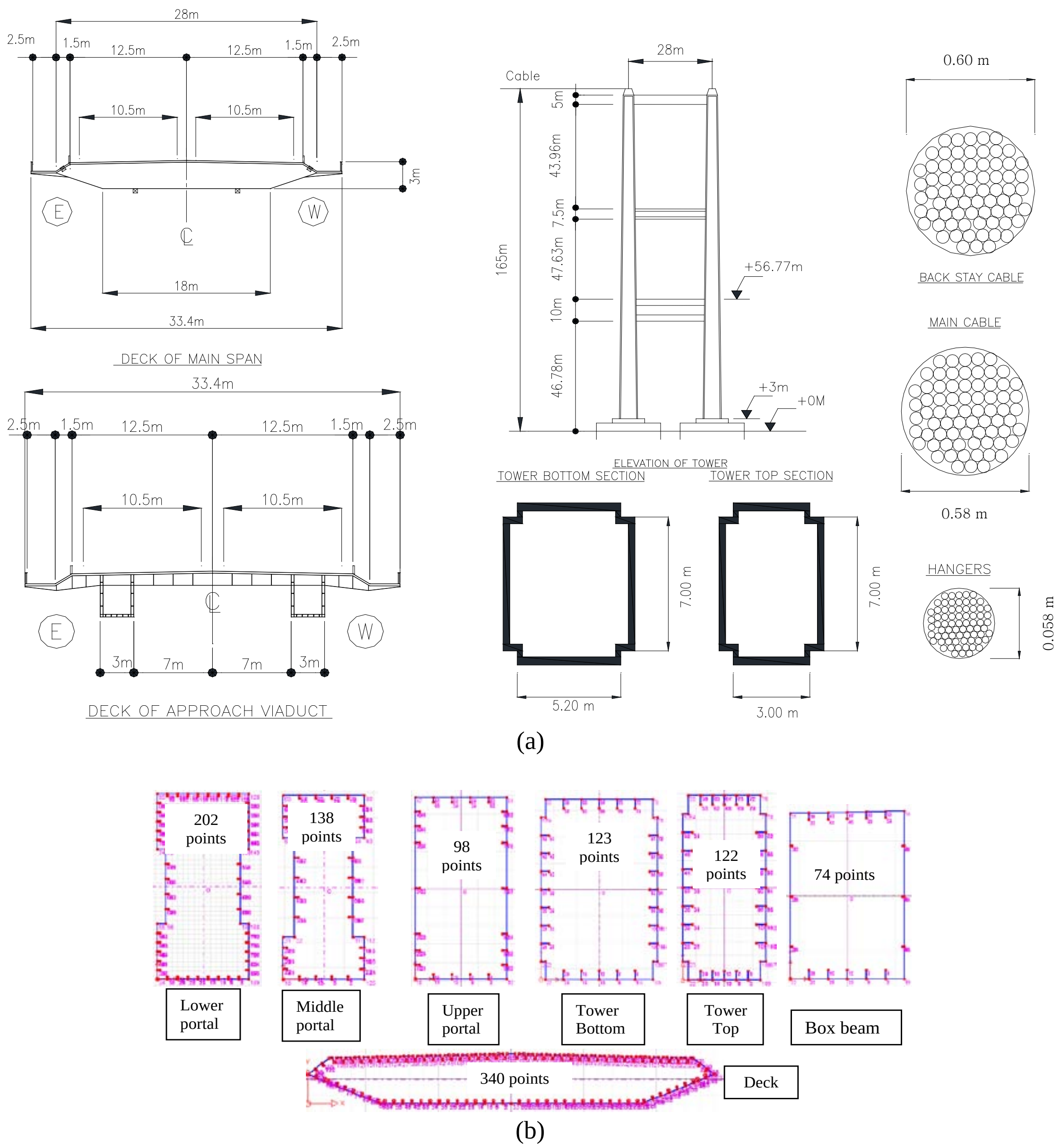


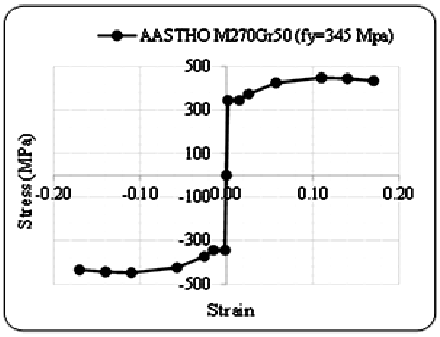
Fig. 2 (a) Sectional properties of the bridge (Freeman *et al*. 1968); and (b) Sectional points of the component

therefore, ASTM/A709-Gr50 steel model that corresponds to the provisions of the BS-968 is considered as structural steel for FE modelling and the dynamic and earthquake analysis of the bridge. More details for the material are given in Fig. 3.

For elaborate sectional properties, all points of the components as indicated in Fig. 2(b) are precisely determined depending on the project drawings, and thus much more realistic dimensions of them are adopted. Considering these realistic dimensions, sectional parameters of the structural components of the bridge given Table 1 are obtained.

As to the FE model considerations for the bridge, 3-D frame element is utilized for the deck, the tower, the portal beams, the approach span box beam and the circular box columns. In order to take the cable sag effects significant for the *P*-Δ analysis into account, the main cable, the back-stay cable and the hangers are modeled cable element. Besides, link elements with no mass are also utilized for the tower-deck and approach span-tower connections. For the rocker bearings, gap elements are also considered. In Fig. 4, these considerations are represented in detail. So as to provide the connection between the hanger elements and the main deck element, rigid link elements with higher rigidity for all degree-of-freedom are used. As shown in Fig. 4, the boundary conditions of the towers, the circular box columns and the approach span box column at anchorage points are also defined. For the asphalt pavement on the approach span deck, and concrete and asphalt pavements on the main span deck, shell element specifications are assigned.

Based on the sectional specifications and the FE modeling considerations, 3-D finite element model of the bridge is developed. The views from the model are given in Fig. 5. As shown in Fig. 5, the camber at the mid-span of the deck is also provided in the model. The established model is considered for the modal analysis and the multi-



Stress (Mpa)	Strain
-434	-0.1700
-444	-0.1400
-448	-0.1100
-425	-0.0572
-373	-0.0256
-345	-0.0150
-345	-0.0017
0	0.0000
345	0.0017
345	0.0150
373	0.0256
425	0.0572
448	0.1100
444	0.1400
434	0.1700

