SEISMIC PERFORMANCE ASSESSMENT OF ISTANBUL AIRPORT VIADUCT ACCORDING TO TURKISH BUILDING SEISMIC CODE 2007 BY PUSHOVER ANALYSIS

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Abstract
The goal of this study is to examine and evaluate the structural system and seismic performance of the viaduct which is located the west side of the Istanbul Airport. For this purpose, a three-dimensional structural system of the viaduct is modeled by using SAP2000 (Structural Analysis Program) software. The piers of the viaduct consist of two columns. It connects the terminal building and the parking building to each other, twelve spans and designed according to AASHTO (2002). Earthquake effects are considered in the transverse and longitudinal directions by taking into account the site-specific response spectra. The pre-stressed and precast girders which is used in the deck of the viaduct is designed under bending moment and shear force considering service loads, such as self-weight and traffic loads. The pre-stress losses and deflection checks are carried out to satisfy the serviceability conditions of the viaduct deck. The elements of the viaduct, such as pier cap, piers and foundation were designed by considering the vertical and lateral loads, so required cross-sections and reinforcement area are determined. In the present paper earthquake performance evaluation of the viaduct is accomplished according to the Turkish Building Seismic Code 2007 (TBSC 2007). Performance evaluation of the viaduct is carried out by applying static pushover analysis, one of the non-linear analysis methods, is applied to the pier located at the middle axis of the viaduct. Under lateral loading, the plastic hinges are developed at the bottom ends of the piers, as anticipated. In the performance evaluation of viaduct piers, the earthquake effect with 10% probability of exceedance over 50 years is considered adopting the parameters given TBSC 2007, whereas the parameters which define the spectrum compared to the Seismic Code of DLH (2007) code. Numerical results are given in tables and figures, comparatively.

Keywords: Viaduct design, seismic performance, static pushover analysis

1. Introduction
Engineering structures such as viaducts and bridges have an important role in the transportation and it is expected that they should continue to give service during and after the major earthquakes. To keep structural damages and economic losses limited, their earthquake design and their performance of viaducts and bridges are quite important. Lessons taken from past earthquakes have shown that, bridge or viaduct pier damages are quite common. Another damage reason is the fallen precast deck beams due to insufficient support length on the pier cap. During the earthquakes, depending on the soil conditions and foundation type, such as pile or surface foundation, if there is small rotation at the bottom end of the piers, the precast deck beams of the bridge are easily fallen down and bridge cannot give service anymore. So, it is important to keep the deck beams where supported on the pier caps. Usually one of the important parts of reinforced concrete bridges and viaducts are their columns (piers). Consequently, various research can be found on nonlinear dynamic behavior of the piers exposed to seismic loads (Fawaz and Murcia-Delso 2018, Papadopoulos, Murcia-Delso and Shing 2018).

Turkey is located in an active earthquake zone and have high seismic risk. Therefore, seismic design and performance of bridges and viaducts are of crucial importance. In the past, design of such kind of structures is fulfilled by considering force-based approach only. In recent years, economic and other practical considerations have led the engineering community gradually towards the performance-based design philosophy, which goes beyond the usual acceptance limits of usual strength and serviceability requirements in terms of seismic performance of structures (IQbal 2007). In terms of seismic performance and damage assessment, nonlinear analysis is used especially for important structures. To evaluate the seismic performance of structures, it is obvious that the nonlinear analysis is preferred. The static pushover analysis, which is one of the nonlinear static method is commonly used. Many factors affect the nonlinear behavior of a bridge and viaduct, such as material nonlinearity, geometric nonlinearity or second-order effects, nonlinear soil-foundation-structure interaction, gap opening and closing at hinges and abutment locations, time-dependent effects of concrete (creep and shrinkage), and so on (Chen and Duan 2014). The design and seismic performance of the viaduct located at Istanbul Airport is studied and numerical results are given in table and figures and discussed in detail.
2. Analysis and Design of the Viaduct According to AASHTO

2.1. Geometry and Structural Characteristics of the Viaduct

The viaduct in Istanbul Airport consists of a total of 12 normal spans and one console span. The span lengths of the viaduct vary in between 20.00m~32.00m and the length is mostly 32.00m. The deck of the viaduct is composed of pre-precast beams of 1.25m height with a slab of 0.25m thickness. The deck width varies between 14.50m and 36.00m. There is also an asphalt coating layer with a thickness of 0.06 m on the deck. However, the study in the scope of the present paper consider the column (pier) P6 consisting of two circular columns with a height of 21.40 m, located at 0 + 870.090km. In the viaduct, the pier columns have circular cross sections of 3.00m diameter. The viaduct piers are supported on a raft foundation having 2.50m thickness. Plan view, longitudinal and transverse section views of the viaduct and its relevant axes are given in Figure 1.

