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FINITE ELEMENT MODELING AND DUZCE EARTHQUAKE SIMULATION OF BOLU VIADUCT

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SUMMARY

In this study, the effects of modeling techniques of large size viaducts are examined by forming different types of 3-dimensional and 2-dimensional virtual computer models in different complexity levels using Bolu Viaduct as an example, which experienced severe damage during November 12 Duzce Earthquake, located at 6 km south of the epicenter and fault line passing across the bridge. The models included shell and frame type members in different combinations for various levels of modeling complexity. Simplified 2D models are composed of frame type members only. The time history analyses resulted member internal forces at pier bases and drifts at pier caps, which are compared in order to understand the importance of third dimension for proper modal simulation. A sensitivity analysis is conducted in order to see the variation of results based on the model complexity. The results indicate that 2D models have shortcoming of modeling dynamic properties in third (transverse) dimension if lumped rotational masses are not included.

INTRODUCTION

Bolu Viaduct is a part of one of the largest highway projects of Turkey, and is still under construction. The region where Bolu Viaduct is located has experienced two major earthquakes; Duzce Earthquake occurred on November 12, 1999 with a Richter Magnitude of 7.2 and Kocaeli-Gölcük Earthquake occurred on August 17, 1999 and lasted 45 seconds with a Richter Magnitude of 7.4. The Bolu Viaduct was in construction stage during the two major earthquakes. Two dimensional (2D) models in elevation were developed by the contractor and used for the design considerations by numerous earthquake simulations. In this study, 3D computer modeling and 3D time history analyses of the Bolu Viaduct are carried out in order to see the effects of various modeling techniques on the behavior of the viaduct. 3D models are generated using varying levels of geometric complexity from simplified lumped mass and beam models to 2D-like frame-grid and detailed frame-shell models. The main objective of the study is to investigate

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effect of “modeling complexity” on the maximum pier member force and superstructure drift demand calculations. Simplified two dimensional (2D) models or modeling segments of a bridge might be misleading especially if the bridge is not symmetric and significantly long in length. The interactions between spans become very complicated especially at curved portions of the bridge. 2D models might fail to simulate inertial mass of the superstructure in longitudinal torsion direction.

VIADUCT DESCRIPTION

The bridge is curved in plan, has a vertical slope of approximately 4% towards Istanbul direction, and consists of two parallel bridges carrying traffic in opposite directions (Figure 1). Left and right bridges have 59 and 58 spans, respectively, which are composed of simply supported spans with expansion joints at every 10 spans. Each span is 39.2 m long. The total length of the bridge is about 2.3 kilometers. City of Bolu is located in the north-central part of Turkey which is at close proximity to the Düzce Earthquake epicenter. The fault line of Duzce EQ passed below the viaduct (between piers 45-47) at an angle of 20-30 degrees with the viaduct axis.

The bridge has tall reinforced concrete piers up to 49 meters high which are rectangular hollow in cross section (close to 8m x 4.5m x 0.6m). Piers are numbered starting from Istanbul side towards Ankara direction. All piers rest on massive and monolithic column footings supported on 12 cast-in-drilled hole friction piles each with diameter of 1.8 m. The depth of the piles ranges from 20 m to 30 m. The typical size of the footings is 18.7 m x 16 m x 3 m.

The superstructure is composed of seven prefabricated prestressed girders connected by a continuous cast-in-place slab of 0.24 m thickness over the girders and 0.54 m thickness between the girders. The superstructure is connected to piers using Energy Dissipating Units (EDU) to minimize possible EQ forces by providing elasto-plastic connections.