Fig. 3 Structural steel specifications of the bridge

Table 1 Cross-sectional properties of the structural components of the bridge

Parameters	Tower		Main deck	Portal beam			Box beam
	Bottom	Top		Lower	Middle	Upper	
Area (mm^2)	6.00×10^5	6.00×10^5	7.00×10^5	3.00×10^5	2.00×10^5	2.00×10^5	2.00×10^5
I_{xx} (mm^4)	5.00×10^{12}	3.70×10^{12}	1.10×10^{12}	4.00×10^{12}	1.70×10^{12}	8.00×10^{11}	7.00×10^{11}
I_{yy} (mm^4)	3.10×10^{12}	1.00×10^{12}	5.00×10^{13}	1.00×10^{12}	2.70×10^{11}	2.70×10^{11}	2.80×10^{11}
Torsional J (mm^4)	1.55×10^{14}	3.54×10^{14}	4.00×10^{12}	8.10×10^{12}	1.10×10^{12}	3.90×10^{12}	4.80×10^{12}
Shear X Area (mm^2)	6.00×10^5	5.00×10^5	6.00×10^5	2.00×10^5	1.00×10^5	2.00×10^5	2.00×10^5
Shear Y Area (mm^2)	6.00×10^5	5.00×10^5	6.00×10^5	2.00×10^5	2.00×10^5	2.00×10^5	2.00×10^5
Plastic Z_x (mm^3)	1.60×10^9	1.30×10^9	8.00×10^8	1.00×10^9	5.00×10^8	3.00×10^8	4.00×10^8
Plastic Z_y (mm^3)	1.30×10^9	7.00×10^8	4.00×10^9	6.00×10^8	3.00×10^8	2.10×10^8	2.30×10^8
Section Mod. S_3 (mm^3)	1.10×10^9	9.00×10^8	1.00×10^8	7.00×10^8	3.00×10^8	3.00×10^8	2.10×10^8
Section Mod. S_2 (mm^3)	9.00×10^8	4.80×10^8	3.00×10^9	4.00×10^8	1.50×10^8	1.40×10^8	5.74×10^9
Gyration, r_3 (mm)	2.89×10^3	2.48×10^3	1.25×10^3	3.65×10^3	2.92×10^3	2.00×10^3	1.87×10^3
Gyration, r_2 (mm)	2.27×10^3	1.29×10^3	8.45×10^3	1.83×10^3	1.16×10^3	1.16×10^3	1.18×10^3

