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SEISMIC PERFORMANCE OF PRECAST SEGMENTAL BRIDGES

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ABSTRACT

Precast segmental bridge construction is currently receiving increasing attention in North America and Europe mainly due to the advantages of accelerated construction and high quality control it offers. Despite these advantages, concerns have arisen amongst the engineering design community regarding the performance of such systems under intense earthquake shaking. In this paper, a novel segmental concrete bridge system, which is going to be used for shake table and quasi-static testing, is presented as well as an approach to efficiently model segmental systems using 2-node elements widely available is most structural analysis software. The results of the analyses conducted for the proposed bridge system using this modeling approach suggest that segmental bridge systems may exhibit high ductility and enhanced self-centering capabilities under severe ground excitation.

Introduction

Precast segmental concrete bridge construction was first introduced in Europe in the 1950s, whereas the first application of this type of construction in the United States was the John F. Kennedy Memorial Causeway in Corpus Christi, Texas in 1972. Since then, the number of applications of precast segmental bridge systems has increased substantially both in the United States and around the world mainly due to the advantages that precast segmental construction offers over the traditional cast-in-place techniques. These advantages include: (i) higher construction quality, since the segments are constructed in a shop under well-controlled quality conditions, and (ii) rapid construction, since, as soon as the segments have been carried to the construction site, only assembly and preparation of the joint connections is required. Assembly is usually achieved by internal grouting or external tendons, while, at the joints, epoxy adhesive high-strength materials or male-female indentation connections are utilized.

Despite the apparent advantages of the precast segmental construction, concerns have

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arisen amongst the engineering design community regarding the performance of such structural systems in regions of moderate to high seismicity. These concerns, which have limited the applications of this construction technique only to low seismicity areas, primarily refer to: (i) the effects of significant joint opening and/or sliding between adjacent segments during strong earthquake shaking on the global stability of the structural system, and (ii) the reliability of existing analysis tools in predicting the dynamic response of segmental bridge systems under tri-axial earthquake excitation, which is necessary in the context of performance-based design.

This paper presents the partial results of a study investigating the seismic performance of precast segmental concrete bridges both experimentally and analytically/numerically. As far as the experimental part is concerned, a large-scale bridge specimen intended to be used for quasi-static and shake table testing is described, whereas, in the context of the analytical investigation, an approach to model efficiently segmental systems using 2-node elements is illustrated.

Test Specimen: Precast Segmental Concrete Bridge

General Description

The prototype bridge structure on which the design of the test specimen is based is a five span single cell box girder bridge considered by Megally et al. (2002). Each span of the prototype bridge is post-tensioned with a harped shaped tendon. The test specimen considered in this study is a large scale (~1:2.4) single-span (referring to the mid-span of the prototype system) segmental concrete bridge with both of its supports overhanging at equal lengths, as shown in Figure 1. Following the principles of the "Accelerated Bridge Construction" (ABC) technique, the deck of the bridge specimen consists of 8 hollow segments of trapezoidal cross-section which are post-tensioned together using 10 to 14 internal unbonded tendons, whereas each pier consists of 5 segments of hollow square cross-section that are post-tensioned together by 8 internal unbonded tendons. Further information on the preliminary design of the specimen superstructure may be found in Anagnostopoulou (2009). The geometric properties of the superstructure segments and the pier segments are shown in Figure 2 and Figure 3, respectively.

Although use of internal unbonded tendons has never been considered in any of the existing segmental bridges and has never been reported in the literature for segmental bridge superstructures, preliminary studies show that they may result in better system performance, since local tendon yielding and rupture, which could result in global system instability, is avoided because deformation is distributed over larger tendon lengths. Consequently, tendon yielding occurs at larger system deformations providing the bridge system with greater ductility capacity and enhanced self-centering capabilities. Similar findings have been reported for bridge segmental piers by Ou (2007). The segment-to-segment joints for both the deck and the piers are simple friction-type connections defined by direct contact of adjacent segments. Thus, shear resistance at the joints is provided only by the friction generated between concrete segment end surfaces in contact, and the "dowel effect" of the tendons. Similarly to the unbonded tendons, use of friction-type joints has never been reported in the literature for bridge superstructures and has never been used in any of the existing bridges; however, this connection type performed adequately in segmental bridge piers (Ou 2007).