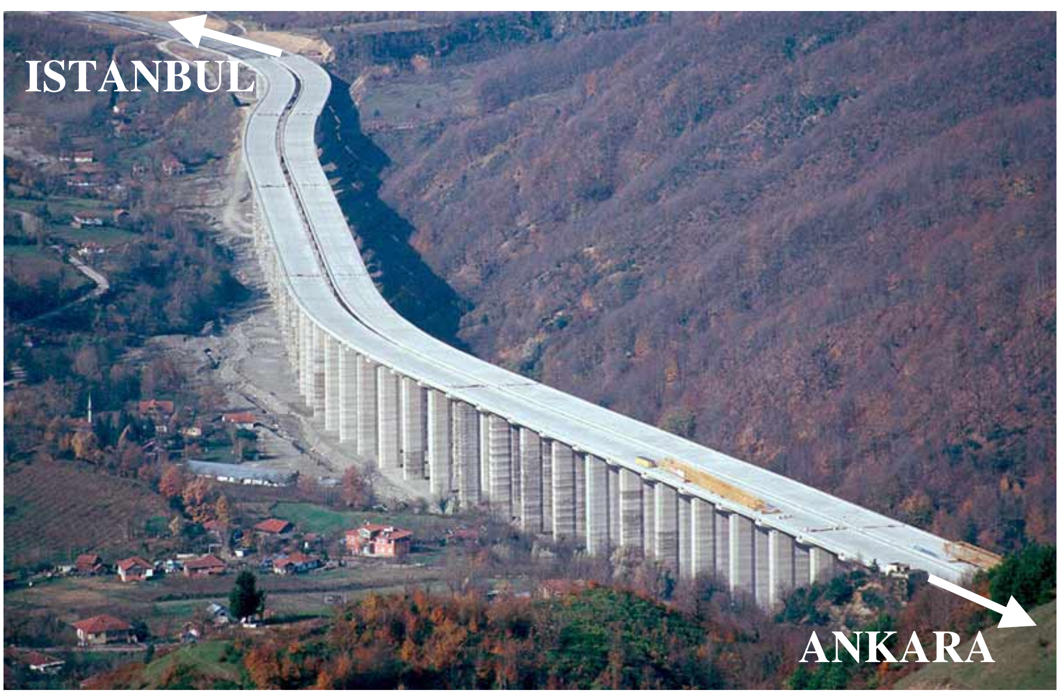


Figure 1 General view of the Bolu viaduct

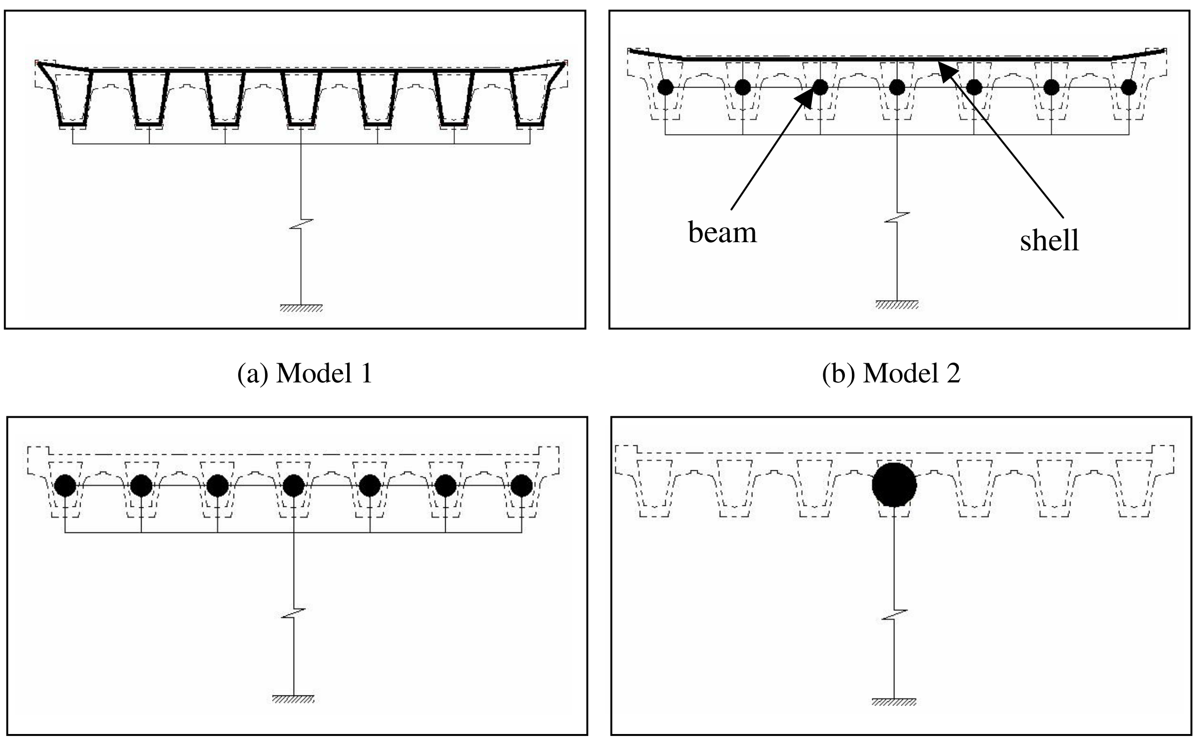
MODELING OF THE VIADUCT

For analytical studies, only the right bridge is modeled since both bridges are similar. Six different models are generated with varying levels of complexity (from 6 degrees of freedom (dof) simple model up to 140,000 dof finite element model). MS-Excel and SAP2000 programs are used for model generation and analysis. Combination of shell and frame type of elements is used in the models. The short descriptions of the models are given below:

- “Most Complex Model” (Left out of the analysis)
- Model 1, “3D Multiple Box Deck Model” (Figure 2a) about 140 thousand dof
- Model 2, “3D Flat Deck Model” (Figure 2b) about 98 thousand dof
- Model 3, “3D Frame Grid Model” (Figure 2c) about 34 thousand dof
- Model 4, “3D Lumped Beam Model” (Figure 2d) about 7,500 dof
- Model 5, “2D Lumped Beam Model” about 7,500 dof
- Model 6, “2D 10-Span Segment Model” 726 dof

A seventh model which is just a crude overview of the 10-segments of the bridge is also constructed to see how close the model is to represent the actual 3D behavior of the bridge.

- Model 7, “Lollipop Model” 6 dof



(a) Model 1 (b) Model 2

(c) Model 3 (d) Model 4, 5, 6 & 7

Figure 2 Modeling types

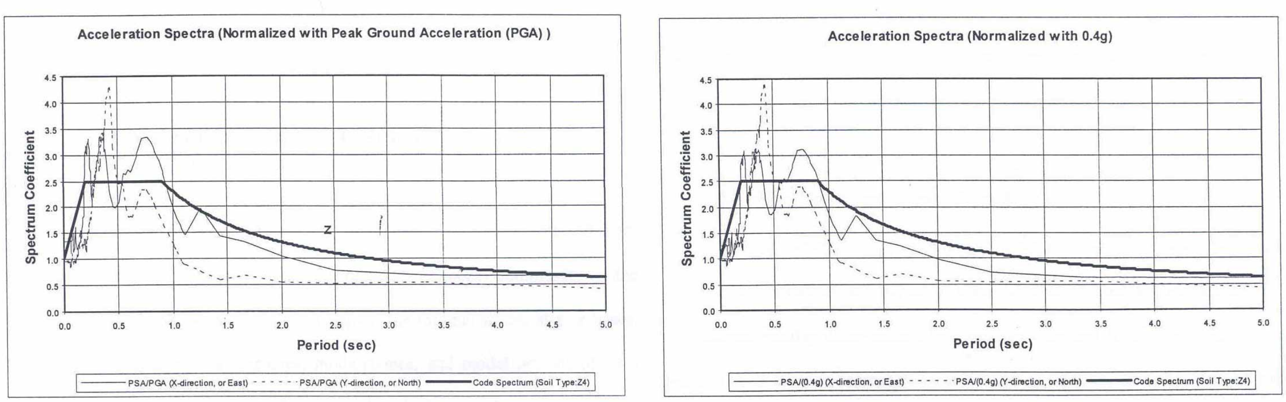
For the first three models, the pier cap beam and piers are modeled using beam elements. The superstructure is composed of shell elements only for Model 1, combination of shell and beam elements for Model 2. In third model, the deck and girders are modeled using frame elements only in longitudinal and transverse directions forming a mesh of frames. The transverse members' section properties are defined to approximately provide equivalent stiffness of the actual structure in transverse direction.

Model 5 is similar to Model 4 as shown in Figure 2d, however the curved nature of the bridge in plan is ignored in Model 5. The bridge is still modeled in 3D space but in the form of a straight line to ignore the curvature as 2D models do. Only 10 spans of the 58 spans are used in Model 6. The 10-span model symbolizes the portion of the bridge between two expansion joints, which is a commonly used 'idealized case'. The idealization is generally used to reduce total number of dof especially for nonlinear analyses. The analytical model used by the contractor was similar to Model 6. Model 7 is a hypothetical model which has very limited linkage to the real geometry. All 10 spans are assumed to be lumped at a single point for a very crude analysis.

DYNAMIC ANALYSIS

Eigenvector analysis yields many local modes that have no relationship to the overall global modes of the viaduct. Ritz-vector Analysis is selected for its robustness in finding modes in accordance to the spatial distribution of the dynamic loading. Ritz vectors are load dependent and such dynamic analyses based on a special set of Ritz vectors yield more accurate results than the use of the same number of natural mode shapes. A damping ratio of 5% is used for all analytical modes. Time history analysis is carried out to see affects of measured November 12, 1999 Duzce EQ and make a comparison between results obtained from different models. A total of 200 modes are used for all analytical models, generating total participating mass values very close to 100% in all directions.

Response spectrum curves in both East (X) and North (Y) directions are obtained using the measured time history data. The magnitudes are normalized using (a) peak ground acceleration and (b) 0.4 times gravitational acceleration (0.4*g) in Figure 3a and 3b, respectively, to have a general idea of the variations from the spectrum curve formed from the criteria in Turkish Seismic Code.