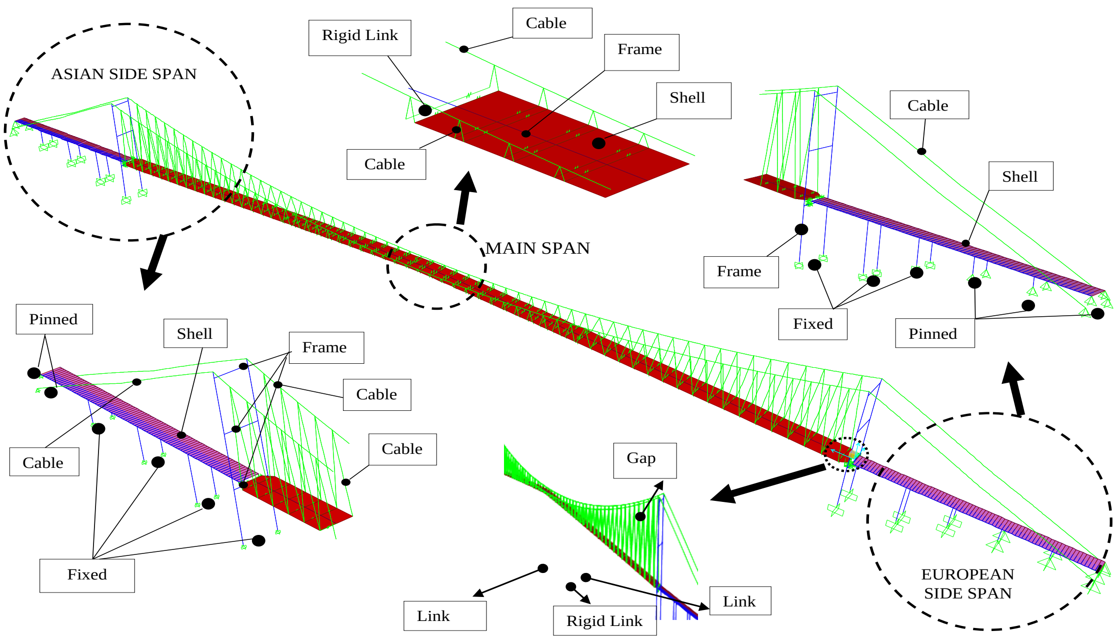


Fig. 4 FE modeling considerations for the Bosphorus Bridge

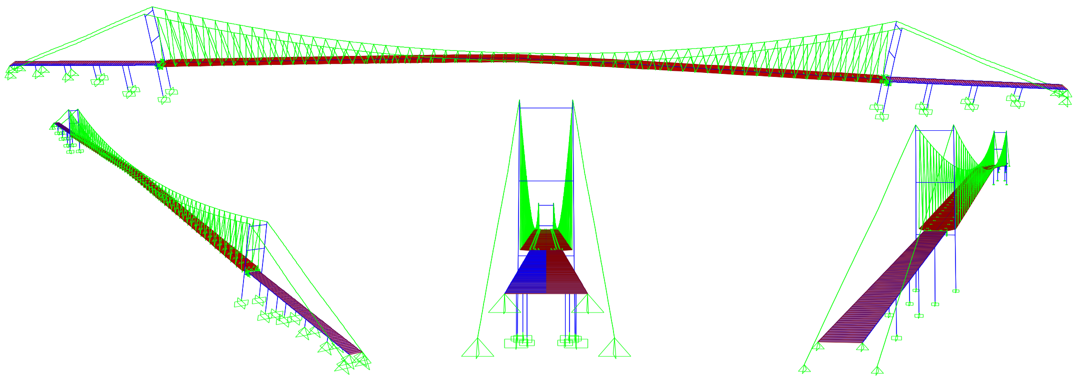


Fig. 5 3-D FE model of the bridge

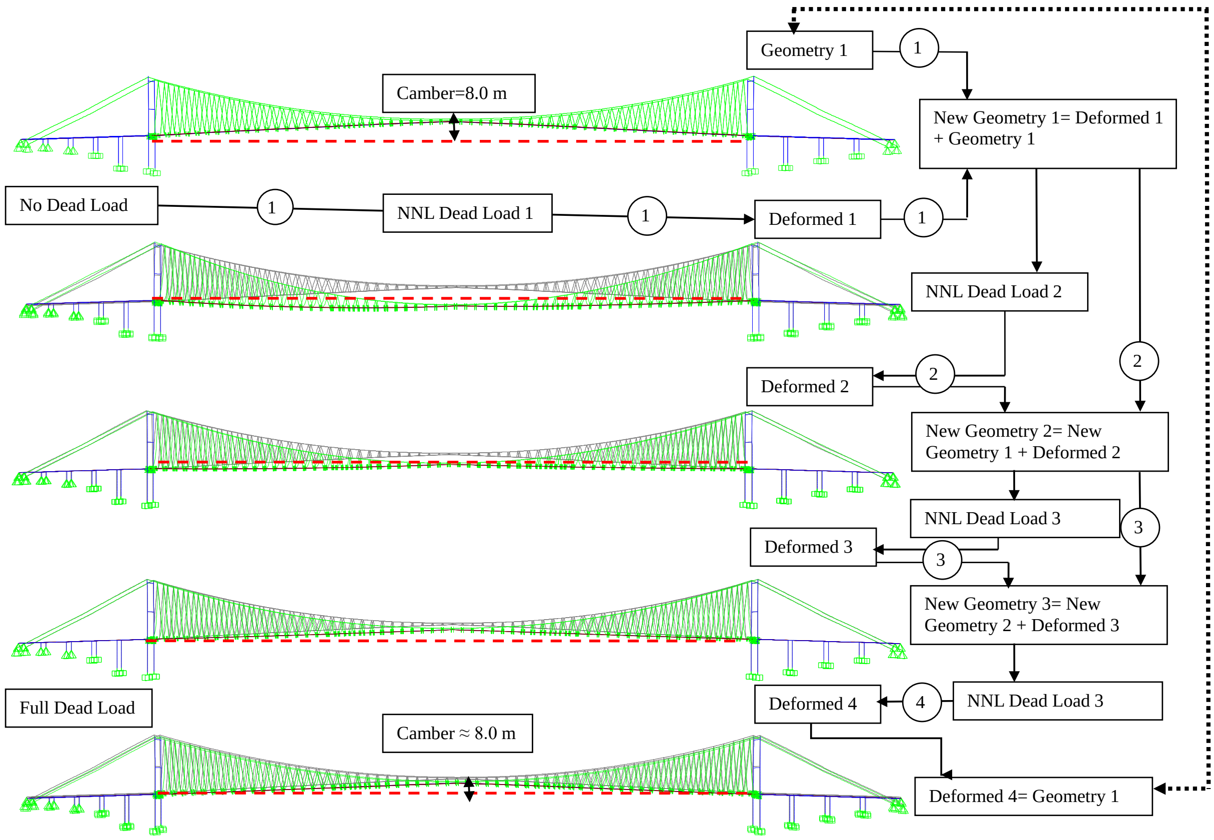


Fig. 6 General steps for dead-load initial condition consideration

point earthquake analysis of the Bosphorus Bridge. The accuracy of the FE model is verified by the comparison with the numerical and experimental studies in literature.

4. Modal analysis of the bridge

The modal analysis of structures is a powerful tool for earthquake excitation analysis of structures. Through this analysis, the response of structures to dynamic input can be estimated and certain outcomes related to dynamic inputs can be explained by Chopra (2012). For large-scale bridge structures with different size of structural component, such as main deck, tower etc., the mode shapes may show which component dominated the dynamic response of long-span bridges (Apaydin *et al*. 2016). In order to verify the developed FE model and to identify the results of the multi-point earthquake analysis of the bridge, the modal analysis is first employed.

Due to relatively less displacement of structures with short span in plan and in elevation, the dead-load (DL) can be ignored to be considered as initial loading condition for the dynamic analysis. However, for long-span cable-supported bridges, this initial condition plays critical role to accurately obtain modal frequencies and corresponding mode shapes of them. To illustrate, the Bosphorus Bridge has a camber of 8.0 m height at the middle of the main deck under dead-load. In an attempt to provide the camber and dead-load as initial condition, the finite-element-based practice-oriented approach is utilized for the Bosphorus

Bridge. Prior to the modal analysis of the bridge, this approach is considered.

For this objective, general steps of the procedure are summarized in Fig. 6. In the procedure, the geometry of the bridge was firstly determined depending on the project drawings. This geometry without dead-load is called “Geometry 1” as shown in Fig. 6. Non-linear geometric analysis (NNLGEO) is then performed considering “Geometry 1”. Thus, the deformed shape of the bridge after from the analysis, “Deformed 1” and “Geometry 1” are used for the next step, “New Geometry 1”. Performing NNLGEO analysis of “New Geometry 1”, “Deformed 2” is determined. Similar steps are conducted iteratively. In all steps, displacement of the nodal points is obtained for each deformed shape and these deformed values are added to the previous geometry. After from three steps, the camber of 8.0 m height is determined under NNLGEO dead-load, which means that all coordinates of the nodal points from the analysis of “New Geometry 3” are provided to be same as those of “Geometry 1”. Accordingly, “New Geometry 3” equal to “Geometry 1” under NNLGEO dead load is considered both for the modal analysis and for the multi-point earthquake analysis of the bridge.

Since the bridge was made of structural steel, modal damping ratio of $\xi = 0.02$ is also considered to calculate the proportional structural damping for both bridges. The first 50 natural frequencies and associated mode shapes are obtained and the first five modes are shown in Fig. 7. From the analysis, the main deck of the bridge is obtained to be effective for lateral and vertical response of the bridge to a dynamic input. Particularly, modal participating total mass ratio for transvers direction of the main deck is determined as 60% at the end of the first five modes directly pertinent to the main deck mode shapes. Compared to modal participating total mass ratio of 96% at the end of the fifty modes, this value indicated the efficiency of the main deck mode shapes on the dynamic response of the bridge. Similar single mode shapes are also determined for the tower and cable after the main deck mode shapes. All these single mode shapes of the main deck, the tower and the cables are seen in the first ten mode shapes. The other mode shapes are obtained as the combination of these single mode shapes. Based on these consequences, the main deck and the tower dynamic response are estimated to dominate the behavior of the Bosphorus Bridge under multi-point earthquake excitation. Since the Bosphorus Bridge has currently been in operation only for cars, truck lanes are not taken into account for the live-load (LL).

For this objective, H30-S24 truck load is adopted for live-load consideration. According to the Technical Specifications for Highways Bridges (KGM 1982), lane traffic load for standard cars can be considered as 1/3 of AASHTO (2002) H30-S24 truck load of 9.0 kN/m. As shown in Fig. 8, each lane of the bridge is loaded with uniformly distributed lane traffic load of 3.0 kN/m. Thus, total uniform load of 18.0 kN/m is considered for the live-load of the Bosphorus Bridge. The results from the modal analysis of the bridge with the live-load are given in Table 2. Based on the comparison in the table, the traffic load is concluded to be necessarily taken into account for reliable performance assessment of the bridge due to relatively change of the modal value between the load cases. The LL is considered for the multi-point earthquake analysis of the bridge.