Another novelty incorporated in this study is the use of gap restrainers in order to distribute the gap opening over the length of segmental members. Theoretical investigation has shown that as soon as gap opening occurs at a joint, and as long as the applied member force distribution does not change, no gap opening (or no significant gap) opening is highly likely to develop in the adjacent sections. The same studies suggest that the gap opening of a joint has to be restrained in order to allow gap opening at the adjacent joints. For this reason, gap restrainers are currently under design for the piers, which are the most critical components for the stability of the bridge system. Distribution of gap opening at segmental piers has already been attempted by energy dissipation bars (Ou 2007). However, although the energy dissipation provided by the member increased, insignificant gap opening distribution was achieved.

To facilitate support of the deck on the piers, a cap beam of trapezoidal solid shape presented in Figure 4 is placed on top of each pier. At the two ends of each cap beam, stoppers preventing lateral sliding of the deck are installed, while sliding of the deck in the longitudinal direction is prevented by stoppers attached to the bottom of the deck segment located on top of the cap beam. Anchorage of each pier on one of the two adjacent relocatable shake tables of the Structural Engineering and Earthquake Simulation Laboratory (SEESL) of the University at Buffalo, where shake table and quasi-testing will be conducted, is achieved by a foundation concrete block mounted on each shake table, as shown in Figure 1. The foundation block, the cap beam and the 5 pier segments are all post-tensioned together by the same tendons, as illustrated in Figure 5. In the same figure, the stoppers against lateral sliding are presented, whereas the stoppers for the longitudinal sliding are shown in Figure 6, which presents a lateral elevation view of the bridge specimen.

The bridge specimen was designed as if it was a monolithic system according to AASHTO LRFD Bridge Design Specifications (2007), while its design was also assisted by the PCI Bridge Design Manual (2003). The performance objectives stated in these codes of practice were properly adjusted to abide by the concepts and principles of the ABC system. For example, the requirement for crack opening control under service loads was extended to include joint opening control under the same loading scenario. To determine the seismic loads, the prototype bridge on which the bridge specimen is based was assumed to be located at a site in the Western United States. The seismic loads were properly scaled in order to be consistent with the assumptions of the similitude analysis.

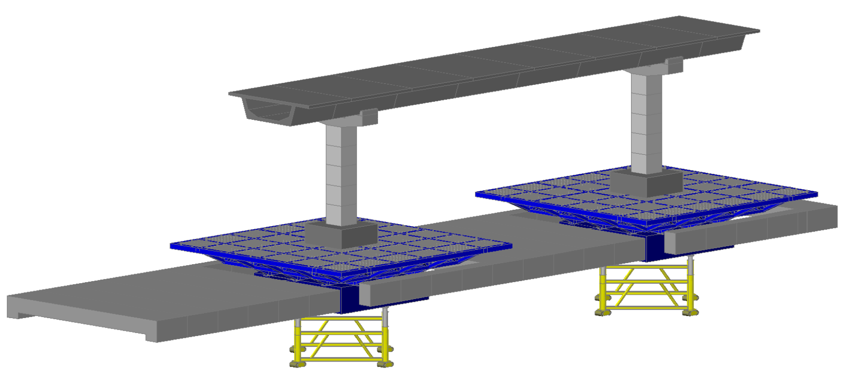

Figure 1. Precast segmental concrete bridge specimen on the two adjacent relocatable shake tables of Structural Engineering and Earthquake Simulation Laboratory (SEESL) at University at Buffalo (UB)