![Figure 1](image1.png)

2.2. Design Parameters

The structural system of the viaduct is modeled by using SAP2000 software (Figure 2) by assuming that piers are fixed at their bottom end, so the flexibility of the foundation is ignored. The top end of the piers are free to experiencing displacements x (longitudinal) and y (transverse) directions. The concrete material class of the viaduct is assumed to be C40 for pre-stressed precast girders and C30 for concrete piers, slabs and pier caps. Reinforcement classes are S420 for conventional reinforcement and S1860 for pre-stressed reinforcement. In addition to the weight of the viaduct, H20-S16 class of truck vehicle load, wind and earthquake loads are considered in the design of viaducts. The special design acceleration spectrum of the site is determined as a result of site specific investigations taking into consideration the ground conditions. The effective ground acceleration coefficient is found to be 0.27g and the spectrum characteristic periods, $T_A$ and $T_B$ are considered as 0.14s and 0.65s, respectively. The design spectrum of earthquakes with a probability of exceeding of 2% and 10% in years is presented in Figure 3. Within the scope of this study, in the force based design, the spectral acceleration values with a return period of 475 years are used. Earthquake forces are applied to the viaduct by taking into account the site-specific spectral acceleration for two directions. The earthquake load reduction coefficient for the piers is taken as 3 and 5, in longitudinal and transverse directions, respectively, according to AASHTO (2005).

2.3. Pre-stressed Beams

The pre-stressing is a technique that has been created due to the problem of not meeting the tensile stresses in large span structural members due to the low tensile strength of the concrete in reinforced concrete structures. Application of pre-stressing to the beams enables the structural engineers to design, slender beams especially in long spans. It is customary to use pre-stressed concrete beams to construct bridges and viaducts up to 30.00m~40.00m spans.

As a result of the capacity checks of the deck made of the pre-stressed beam, 30 strands are to be used. A total of 14 harped strands used in a pre-stressed girder and there are 16 active strands (Figure 4). Total pre-stressed losses are calculated in accordance with AASHTO (2002) by effecting pre-stressed beam inherent loads (beam weight, floor, asphalt, borders, railings, pedestrian loads, etc.) and vehicle load.
Figure 2. SAP2000 modeling of structural elements of the viaduct

Figure 3. Design spectrum

Figure 4. Section of pre-stressed girder which in the upper structure part (cm)

Stress analysis was carried out as the transfer and the service status of the pre-stressed girder depending on the manufacturing stage. The transfer status is the pre-stage application phase. At this stage, the pre-stressed precast girder only carries the weight of the beam. Stress losses are kept to be minimum and the stresses in the sections after the losses should satisfy the following limit values:

\[
\sigma_{\text{bottom}} = \frac{P_i}{A} \frac{P_e}{W_{\text{bot}}} + \frac{M_{G1}}{W_{\text{bot}}} \geq 0.6 f' \cdot \sigma_{\text{ci}} \quad (1)
\]

\[
\sigma_{\text{top}} = \frac{P_i}{A} \frac{P_e}{W_{\text{top}}} + \frac{M_{G1}}{W_{\text{top}}} \leq 0.623 \sqrt{f' \cdot \sigma_{\text{ci}}} \quad (2)
\]

where \(P_i\) is the total initial prestressing force involving the total number of strands in the section, \(M_{G1}\) is the bending moment of the beam weight and \(e\) represents the eccentricity. Also, \(W\) refers to the section modulus of the prefabricated beam with respect to the upper (\(W_{\text{top}}\)) and lower (\(W_{\text{bot}}\)) edge. Furthermore, \(A\) is the cross-sectional area of the precast section and \(f'_{\text{ci}}\) represents the concrete compressive strength at the prestressing stage.