5. Spatially varying site-specific earthquake motion

In order to simulate site-specific ground motions, geographic coordinates of the bridge’s support points has firstly to be determined. As indicated in Fig. 9, the support coordinates of the bridge are obtained depending on the general coordinates of the bridge. Fig. 9 also presents the general considerations of the practice-oriented multi- point earthquake analysis of the bridge.Utilizing these coordinates of proposed by Boore the bridge, the stochastic

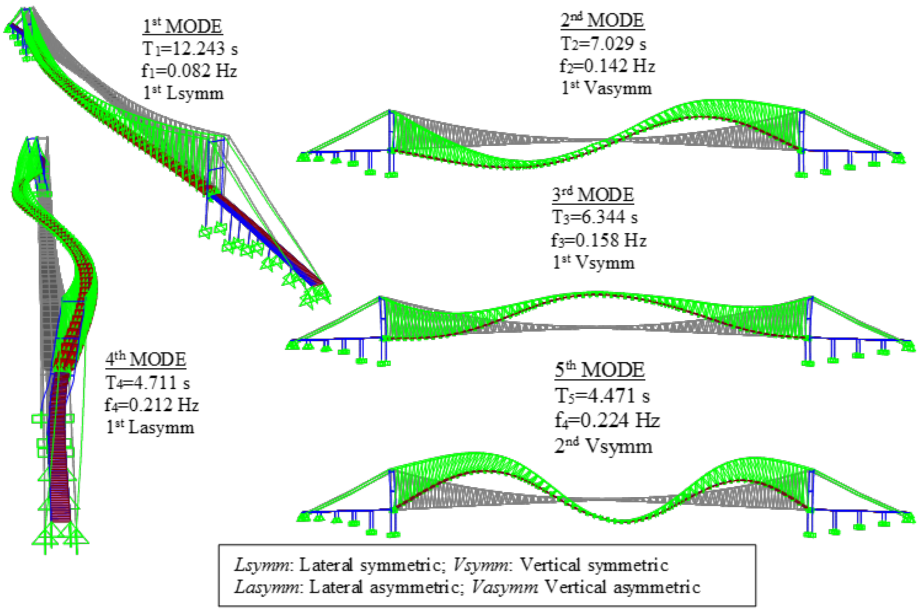


Fig. 7 The first five mode shapes of the Bosphorus Bridge

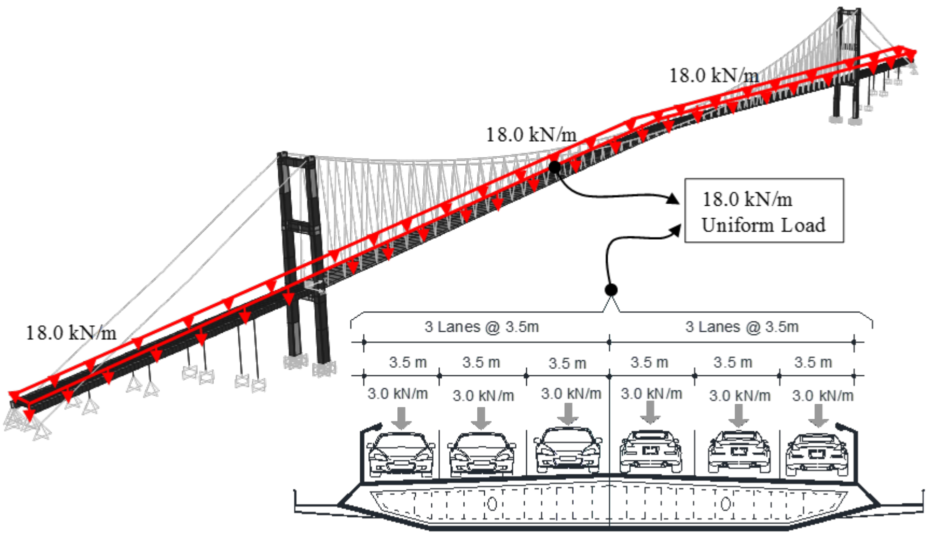


Fig. 8 Considerations for live-load of the bridge

Table 2 Modal analysis results

Mode Number	Mode Shape	Frequency/Period [Hz]/[s]							
		(Brownjohn *et al.* 1989)	(Erdik and Uckan 1989)	(Kosar 2003)	(Apaydin 2010)	Current FE (DL)		Current FE (LL)	
						Period [s]	Freq. [Hz]	Period [s]	Freq. [Hz]
Mode-1	1^{st} L_{sym}	0.073	0.072	0.069	0.074	12.243	0.082	13.628	0.073
Mode-2	1^{st} V_{asym}	0.126	0.144	0.125	0.120	7.029	0.142	7.316	0.137
Mode-3	1^{st} V_{sym}	0.165	0.202	0.190	0.158	6.344	0.158	6.990	0.143
Mode-4	1^{st} L_{asym}	0.180	0.225	0.223	0.210	4.711	0.212	5.369	0.186
Mode-5	2^{nd} V_{sym}	0.218	0.323	0.273	0.262	4.471	0.224	4.734	0.211

L_{sym}: Lateral symmetric; L_{asym}: Lateral asymmetric; V_{sym}: Vertical symmetric; V_{asym}: Vertical asymmetric; DL: Dead-load; LL: Live-load

modeling technique (1983) is used to generate earthquake ground motions. This technique that considers the earthquake as a point source was then extended by Beresnev and Atkinson (1997, 1998). They divided the fault plane into small rectangular subfaults treating as point source. The subdivided faults featured separated point sources. This extended new version having stochastic character was programmed by Beresnev and Atkinson (1998) with the name of FINSIM (FINite fault SIMulation program).

Taking the scenario earthquake of Mw = 7.4 predicted to occur with the probability of 70% in next 30 years in Istanbul into consideration, the FINSIM was modified and adapted by Böse (2006) to develop early-warning system for Istanbul estimating site-specific ground motions. Further explanations for these considerations could be found in the study of Böse (2006). Another important step to produce site-specific earthquake ground motion for the bridge is to specify the parameters to be used in the modified FINSIM. For this purpose, the detailed study of Ansal *et al*. (2009) is utilized in the FINSIM. The parameters that are determined for the loss estimation in Istanbul and that reflect deterministically the seismicity of Istanbul are given in Table 3.

Considering the scenario earthquake with 7.4 magnitude in Istanbul, the simulated ground motions are then prepared

Table 3 The parameters for simulation of ground motions

Parameter	**Parameter value**
Fault orientation	Strike 81.5°, Dip 90°
Fault dimensions along strike and dip (km)	108 by 20
Stress parameter (bars)	100
Subfault dimensions (km)	10 by 10
Moment (dyn·cm)	1.7×10^{27}
Moment magnitude	7.4
Inelastic attenuation $Q(f)$	$180 \cdot f^{0.45}$
Geometric spreading	$1/R \leq 30$ km $1/R^{0.4}$ 30-60 $1/R^{0.6}$ 60-90 $1/R^{0.8}$ 90-100 $1/R^{0.5} > 100$
Windowing function	Saragoni-Hart
Crustal-shear wave velocity (km/sec)	3.3
Crustal density (g/cm^3)	2.7
Focal mechanism	Strike slip