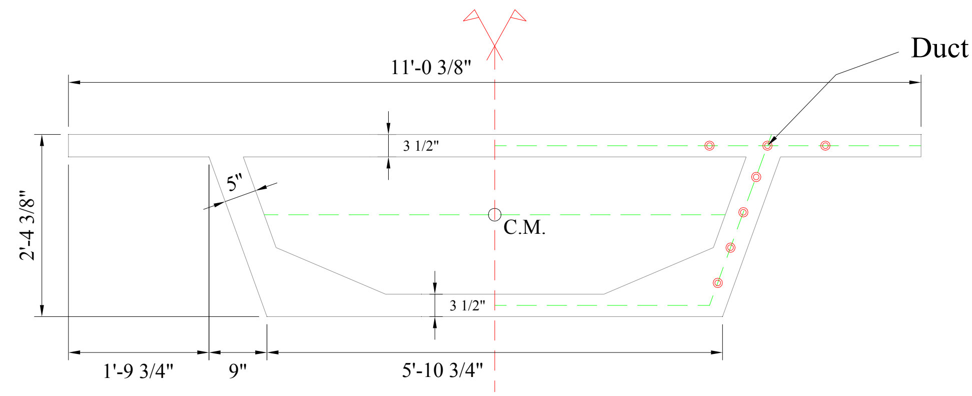

Figure 2. Superstructure cross-section

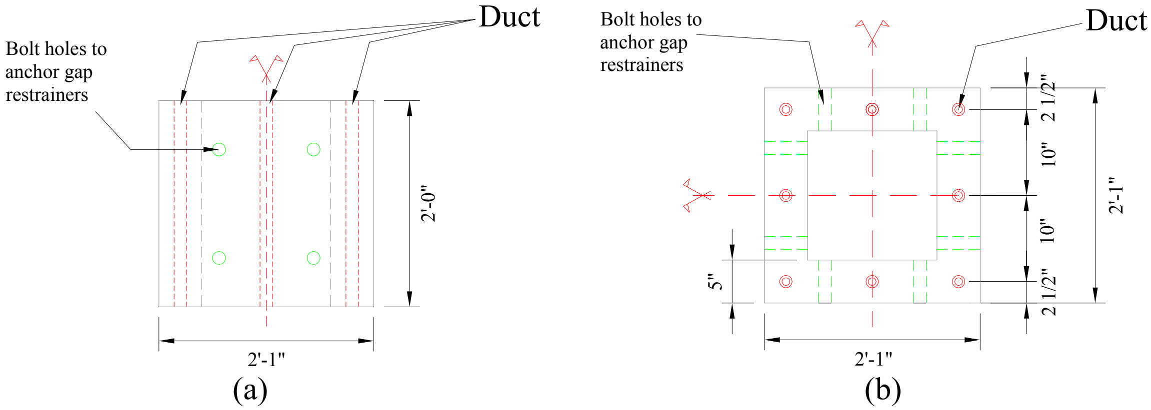

Figure 3. Pier segment: (a) Elevation view, (b) Cross-section plan view

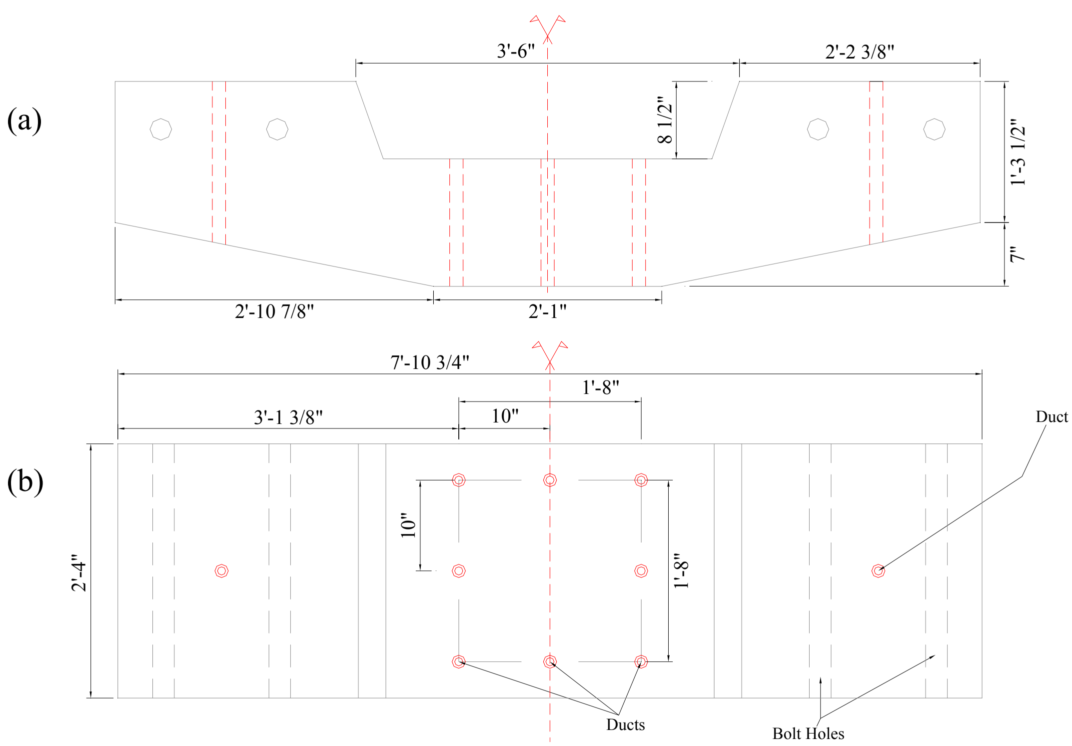

Figure 4. Cap beam: (a) Elevation view, (b) Plan view

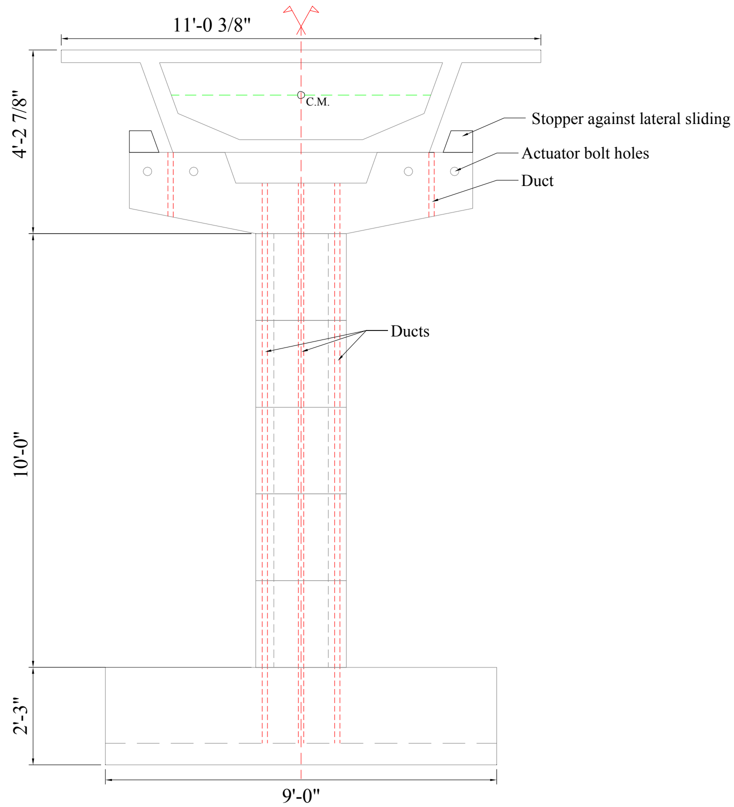

Figure 5. Bridge specimen longitudinal elevation view

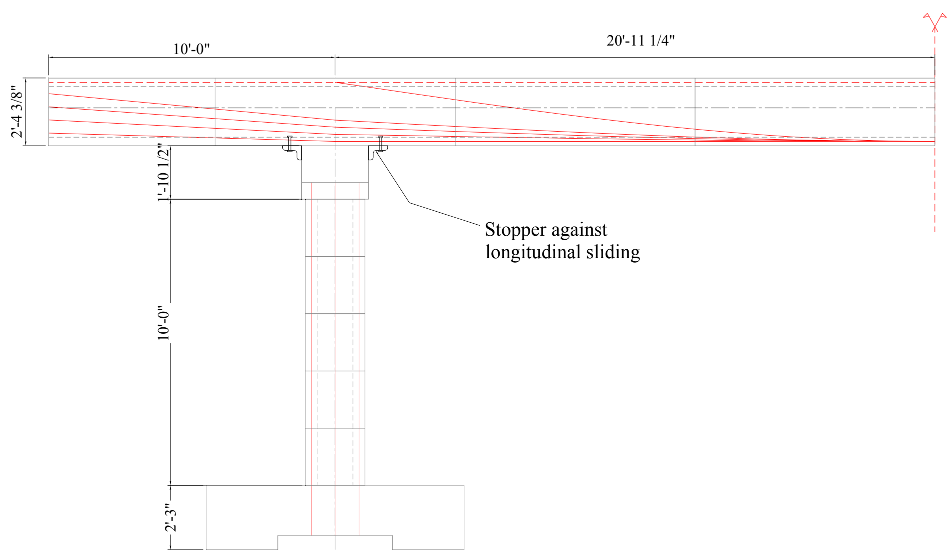

Figure 6. Bridge specimen lateral elevation view

Analytical/Numerical Investigation using 2-node Elements

Proposed Modeling Approach

The response of precast segmental systems under static and/or dynamic loading is significantly different from the response of conventional monolithic systems, since they potentially experience joint opening and sliding at the segment-to-segment interface. Although finite element analysis may efficiently predict the response of such non-conventional systems (Ou 2007, Aref et al. 2008), it requires large computational resources, extensive time, and use of complicated analysis software, which discourage practicing engineers from using it. On the other hand, widely available commercial structural analysis software, which mainly incorporate two-node elements or two-node macro-elements to predict the response of conventional structural members (i.e. beams and columns), seem incapable of capturing the main characteristics of the response of segmental systems.