Serviceability conditions should be satisfied in all loading cases, except the moving loads. It is accepted that the beams and slab weights are carried by precast beams and the additional loads are carried by the composite beams. When all stress losses considered, the sections should satisfy the following limit stress values given equations 3 and 4. In the following equation, the value of \(P_e\) is the force result of total loss addition to the first pre-stressed force:

\[
\sigma_{\text{bottom}} = \frac{P_i}{A} \frac{P_e}{W_{\text{bot}}} + \frac{(M_{G1} + M_{G22})}{W_{\text{bot}}} + \frac{(M_{G3} + M_{h})}{W_{\text{bot}}} \geq -0.4 f' \cdot \sigma_{\text{ci}} \quad (3)
\]

\[
\sigma_{\text{top}} = \frac{P_i}{A} \frac{P_e}{W_{\text{top}}} - \frac{(M_{G1} + M_{G22})}{W_{\text{top}}} - \frac{(M_{G3} + M_{h})}{W_{\text{top}}} \leq 0.498 \sqrt{f' \cdot \sigma_{\text{ci}}} \quad (4)
\]

where \(M_{G22}\) is the bending moment due to the deck weight, \(M_{G3}\) is the bending moment due to additional loads, \(M_{h}\) is the bending moment due to the moving loads and \(W_{c}\) corresponds to the section modulus of the composite beam relative to the lower and the upper edge of the precast-beam. Furthermore, \(f'_{\text{ci}}\) is the concrete compressive strength of the precast pre-stressed beam.

Table 1. Stress analysis table of pre-stressed prefabricated girder

| Strands low distance | Number of strands at 0.1L locations of the girder |
|----------------------|-------------------------------------------------|
| 1st row strand       | 6 cm 6 8 8 8 8 10 | 5th row strand | 30 cm 0 0 0 0 0 0 |
| 2nd row strand       | 12 cm 6 6 6 6 8 10 | 6th row strand | 24 cm 0 0 0 0 0 0 |
| 3rd row strand       | 18 cm 6 6 6 6 8 10 | Total number of strands in section | 16 20 20 24 24 30 |
| 4th row strand       | 24 cm 0 0 0 0 0 0 | Strand center of gravity (cm) | 11.25 11.40 11.40 12.00 12.00 12.00 |
| 5th row strand       | 30 cm 0 0 0 0 0 0 | Stress (\(\sigma_{\text{top}}\)), (N/mm²) | -13.53 -14.87 -12.47 -14.20 -13.17 -18.00 |
|                       |                    | Stress (\(\sigma_{\text{ci}}\)), (N/mm²) | 2.28 0.85 -1.49 -2.54 -3.55 -2.76 |
2.4. Design of Pier Cap and Piers

The piers of the viaduct are circular columns with a diameter of 3.00m as shown in Figure 5. There are two layers of reinforcement in the column and in the first and the second layers there are number of 72 and 36 reinforcement having a diameter of 32mm (72Ø32) and 26mm (36Ø26) reinforcements, respectively. The pier cap of the viaduct has a section with a height of 3.50m as shown in Figure 5. Structural system of the viaduct is modeled by SAP2000 and numerical analysis is carried out for the cases of free and forced vibrations. The modal parameters, such as vibration periods, mode shapes, mass participation ratios, elastomeric support displacements, internal forces and the displacements of the structural system are determined. Under all loading combinations, the most unfavorable cross-sectional effects P (axial force), M2 moment (bending moment in the longitudinal direction), and M3 moment (bending moment in the transverse direction) obtained from analysis of SAP2000. The moment of M2 and M3 are reduced by the corresponding Ra, seismic demand reduction factor. In addition, the pier section of the viaduct is modeled by using the XTRACT program (Figure 5). For the axial forces corresponding to the maximum axial force, maximum M2 and M3 bending moments, the capacity diagram of the piers was separately plotted. Reduced yield curve is obtained by decreasing the reduction factor found by making proportional 0.2 $f_c' A_g$ the limit value given in ASSHTO (2002) The $A_g$ value in the formula is the cross-sectional area of the pier. The pier design has been completed within this yield curve by the reduced moment values obtained by using the reduction factor $R$. An example of the capacity diagrams for the column design is shown in Figure 6.

![Figure 5. Modeling of the pier and the pier cap in the XTRACT program](image)

Each axis was examined separately for the design and in this study only P6 axis is considered. All elastomer bearings are defined as link element in the model (Figure 7). While the earthquake force is applied to P6 axis model, the most unfavorable V2-V3 shear forces were taken from the model. Capacity diagrams of section were created by using XTRACT. These diagrams are obtained using the P axial force corresponding to maximum M2 and M3 bending moments. Maximum moment capacity that obtained analysis report is increased by a factor of 1.3 according to AASHTO (2002). The capacity shear force obtained by using the increased bending moment. If the maximum shear force obtained from the model is smaller than the shear force obtained from the capacity, the value obtained from the model is used as earthquake effect. If the maximum shear force obtained from the model is greater than the shear force capacity, the value obtained from model is used as earthquake force for the pier cap. In addition, the pier cap is modeled according to the XTRACT software.