(a) PGA normalized (b) 0.4g normalized

Figure 3 Response spectra curves used for dynamic analyses

COMPARISON AND DISCUSSION OF RESULTS

In the following sections, the internal forces at pier bases, maximum displacements at pier caps, mode shapes, and modal periods of each bridge model analysis are presented, compared against each other, and discussed. The internal forces used for comparison are the shear and bending moments in longitudinal and transverse directions. Likewise, maximum horizontal displacements are compared in longitudinal and transverse directions.

There are two cases for each model: Case A and Case B. In Case A, the central piers for 10-span segments are fixed and all other piers are free to move in longitudinal direction. However, this case does not reflect the actual behavior of the bridge in case of an earthquake. There are "Lock-up Devices" on each pier cap and these devices restrain the movement of the piers during a strong excitation like in the case of an earthquake. Therefore, all models were run by taking this feature into account, and this case is called as Case B.

Mode Shapes and Modal Periods

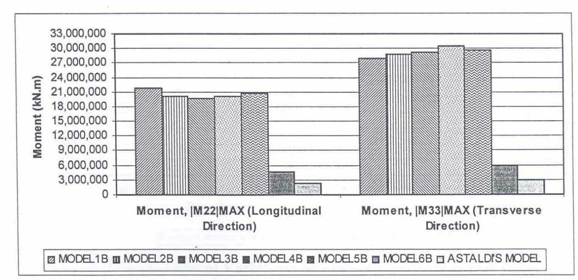
For Case B, modals periods for first longitudinal mode are found as 1.24, 1.19, 1.10, 1.05, 1.05 and 1.23 seconds for Models 1, 2, 3, 4, 5 and 6, respectively. Modals periods for first transverse mode are also calculated and found as 1.11, 1.10, 1.09, 1.08, 1.08 and 1.04 seconds for Models 1, 2, 3, 4, 5 and 6, respectively. As a result of these values, it can be generalized that finite element models become more flexible when finer meshing and larger number of members are used for a structure. However, the differences in the values of periods of Model 6 from the other models are due to the fact that it is only a model of 10-span segment and it cannot simulate the behavior of the whole bridge since a fraction of the whole length is considered.

As a result of modal analyses, it can be concluded that modal periods of Model 5 are generally same or greater than those of Model 4, which does not comply with the previous generalization. However, the complexity of Model 4 over Model 5 is not due to finer element meshing, but due to 2D – 3D geometry effect. Both Model 4 and Model 5 are composed of frame elements for the deck (lumped beam model), but the latter one is a straight 2D model and ignores the curves in plan.

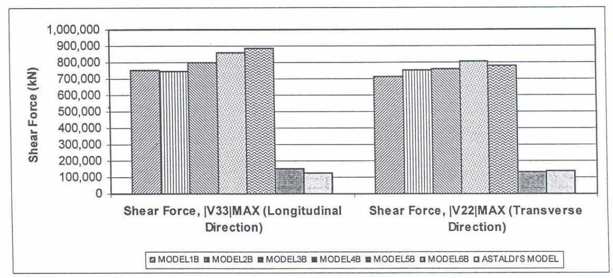
Internal Forces

For very short piers like Pier 4, 5, 6, 56, and 57, large peak moment and shear values are found. This is an expected situation; because, usually short members are stiffer compared to long ones and attracts more forces.

If we look at the sum of the forces at piers rather than individual central piers, a smooth increase or decrease cannot be seen in longitudinal direction. Although there is an increase in longitudinal shear force values as complexity of the modeling decreases, there are both decreasing and increasing patterns for the moment case. This may happen in certain cases since moment also depends on the moment arm. If the shear value increases while the member height decreases (at a larger scale) the moment might also decrease. Therefore, increase in shear together with decrease in moment is possible and logical (Figure 4).