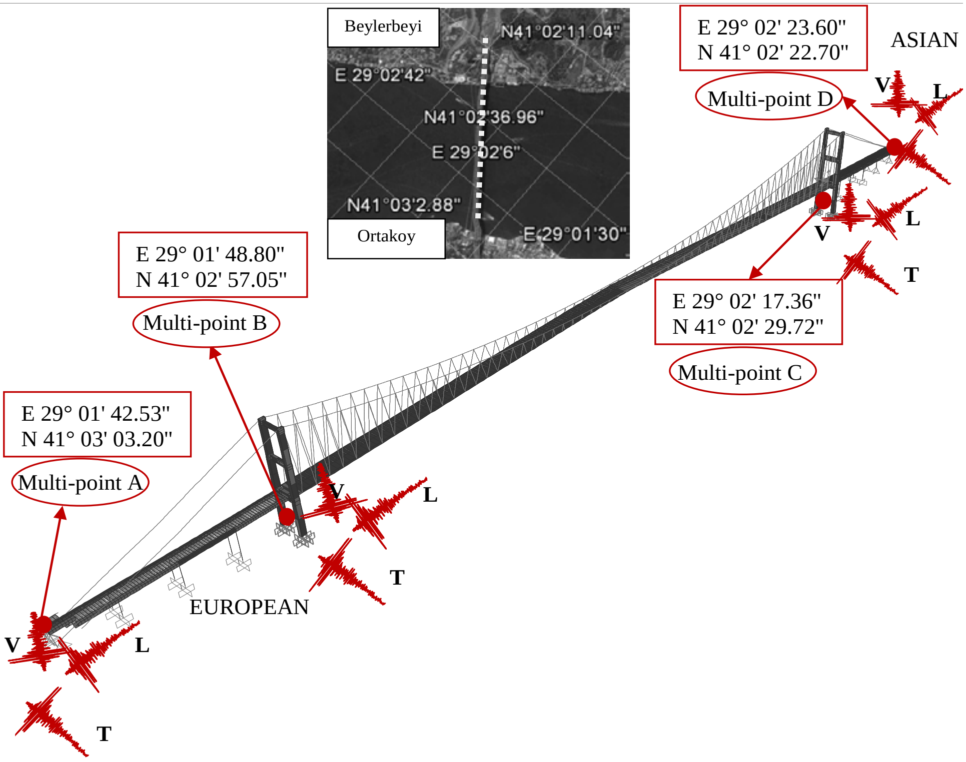


Fig. 9 Geographic coordinates and multi-support earthquake analysis considerations

Table 4 DD-1 design response spectrum parameters

DD-1 Earthquake Level Spectrum		
S_s	Short period spectral acceleration constant =	1.50
S_1	T = 1.0 s period spectral acceleration constant =	0.40
F_s	Local site effect coefficient for S_s =	1.00
F_1	Local site effect coefficient for S_1 =	1.00
γ_F	Near-fault constant =	1.20

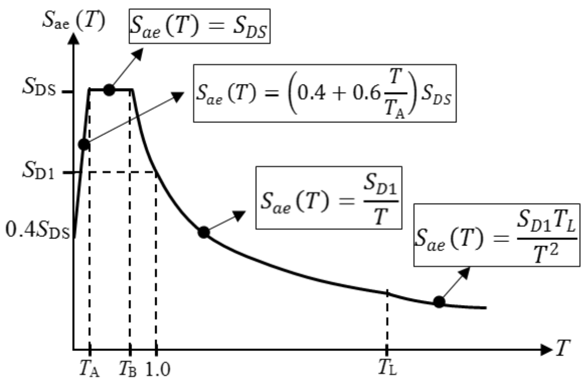


Fig. 10 General curve for response spectrums in TSC (2017)

to be spectrum-compatible earthquake records. For this aim,the DD-1 design spectrum given in the new Turkish Seismic Code (TSC 2017) as shown in Fig. 10 is considered with the parameters in Table 4. The DD-1 earthquake level with a 2% probability of exceedance in 50 years corresponding to return period of 2475 years is the maximum earthquake to be considered in earthquake resistant design of structures. Based on these considerations, the simulation process is performed and the acceleration ground motion time-histories (ATH) are generated for the Bosphorus Bridge.

Although the process yields to the ATHs, the displacement ground motion time histories (DTH) need to be obtained from the multi-point earthquake analysis. Therefore, the DTHs are presented in Fig. 10(a) instead of the ATHs.

As shown in Fig. 11(a), the triple-direction (two horizontals and one vertical) ground motions are generated for the each considered multi-point, A, B, C and D.

Total number of twelve ground motions was defined for the analysis. The process of base-line correction and detrending is also performed to obtain relatively precise displacement ground motions and thus results from the multi-point earthquake analysis. Besides, the uniform ground motions used in the study of Apaydin (2010) are also given in Fig. 11(b) with the inclusion of the multi-point earthquake motions so as to better understand the results from the *Mp-Sup* analysis. Apaydin (2010) is also utilized the FINSIM simulation for uniform ground motions due to the consideration of only one specific geographical coordinate close to the potential fault system of Istanbul. Also, the simulated earthquake records then are scaled according to the specific design spectrum of SEE that is the Safety Evaluation Earthquake Ground Motion associated with a 2% probability of occurrence in 50 years. In JBSI retrofit report, the real earthquake acceleration records are reported to be used instead of simulated and scaled earthquake records. Thus, the results from these studies can be considered to be obtained from the *U-sup*.