The moment curvature diagram obtained corresponding the axial force and bending moments taken from P6 axis model for the pier cap. The effective yield moment was obtained at 39770kNm according to the moment curvature diagram. The result of the analysis is reduced by a reduction coefficient of 0.9 at the moment at first yield value according to AASHTO. By using sectional properties of pier cap, $M_{capacity}$ was obtained 3150kNm. In addition, the bending moment value obtained from the model of SAP2000 was found to be 34748kNm. As a result, the resultant moment value from the P6 axis SAP2000 model is smaller than the moment value (bending moment at support and bending moment at mid span) obtained from the capacity, it means that the pier cap section and reinforcement is sufficient.

![Figure 6. Unreduced and reduce capacity interaction diagram](image)

![Figure 7. P6 axis model](image)
3. Performance Analysis

To ensure the serviceability of the structure; the most important point is to prevent the brittle collapse. This can be assessed by analyzing the structure nonlinearly (Çakıroğlu and Özer 1980). The purpose of the non-linear analysis is to determine the structural performance under earthquake effect and to evaluate plastic deformation demands. Nonlinearity can be attributed to multiple system properties, for example, materials, geometry, nonlinear loading and constraints (Kaya 2016). In this study, the static pushover analysis is applied to the piers in the P6 axis of the viaduct, only. In the nonlinear analysis used material models are given in Figure 8. The confined and unconfined concrete stress-strain variation diagrams (Mander et all, 1998) are given in Figure 8 where the stress-strain diagram of the reinforcement steel model is given as well. The sections of pier are modeled by using reinforced and confined concrete models.

The first two periods are obtained for the longitudinal and the transverse directions as 0.614s and 0.369s, respectively, where gross bending rigidity employed when the bending rigidity is reduced by taking into account the concrete cracking and nonlinear behavior of concrete and steel. The periods are found as 1.160s and 0.632s. In the numerical solutions concentrated plasticity (plastic hinge) assumption is used. Plastic rotations or plastic deformations are more effective in structural elements where the bending moment values are higher. In the plastic hinges, plastic rotations occur and the section continues to rotate without any increase in the moment. P-M2-M3 type of plastic hinge is used in the columns to define the interaction between normal force and bending moments. Additionally, while defining the plastic hinges, the moment-curvature variations obtained from XTRACT program is idealized and converted into a moment-rotation variation. Pushover analysis is accomplished in the two directions separately. Plastic hinges are defined at the lower ends of the column because the bridge piers exhibit cantilever behavior in the longitudinal direction, and they are defined at the lower and at the upper ends of the column as they exhibit frame behavior in the transverse direction.

The objective of static performance analysis is to determine the damage level of a structure that is generally subjected to a certain earthquake motion. This target is accepted as the minimum damage level for bridges and viaducts. The plastic hinges shown in Figure 9 are obtained as a result of the static pushover analysis with 0.50m initial displacement and Figure 9 shows the performance target corresponding to the plastic hinge formed. The reason for pushing 0.5 m initial displacement in the analysis is to determine the formation of plastic hinges as result of nonlinear analysis. According to the regulation, in order for non-linear behavior to occur, the initial displacement value can be determined greater than 0.02, which is the relative floor displacement limit (i.e. greater than 0.02 of pier height). Furthermore, since the system exhibits different behaviors in two directions, formation of plastic hinges is different in longitudinal and transverse directions.

![Figure 8](image1.png)

**Figure 8.** Confined and unconfined stress-strain diagrams of (a) concrete and (b) the reinforcement

![Figure 9](image2.png)

**Figure 9.** Plastic hinges formed in the a) longitudinal and b) transverse directions in the pushover analysis in the corresponding directions in the pier
Static pushover analysis was applied to the columns in the P6 axis with a 0.50m initial displacement under vertical loads. The pushover curve shows relation between the base shear force and the top displacement. In the longitudinal and transverse directions, pushover curves are given separately (Figure 10). The pushover curves are transformed into the modal capacity curve that is a relationship between the modal acceleration and the modal displacement. The modal acceleration and the modal displacement can be obtained by using the following equations (Figure 11).

\[ a_1^{(i)} = \frac{V_{x1}^{(i)}}{M_{x1}}, \quad d_1^{(i)} = \frac{U_{x1}^{(i)}}{\Phi_{x1} X_{x1}} \]  

(5)

where \(V_{x1}\) is the base shear force at the end of (i) th push step, \(M_{x1}\) refers to corresponding active mass, \(U_{x1}\) is the replacement of the first mode obtained modal displacement at the end of (i) the push step at the top of the building. Furthermore, \(\Phi_{x1}\) is the mode shape amplitude and \(\Gamma_{x1}\) is the modal contribution coefficient.