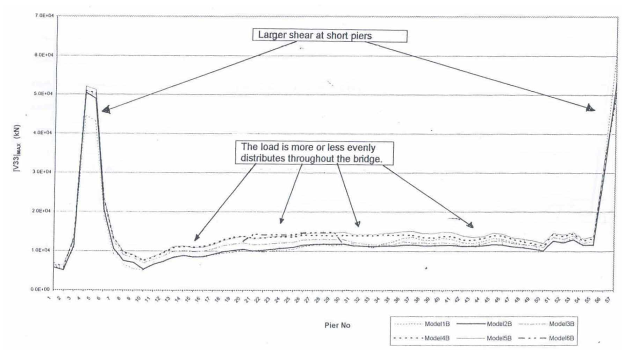


(a) Moment

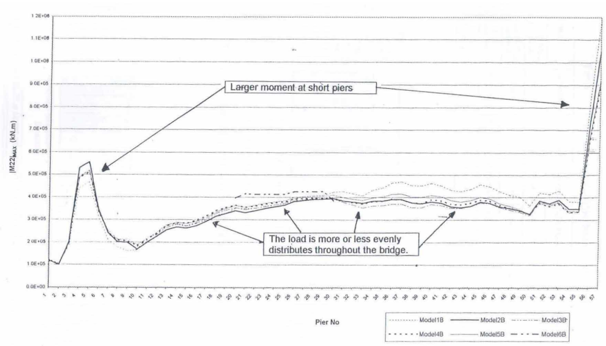


(b) Shear

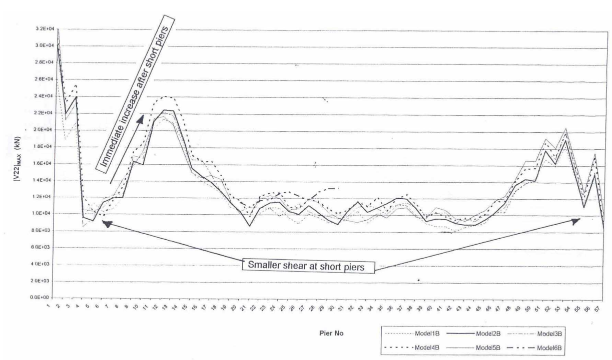
Figure 4 Sum of Internal Forces at Pier Bases (CASE B)



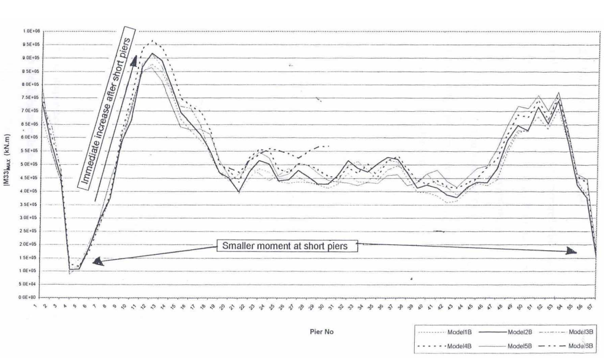
(a) Shear Force in Longitudinal Direction



(b) Moment in Longitudinal Direction



(c) Shear Force in Transverse Direction



(d) Moment in Transverse Direction

Figure 5 Maximum Absolute Internal Forces

As seen in Figure 5c and 5d, all generated models give closer shear force and moments. However, there is a big amount of decrease in transverse shear and moment in Pier 4, 5, 6, 56, and 57, which is not an expected situation. Usually, short members are stiffer and attract more shear force compared to long ones. Possible explanation of small shear is due to the fact that natural periods of such stiff structures are outside the frequency content of the seismic excitation. After this point on, shear and moment values immediately increase.

A lollipop like model is formed in order to make a reality check and compare against all models constructed so far. This can be done only for Case A, since in Case B all piers are transferring shear in longitudinal direction. In Case A, all the loads gather in the central fixed pier. In Table 1, a comparison is shown between Model 1A to 6A and Lollipop Model.

Table 1 Comparison od the Internal Forces in Longitudinal Direction with Simple Lollipop Model

MODEL	$\|V33\|_{MAX}$ (Long.) (kN)	$\|M22\|_{MAX}$ (Long.) (kN.m)
Lollipop Model (Pier 20-30 lumped at Pier 25)	39,139	1,911,690
Model 1A - Pier 25	37,891	1,747,111
Model 2A - Pier 25	35,817	1,583,994
Model 3A - Pier 25	32,890	1,388,359
Model 4A - Pier 25	25,273	900,538
Model 5A - Pier 25	26,765	955,738
Model 6A - Pier 25	26,776	956,113

As it can be seen from Table 1, the values of shear forces and moments in longitudinal direction calculated for Lollipop Model are close to Model 1A. However, it should not be forgotten that all the dimensional features are ignored in Lollipop Model. For example, moments coming from the width of the deck are ignored and it is assumed that the whole mass is on top of the pier. Although Model 6A is more close to Lollipop Model when simplicity is concerned, it cannot be said that Lollipop Model completely represents the correct behavior of a bridge in case of an earthquake.