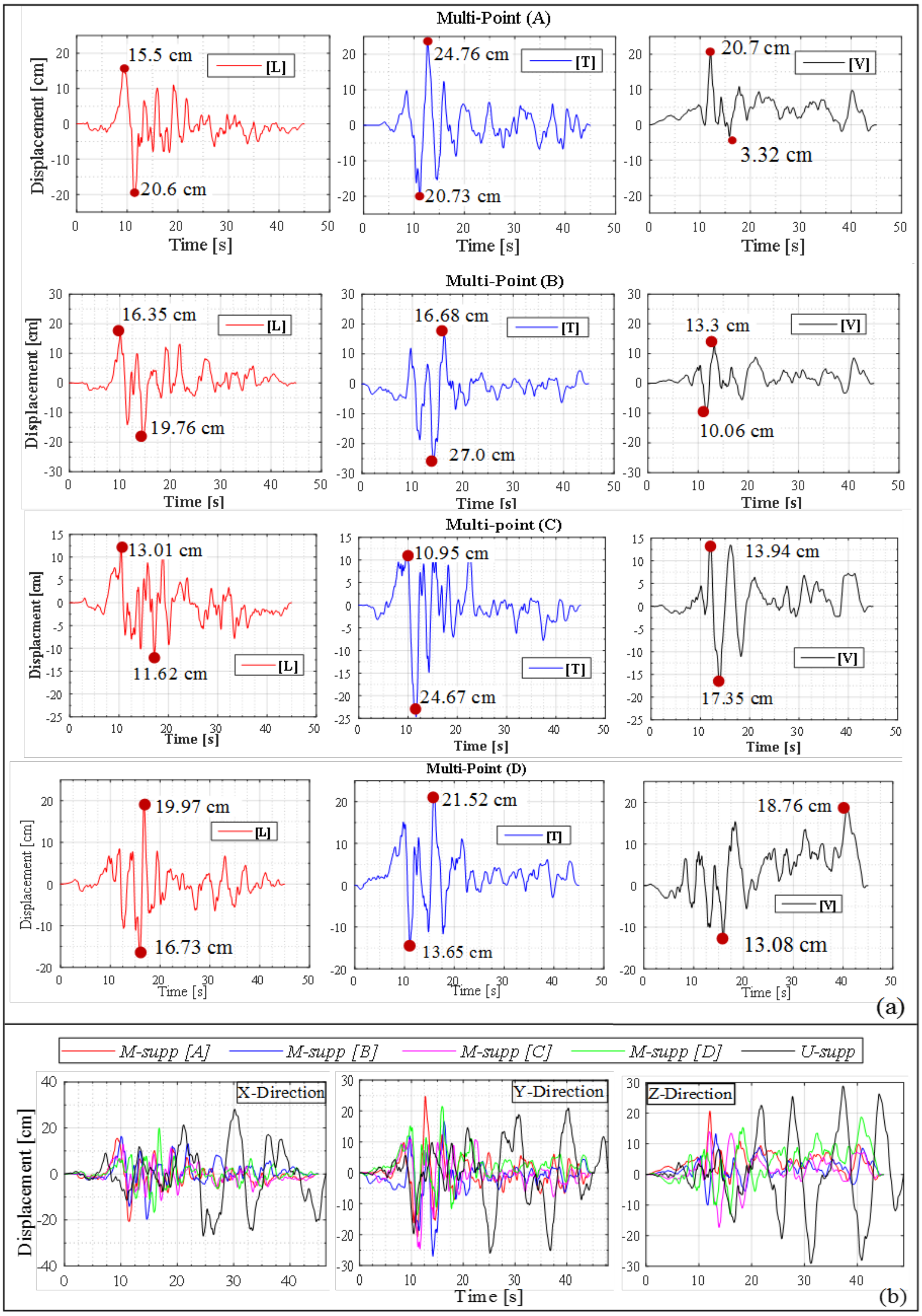


Fig. 11 (a) Spatially varying site-specific earthquake motions; and (b) Comparison with the uniform support e arthquake motions

6. Multi-point earthquake analysis (*Mp-Sup*)

Considering the produced spatially varying site-specific ground motions, the non-linear geometric time-history analysis of the bridge is performed. The multi-points of the bridge as shown in Fig. 9 are utilized to make the multi-point earthquake analysis (*Mp-sup*). For this aim, the practice-oriented procedure is developed as schematically summarized in Fig. 12. Although general assumption for the earthquake analysis is to utilize acceleration ground motion time-history (ATH), the developed *Mp-sup* requires use displacement ground motion time-history (DTH) instead of ATH. Based on this consideration, unit support displace-ment (d = 1.0 m) is separately defined in three directions for

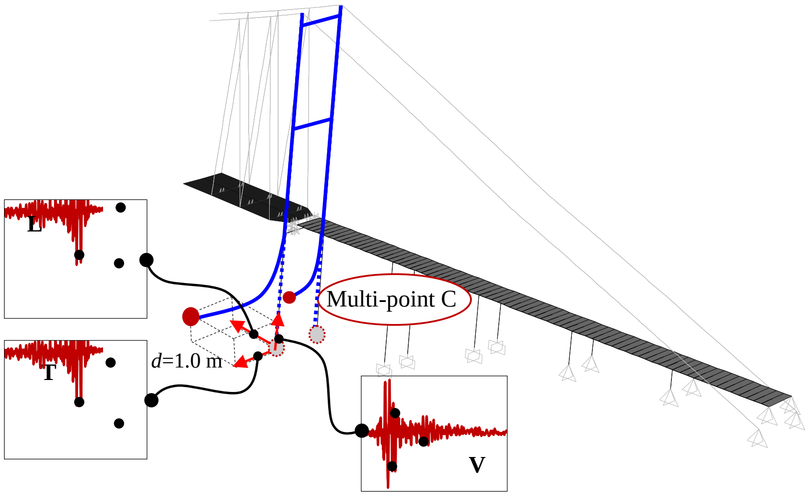


Fig. 12 Definition of the multi-point earthquake analysis (*Mp-sup*)