To obtain the performance point of the piers, the spectrum curve is converted to the same format, as it is done in Figure 12 from Figure 3. Performance analysis of the viaduct in the longitudinal (x direction) and in the transverse (y direction) directions are carried out, separately. The performance point is determined at the intersection points of the variations of the spectral acceleration displacement and the capacity curve. In this evaluation of the performance point the requirements of the Turkish Building Seismic Code 2007 (TBSC, 2007) is employed. The seismic demand limits (performance points) are obtained 0.160m and 0.080m in the longitudinal and in the transverse directions, respectively (Figure 13 and Figure 14). It is seen that the modal capacity curves intersect the demand curves in the linear range of the modal capacity curves. It yields that the design of piers is conservative, so the section of piers has sufficient capacity for earthquake demand.

![Figure 10](image.png)

**Figure 10.** a) Longitudinal and b) transverse direction pushover curves

![Figure 11](image.png)

**Figure 11.** Modal capacity curves in the a) longitudinal and b) transverse direction
Figure 12. Spectral acceleration-displacement curve (spectrum demand)

\[ S_{\text{aur}} = A_0 I S(T) g = 0.27 \times 1 \times 1.573 \times 9.81 = 4.17 \text{ m/s}^2 \]

\[ \omega = \frac{2\pi}{T} = \frac{2\pi}{1.160} = 5.416 \text{ rad/s} \]

\[ S_{\text{dl}} = \frac{4.17}{29.34} = 0.142 \text{ m} \]

\[ S_{a1} = C_{R1} S_{\text{dl}} = 1 \times 0.142 = 0.142 \text{ m} \]

\[ d_i = S_{a1} = 0.142 \text{ m} \]

\[ U_{x_i} = \Phi_{x_i} I x_1 d_i = 0.0420 \times 27 \times 0.142 = 0.160 \text{ m} \]

Figure 13. Evaluation of the performance point in the longitudinal direction

\[ S_{\text{aur}} = A_0 I S(T) g = 0.27 \times 1 \times 2.5 \times 9.81 = 6.62 \text{ m/s}^2 \]

\[ \omega = \frac{2\pi}{T} = \frac{2\pi}{1.160} = 9.942 \text{ rad/s} \]

\[ S_{\text{dl}} = \frac{6.62}{98.84} = 0.067 \text{ m} \]

\[ S_{a1} = C_{R1} S_{\text{dl}} = 1 \times 0.142 = 0.067 \text{ m} \]

\[ d_i = S_{a1} = 0.067 \text{ m} \]

\[ U_{y_i} = \Phi_{y_i} I y_1 d_i = 0.0392 \times 29.23 \times 0.067 = 0.080 \text{ m} \]

Figure 14. Evaluation of the performance point in the transverse direction

Table 2. Damage states of the pier at P6 axis.

| Column Size | Plastic Hinge Name | Plastic Rotation (\(\Theta_p\)) rad | \(L_p\) (m) | \(O_p/ L_p\) | P(kN) | \(\Theta_p\) (rad) | \(O\) (rad) | \(\varepsilon_{\text{cu}}\) | \(\varepsilon_s\) | Damage Condition |
|-------------|--------------------|---------------------------------|------------|--------------|-------|-----------------|------------|-----------------|-------------|-----------------|
| Longitudinal direction (x direction) | 0300 24H1 | 0.008043 | 2.007 | 0.00401 | -10385.29 | 0.001073 | 0.005081 | 0.0030 | 0.0114 | Minimum damage |
| 0300 25H1 | 0.008039 | 2.007 | 0.00401 | -10872.82 | 0.001077 | 0.005083 | 0.0030 | 0.0114 | Minimum damage |
| Transverse Direction (y direction) | 0300 24H2 | 0.001836 | 2.007 | 0.00091 | 2420.5 | 0.000960 | 0.001875 | 0.0011 | 0.0042 | Minimum damage |
| 0300 25H1 | 0.001952 | 2.007 | 0.00091 | -24122.53 | 0.001180 | 0.002153 | 0.0019 | 0.0042 | Minimum damage |
| 0300 25H2 | 0.001819 | 2.007 | 0.00091 | -20921.74 | 0.001155 | 0.002061 | 0.0018 | 0.0040 | Minimum damage |
The section rotation is calculated at the plastic hinges and the damage states were determined and they are found to be at the minimum damage limit (concrete strain 0.004, reinforcement strain 0.010) specified in the DLH (2007) and presented in Table 2. It means that, the seismic performance of the piers studied viaduct is satisfied by requirements of DLH (2007).