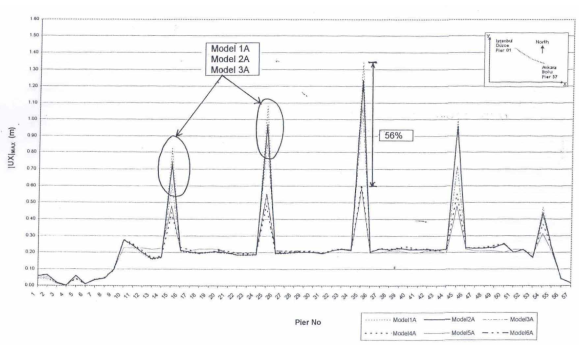
Displacements

In Case A, it can be said that first three models (Model 1A, 2A, and 3A) give larger displacements at central fixed piers than the other models in both directions (global X-direction (East) and global Y-direction (North)) (Figure 6a & 6b). This is more clearly seen in global X-direction, because global X-direction is close to the longitudinal direction of the bridge. (There is approximately an angle of 31 degrees between longitudinal direction of the bridge and global X-direction).

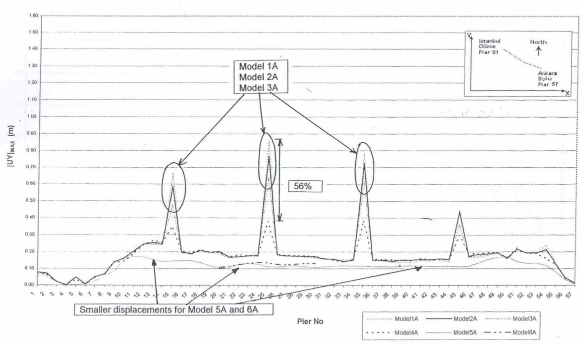
The fact that larger displacements occur in first three models means that the third dimension (width factor) affects the displacement values. About 56% difference in both directions is observed (Figure 6a & 6b). In Figure 7, maximum displacements at central fixed pier caps are calculated by using the square root of sum of squares of displacement values at each time step in both directions. It is observed that large differences occur at the Piers 15. 25, 35, and 45, which are high central piers; and small differences occur at the Piers 5 and 54, which are short central piers.

The displacements for Model 5A and Model 6A in global Y-direction (North) are smaller than values of the other models (Figure 6b), since the orientation of the bridge in Model 5 and Model 6 is not same as in the other models. In order to make the models 2 dimensional, approximately an angle of 31 degrees from global X-direction is ignored in these models; and this leads the difference in the displacement values.

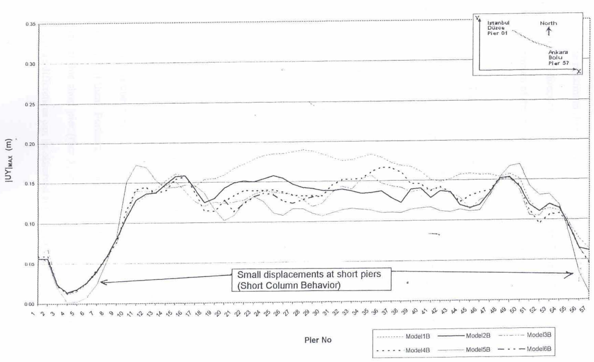
In Case B, there are no peak displacement values, since there are no specific central fixed piers. The deformation is more or less evenly distributes throughout the bridge (Figure 6c & 6d). On the other hand, for Piers 4, 5, 6, 56, and 57, which are very short compared to other piers, there are small displacement values for both in global X- and Y-directions. The decrease in the vicinity of Pier 5 demonstrates the general short column behavior very well (Figure 6c & 6d).



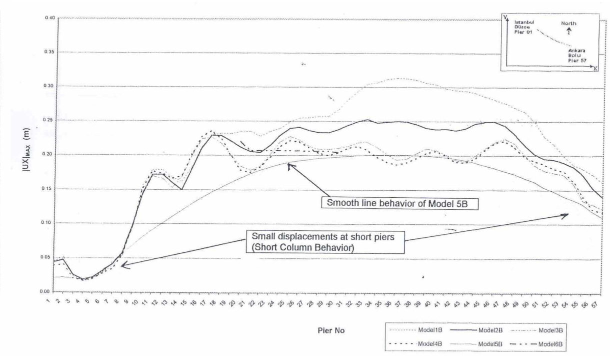
(a) In East Direction (Case A)



(b) In North Direction (Case A)