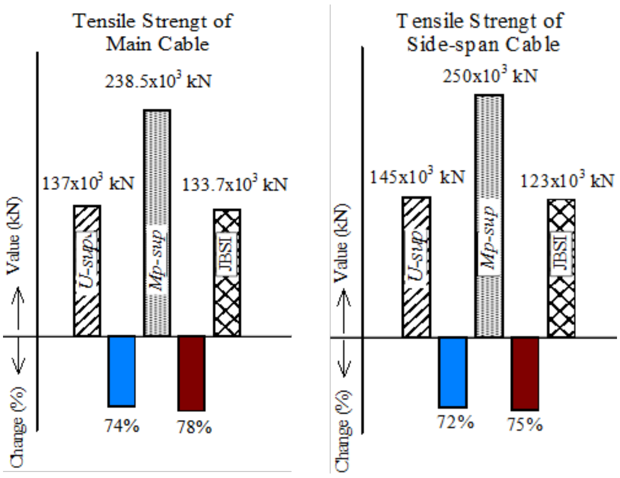


Fig. 13 Comparison of tensile strength of side-span and main cables

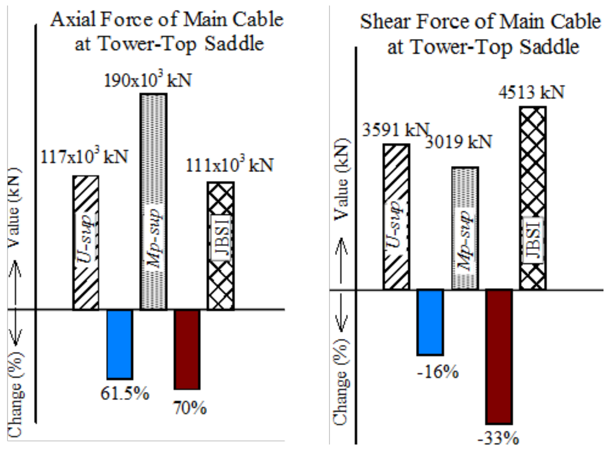


Fig. 14 Comparison of section force of main cable at tower-top saddle

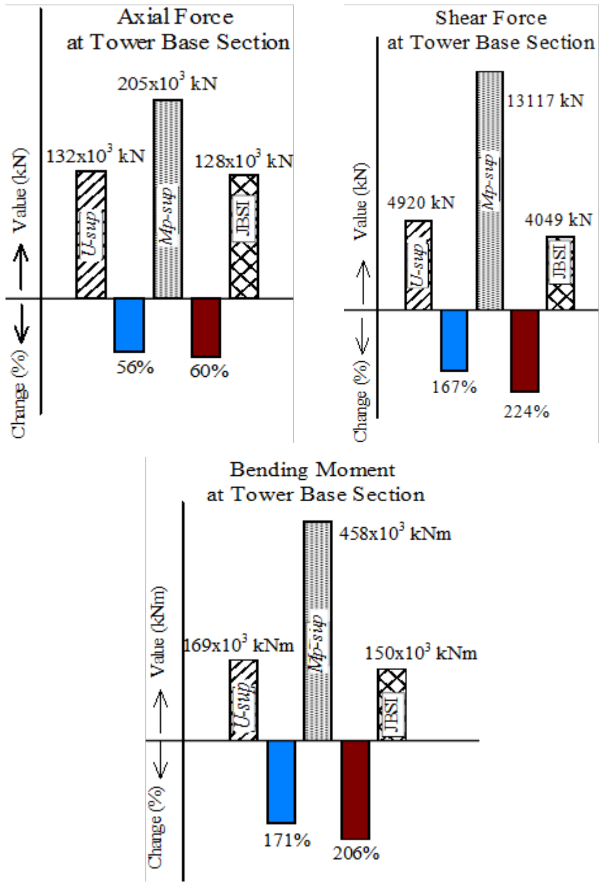


Fig. 15 Comparison of sectional force of tower base section

each multi-point (A, B, C and D). Accordingly, displacement of the bridge's support is provided to be changed in time in accordance with the DTH as shown in Fig. 12 for the multi-point C. After definition of this load case for each support of the bridge, total number of twelve separated time-history load cases are synchronically combined in one time-history load case. These steps can be easily implemented with the help of a structural analysis software, such as SAP2000 (CSI 2016), Open Sees etc.

Through the developed procedure, the multi-point earthquake analysis of the Bosphorus Bridge is performed and the results are given in Figs. 13-16 that present section force of the critical elements and displacement of the center

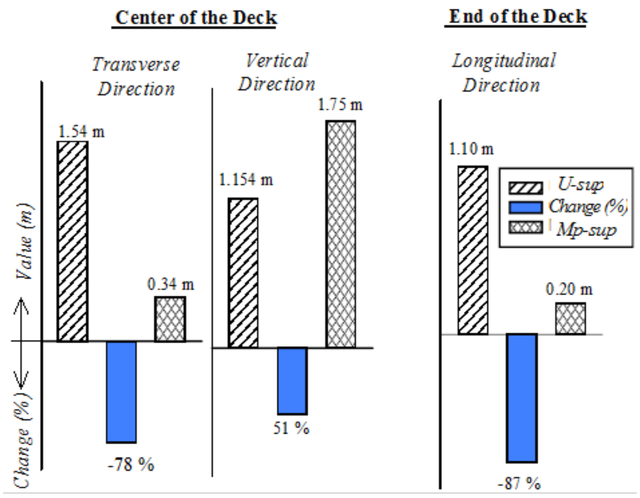


Fig. 16 Comparison of displacement of the deck

Table 5 Displacement of the tower top-saddle

Location	Max. displacement of tower top-saddle	
	Transverse (m)	Longitudinal (m)
European	0.578	0.214
Asian	-0.49	-0.184

and the end of the deck. In order to show the influence of the *Mp-sup*, the results from Apaydin (2010) and JBSI (2004) also given in the figures. Presented the outcomes of the *U-sup* of the Bosphorus Bridge, this study is called as the *U-sup* in the current study. The difference between the *Mp-sup* and *U-sup* for each defined section is considered as the key indicator for determining the *Mp-sup* effects. In addition to the displacement of the deck, the displacement at the each tower top-saddle is also obtained and given in Table 5 to show the relationship between the sectional forces and the displacements. Besides, modal response of the bridge is also pertained with the displacements and the sectional forces. For this purpose, the response acceleration data obtained at the critical points of the bridge under multi-point earthquake excitation is used and the frequency-domain analysis (FFT) is performed. All details and outcomes are indicated in Fig. 17.

In Fig. 13, the variation of the tensile strength of the main and the back stay cables is presented. The tensile strength value of the main cable increases as 74% and 78% under the *Mp-sup* compared to the *U-sup* and JBSI retrofit project, respectively. The different percentage change reveals that JBSI retrofit project is highly conservative in terms of sectional forces. Similar percentage change is obtained for the back-stay cable as shown in Fig. 13. Based on these results, high vertical displacement under the *Mp-sup* is expected to be at the center of the deck when compared to the *U-sup*. Fig. 16 shows expecting results that the vertical displacement at the center of the deck relatively increased as 51% under the *Mp-sup*. All these results demonstrate the importance of the behavior of the deck.

As seen from Fig. 14, the axial force of the main cable at the tower top-saddle also increases relatively. This increase is mostly related to the displacement of the deck and high increase in the tensile strength of the main and the back-stay cables. However, the decrease in the shear force of the main cable at the tower top-saddle is obtained. This decrease is depended on balancing effect of the deck at the deck-tower connection (expansion joints). The results exhibit again the efficiency of the deck.

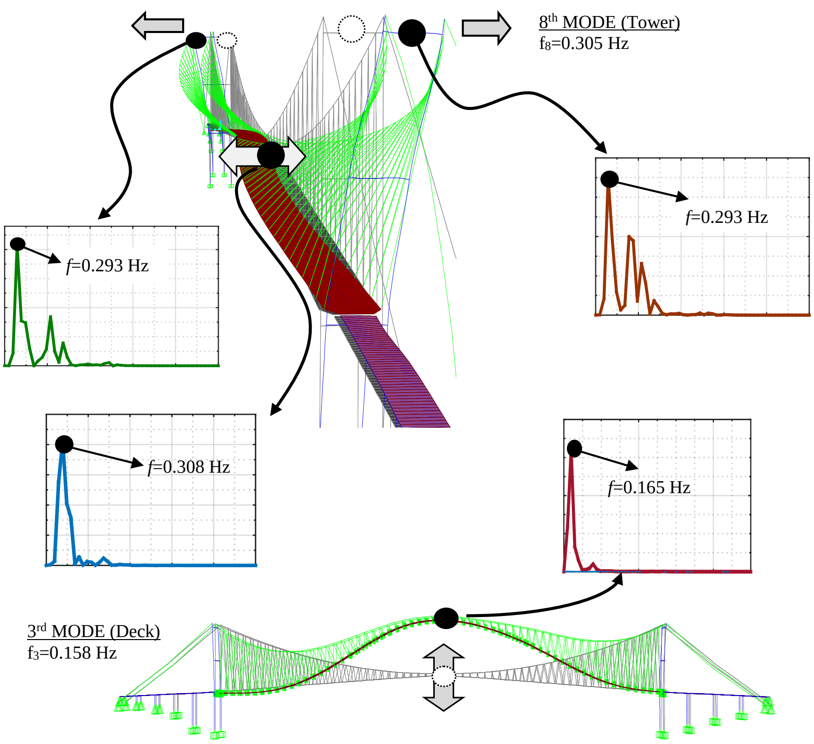


Fig. 17 Relationship between critical displacements and modal response of the bridge

Table 6 Comparison of the results from FFT with those from the modal analysis

Mode Number	Mode Shape	Frequency [Hz]		
		Modal analysis (f_m)	FFT	
			Point	(f_r)
Mode-3	1^{st} V_{sym} (Deck)	0.158	Deck mid-span	0.16
Mode-8	1^{st} L_{asym} (Tower)	0.305	Tower top saddle	0.29
			Deck mid-span	0.31

Another important point of the bridge is the base-section of the tower columns, which is first considered for the retrofit investigation for long-span bridges. The maximum value of the sectional forces including the axial force, the shear force and the bending moment is given in Fig. 15. Due to noticeably high increase in the tensile axial force of the main and the back-stay cables, the axial force of the tower directly increased as 56% and 60% according to the *U-sup* and JBSI retrofit project, respectively. Although the shear force of the main cable at the tower top-saddle decreases, the shear force of the tower at the base considerably increases since the deck forces the tower at the level of the expansion joints (tower-deck connections) leading to additional high shear force. Therefore, the bending moment of the tower at the base highly increases as similar percentage increase to that of the shear force as shown in Fig. 15. These conclusions reveal that the base section of the tower is mostly affected with the change in the sectional force of the critical points of the other elements and thus, that retrofit investigation should be made on the tower base section of long-span suspension bridge.