4. Conclusion
In this study, the design and the seismic performance assessment of the viaduct at Istanbul Airport were investigated. The numerical solution results of the viaduct under service loads and earthquake effect can be given as follows:

- As a result of calculations of the pre-stressed girder construction which is one of the structural elements of the viaduct, it is seen that the stresses satisfy the limit values given in AASHTO (2002). It was also found that the total stress losses in the pre-stressed beam did not exceed the estimated loss at the beginning of the design calculations.
- The amount of longitudinal reinforcement of the piers are found to be sufficient under the most unfavorable bending moment and the axial force.
- In the analysis the different seismic load reduction factor \( R_{\text{longitudinally}} = 3, R_{\text{transverse}} = 5 \) in the transverse and longitudinal directions is adopted. The other structural elements, sections of pier cap and amount of reinforcement used in force based design are sufficient with the performance evaluation is carried out.
- The plastic hinges are formed at the lower and upper end of the column (pier) in the transverse direction, while under longitudinal direction they developed only at the bottom end as they are expected. It is found that each column has a minimum damage limit according to the structural damage limit values given in the DLH (2007) code, when the section rotation and unit strain values for each column are examined. It reveals that the pier of the viaduct satisfies the expected performance targets.
- Although the performance based design approach is getting popular, the importance of strength based design will not lose its importance. The satisfaction of the performance requirements is directly related to strength based design.

References
AASHTO. (2002). Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials, Washington D.C
AASHTO LRFD. (2005). Bridge Design Specifications, American Association of State Highway and Transportation Officials, Washington D.C
Caltrans (2015). Bridge Design Practice, Precast Prestressed Concrete Girders (Chapter 8), Erişim: http://www.dot.ca.gov/des/techpubs/manuals/bridge-design-practice/page/bdp-8.pdf
Chen W. and Duan L. (2014). Bridge Engineering Handbook
Çakıroğlu, A & Özer, E. (1980). Nonlinear Systems in terms of Material and Geometry Exchange, Istanbul (Turkish)
DLH. (2007). Coastal and Harbor Structures, Railways, Airfields Construction Earthquake Technical Regulations, Ministry of Transport, Ankara (Turkish)
Fawaz, G. & Murcia-Delso, J. (2018). Finite Element Analysis of the Seismic Response of RC Columns with Modified Bond Properties, Eleventh U.S. National Conference on Earthquake Engineering, Los Angeles, and June 25-29
IGA. (2016). Istanbul Airport Car Park and Viaducts Outbound Passenger West Viaduct Account Report, Istanbul (Turkish)
Iqbal A. (2007), Performance Evaluation of Bolu Viaducts in terms of Performance Based Seismic Design of Bridges (Master's thesis). Bogazici University
Kaya, M. P. (2016). A Numerical Study on the Comparison of Linear and Nonlinear Methods in Determining Earthquake Performances of Reinforced Concrete Buildings (Master's thesis). Istanbul Technical University, Institute of Science and Technology, Istanbul (Turkish)
KGM. (2013). Technical Specifications of Highway, General Directorate of Highways, Ankara (Turkish)
Mander, J.B., Priestly, M.J.N., Park, R. (1988). Theoretical Stress-Strain Model for Confined Concrete, Journal of Structural Division (ASCE), 114(8), 1804-1826.
Papadopoulos, V. & Murcia-Delso, J. & Shing, P.B. (2018). Seismic Performance of Bridge Slab-Column Joints with Headed Reinforcement, Eleventh U.S. National Conference on Earthquake Engineering, Los Angeles, June 25-29
SAP 2000, (V.20.1.0), [Structural Analysis Program, Computers and Structures Inc.], Berkeley, California.
TBSC, (2007). Regulation on Buildings to be Constructed in Earthquake Areas, Ministry of Public Works and Settlement, Ankara (Turkish)
XTRACT. (V.3.0.8), [Cross Sectional Analysis Program for Structural Engineers, Imbsen and Associates Inc.], California