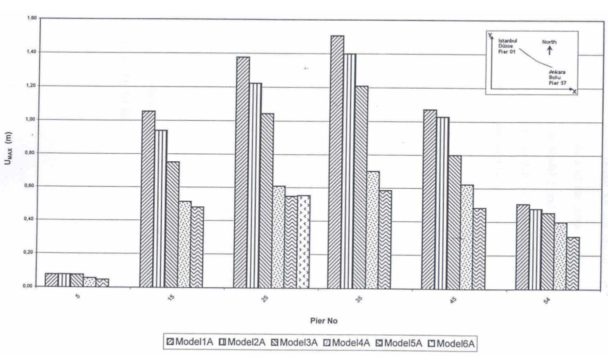


(c) In East Direction (Case B)

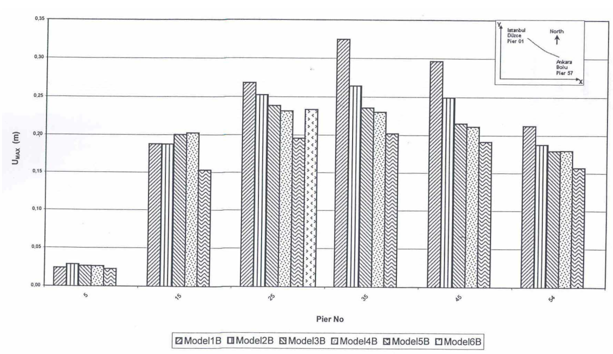


(d) In North Direction (Case B)

Figure 6 Absolute Maximum Displacements at Pier Caps



(a) Case A



(b) Case B

Figure 7 Modified Maximum Displacements at Selected Pier Caps

For the absolute maximum displacements at pier caps in global X-direction, it is interesting that Model 5 gives a smooth line and Model 3B and 4B are almost equal (Figure 6c). Model 1B makes large deviations in the middle portions of the whole bridge. Model 2B behaves as an average of Model 1B and Model 3B (or Model 4B).

The maximum displacements in both directions (global X-direction (East) and global Y-direction (North)) is summarized in Table 2. As it is seen in this table, maximum of the maximum displacements is in Model 1B.

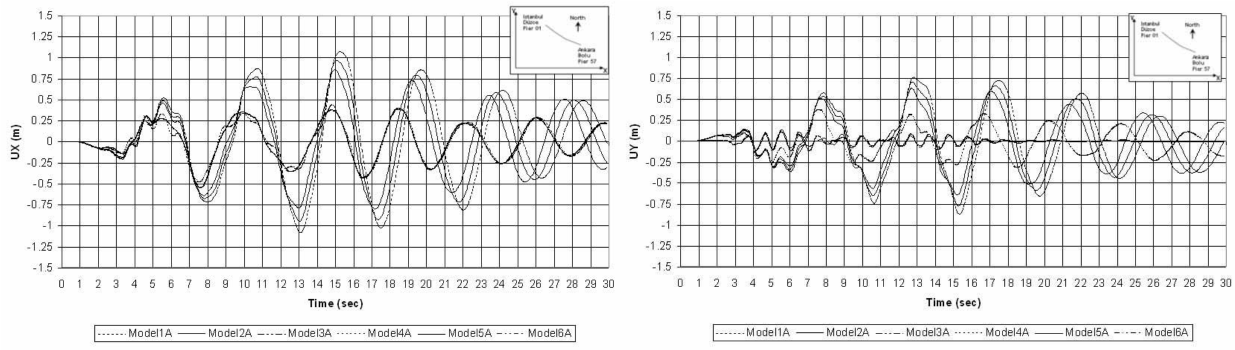
Table 2 Maximum Displacements for Case B

MODEL	$\lvert UX \rvert_{MAX}$ (m) (Global X-direction, or East)	$\lvert UY \rvert_{MAX}$ (m) (Global Y-direction, or North)
Model 1B	0.3136	0.1898
Model 2B	0.2528	0.1586
Model 3B	0.2275	0.1586
Model 4B	0.2357	0.1670
Model 5B	0.2011	0.1723
Model 6B	0.2206	0.1361

Note: There is an angle of approximately 31 degrees between the longitudinal direction of the bridge and global X-direction (East).

In Figure 7, maximum displacements at the caps of selected piers are calculated by using the square root of sum of squares of displacement values at each time step in both directions. It is observed that the smallest displacements occur at Pier 5, which is the shortest of them. Furthermore, the modified "maximum displacements for all models at this short pier (Pier 5) are slightly different (almost same). However, some large differences can be observed at the other high piers (Figure 7).