For retrofit investigation, tower part from the base to the deck level should be particularly considered and the sectional capacity of this part of the tower leg should be increased.

The change in the displacement at the critical points of the center and the end of the deck is also given in Fig. 16. In transverse direction, approximately 78% decrease is determined under the *Mp-sup* compared to the *U-sup*. Based on the relatively low displacement value of 0.34 m under the *Mp-sup*, it is expected that the towers moves oppositely in each other in transverse direction. For this aim, displacement at the tower top-saddle are obtained in transverse and longitudinal directions. These results are given in Table 5.

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All details and results are presented and summarized in Fig. 17 and Table 6, respectively. Another interesting outcome from the analysis is to obtain movement of 0.20 m in lateral direction at the ends of the deck. This result indicates that it is not enough to precisely assign the boundary conditions to the expansion joints of the bridge. Based on the result from the *Mp-sup*, it is most likely possible to be displacement in the operation of the bridge's suspended deck in lateral direction. Therefore, additional measures need to be taken under extreme events, especially under strong wind event.

7. Conclusions

All details and results are presented and summarized in Fig. 17 and Table 6, respectively. Another interesting outcome from the analysis is to obtain movement of 0.20 m in lateral direction at the ends of the deck. This result indicates that it is not enough to precisely assign the boundary conditions to the expansion joints of the bridge. Based on the result from the *Mp-sup*, it is most likely possible to be displacement in the operation of the bridge's suspended deck in lateral direction. Therefore, additional measures need to be taken under extreme events, especially under strong wind event.

From the consequences of the modal analysis, the deck and the tower of the bridge are estimated to be effective in the *Mp-sup* due to high total modal mass participation ratio of about 80% at the end of the first eight modes. Therefore, it is concluded that these components should be investigated in the *Mp-sup*. Since these elements directly influenced the cable elements, the main and side-span cables should also be assessed. The other interestingly important result is the absence of the mode shapes of the approach viaducts. This is based on the high rigidity of the approach viaduct decks along with the circular box-columns. Depending on the results, the side-span of long-span bridges could not be considered for the global analysis.

One of the most important step to perform the *Mp-sup* is to generate the spatially varying site-specific ground motions. As indicated in Fig. 9, geographic coordinates of the bridge are identified and considered for generation of the earthquake motion using FINSIM technique. In the simulation of the ground motions, the scenario earthquake of Mw = 7.4 with the probability of 70% in next 30 years, the parameters specified for the loss estimation in Istanbul and the seismicity of Istanbul are utilized. Thus, spatially varying site-specific displacement ground motions as shown in Fig. 11 are produced to define for the *Mp-sup*.

In order to easily implement the multi-support earthquake analysis to large-scale structures, the practice-oriented procedure is proposed. For this aim, unit displacement in triple directions as demonstrated in Fig. 12 is defined for each multi-point given in Fig. 9 and they are assigned to each corresponding simulated ground motion. Then, twelve time-history cases are combined into one time-history case. These steps could be easily achieved with a structural analysis software. Utilizing the ground motions specifically simulated for the bridge's supports, the *Mp-sup* is conducted through the procedure and the following points and conclusions are obtained in the study:

- Compared to the *U-sup* and JBSI retrofit project, the tensile strength of the main cable increased as 74% and 78% under the *Mp-sup*, respectively. As shown in Fig. 13, similar percentage change was obtained for the back-stay cable. These results are mostly related to the estimation of high increase of 51% in vertical displacement of the deck at the mid-span as indicated in Fig. 16. In addition, this outcome clearly revealed the significance of the response of the deck.

- As given in Fig. 14, the axial force of main cable at tower-top saddle considerably increased except for the shear force. The decrease in the shear force is based on the balancing effect at the expansion joints, which means that additional but reverse shear effect compared to that of the tower top-saddle leads to such decrease.
- Due to noticeably high increase in the tensile axial force of the main and the back-stay cables, the axial force of the tower base-section directly increased as 56% and 60% according to the *U-sup* and JBSI retrofit project, respectively. As indicated in Fig. 15, the shear force also considerably increased. Such increase in the shear force directly caused to increase in the bending moment of the tower base section as presented in Fig. 15. Due to the highest increase in the tower base section, special attention to the towers of the bridge is concluded to be given in the performance evaluation of the bridge. Particularly, tower leg part from the base to the suspended deck level should be strengthened to increase sectional capacity the tower section. For this retrofitting, stiffener elements, such as doubly symmetric steel I-beam along the axis of the tower leg part can be considered since easily located on interior surface of the tower with welding/bolt connection. This rehabilitation schema also provides additional increase in the buckling capacity of the tower along with the existing bracing ribs.
- As demonstrated in Fig. 16, 51% increase in vertical displacement was obtained at the center of the deck. Displacement in transvers and longitudinal directions; however, decreased as %78 and %87, respectively. Depending on the displacement at the tower top-saddle as given in Table 5, the reason for relatively small displacement in these directions is pertained to the opposite movement of the towers in each other. Such behavior of the bridge's towers also leads to relatively increase in the main and back stay cables.
- The opposite movements of the towers in transverse direction and also small displacement at the deck mid-span in transvers direction were considered to be noticeably pertinent to the mode shapes of the bridge. As given in Fig. 17, the frequency response obtained under the *Mp-sup* from response acceleration data at the tower and deck mid-span in transverse direction indicated considerably close relation to the 8th mode of the bridge, transverse tower mode. Similar outcome of relatively close relation between the 3rd mode and the dominant frequency of the acceleration data at the deck mid-span in vertical direction was also obtained.

Depending on the project specifications for the boundary conditions of both ends of the suspended deck, no movement in lateral direction has to be obtained. This restrain condition is provided with the rigid link element in the FE model of the bridge. The results from the *Mp-sup* analysis indicated the presence of lateral movement of 0.20 m at the both ends. The unexpected outcome is expected to be particularly more critical under extreme events, such as strong wind than operational condition of the bridge. Therefore, it is also stated in the study that certain measure should be taken. For instance, shear tongue can be placed on the both ends of the deck. Thus, movement ability of the deck in lateral direction can be limited. Nevertheless, the device should be inactive in operation of the bridge; in other words; should be only active under extreme events since the project boundary conditions of the bridge have to be provided.

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