Pier 25 is selected as an example for comparison of whole time history traces of pier cap displacements and results for Case A are shown in Figure 8. There are significant differences between the time history traces of groups of models for Case A. It is observed that Model 1A, 2A, and 3A have similar behavior. Similarly, Model 4A, 5A, and 6A have also similar responses. The differences and similarities are mainly due to modeling techniques used for those models. The first thee models have third dimensional effect whereas others are mostly wire frame models. This result highlights the importance of fully considering third dimensional shape in modeling. On the other hand, for Case B, this grouping is not observed; however, it can be said that the behaviors of all models in Case B are similar.



(a) In East Direction (b) In North Direction

Figure 8 Displacements at Pier Caps of Pier 25 (a central fixed pier) (Case A)

CONCLUSIONS

Bolu Viaduct survived August 17, 1999 Kocaeli-Gölcük Earthquake (M_w=7.4, 107 km to the epicenter) without a major damage. However the 2.4 km long bridge underwent an extensive damage during November 12, 1999 Düzce Earthquake (M_w=7.2, 6 km south of the epicenter) the fault line passing through the bridge at an angle of 20-30 degrees with the viaduct longitudinal axis. Four 3D and two 2D analytical models were constructed using the geometric properties of the Bolu Viaduct. The effects of different modeling complexity levels on the linear response of constructed analytical models are investigated using the recorded seismic data at Duzce station.

The maximum displacements at pier cap locations are obtained for all piers after the time history analysis. The maximum of all maximum pier displacements are determined for each models and compared in Table 2. The largest displacement calculated is 0.3136 meters for Model 1B, which belongs to the most finely meshed (refined) model having the closest geometrical representation of the viaduct (see Figure 2a). Usually, refined finite element meshing causes models to converge towards a more flexible state. Maximum displacements calculated for each model become larger as the model complexity increases. This concludes that displacements increase as the complexity of the modeling increases.

The “most complex” model, which was excluded from the analysis, had a very large number of degrees of freedom and was not suitable for solving using current PC technology. Moreover, as the model size gets larger and depending on the solution method, numerical errors due to the round-off and matrix inversion would normally build up causing unacceptable errors in the results.

Although the shear, moment, displacement, mode shapes, and periods obtained from all models are different at most two times, there is a general grouping of models 1,2,3 and 4,5,6 based on their response to earthquake excitation. The major differences between these two groups come from modeling of the third dimension in transverse direction. It is concluded that the wire frame models (i.e. Models 4, 5, and 6) has shortcomings to simulate the dynamic behavior of deck rotation and deck rotational inertia in transverse bending modes, which is probably the most important conclusion of this study. Simplified 2D models have to include the lumped rotational mass for rotational components of modes in transverse direction. The two dimensional models might closely represent the stiffness and translational mass but commonly ignore the rotational mass which leads to errors in structural analysis results especially in transverse bending and torsional modes.

Modeling segments of a bridge, especially between expansion joints, is a practical simplification to reduce the total number of degrees of freedom. Modeling and analysis of 10-continuous-spans at a time (named as a segment) should be repeated for all segments of the bridge that has different pier height and/or configurations. The interaction between segments would be in the order of 10% to 20% of the total force acting on a segment, since each expansion joint resides on a single pier and segments do interact through EDU-pier-EDU chain (at both sides of each intermediate segment). Bridges that are curved in plan are expected to have more complicated interactions at the expansion joint connections.

Time history analyses using non-linear models will be performed as a future study. The non-linear behavior of Energy Dissipating Units (EDU) would greatly affect the seismic response of the bridge. A full non-linear analysis of complex models may not be feasible; however, semi-linear hybrid models (i.e. linear bridge + non-linear EDU) can also be used. The time lag between seismic waves reaching each pier might also be significant. Additional studies can also be conducted using different earthquake records to see and compare response and behavior of linear and/or non-linear models to specific EQ records. Comparison between the linear time history and response spectrum curves analyses results will also be studied.

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REFERENCES

1. Government of Republic of Turkey, Ministry of Public Works and Settlement, "*Specification for Structures to be Built in Disaster Areas*", Issued on September 2, 1997 (Official Gazette No: 23098), Amended on July 2, 1998 (Official Gazette No: 23390).

2. Roussis, P. C., "*Assessment of Performance of Bolu Viaduct in the 1999 Düzce Earthquake*", Department of Civil, Structural & Environmental Engineering, University of Buffalo.

3. Turker, O., *"Finite Element Modeling and Earthquake Simulation of a Highway Bridge Using Measured Earthquake Data"*, Middle East Technical University, 2003.

4. Xiao, Y., Yaprak, T.T., Tokgozoglu F., "*Observations of the November 12, 1999, Düzce Earthquake*".

5. Yılmaz, C., "*Paper: Design Philosophy and Criteria For Viaduct No:1 (Bolu Viaduct)*", Middle East Technical University, Ankara.