In the present study, an approach to consistently model the response of segmental systems using typical nonlinear two-node beam-column elements and special connection elements available in most structural analysis programs is presented, and utilized to model the bridge specimen described earlier. In the proposed approach, segments are modeled as beam-column elements except for a region at the ends of each segment which is discretized into several 2-node axial (or axially-dominated) "fiber" springs. The "fiber" springs, which are independent from each other, are connected to the ends of the corresponding beam-column element through rigid body constraints/links. In order to ensure consistent behavior of the segment super-element (beam-column element and fiber springs), the number (minimum) and appropriate distribution of the "fiber" springs over a cross-section is recommended to be selected such that the exact second moments of inertia of the cross-section, I_y and I_z, and the approximate values of the associated integrals based on the selected discretization deviate from each other by less than 1%. The approximation of these integrals is shown below:

$$I_y = \int_A y^2 dA \approx \sum_{i=1}^{N} y_i^2 A_i \quad and \qquad I_z = \int_A z^2 dA \approx \sum_{i=1}^{N} z_i^2 A_i \tag{1}$$

where A_i is the area of the i-th fiber spring, and (y_i, z_i) is its location on the cross-section. If non-uniform material is considered, this concept should be extended to the cross-section moduli EA, EI_y and EI_z.

The nonlinear properties of the "fiber" springs are selected so that the segment super-elements demonstrate consistent axial and bending behavior. Thus, assuming bilinear hysteretic response, the total axial yield force provided by the "fiber" springs should be the same as that provided by the beam-column element, while the yield axial strain should be the same for all "fiber" springs and the beam-column element.

$$F_y^{Beam-Column} = \sum_{i=1}^{N} F_{y,i} \quad and \qquad \varepsilon_y^{Beam-Column} = \varepsilon_{y,i}\,, \;\; for\ i = 1,\ldots,N \tag{2}$$

Elastic and post-yield stiffnesses should also satisfy similar concepts.

Considering that moment – axial force interaction is directly taken into account at the end regions of a segment by the distributed "fiber" springs, it is necessary to consider inelastic behavior at the beam-column elements with moment-axial force interaction envelopes in order to maintain consistent nonlinear behavior over the super-element. Furthermore, to avoid penetration of a segment into its adjacent ones, contact springs (e.g. hertzian contact law) should be considered at the perimeter of the end cross-sections.

As far as the selection of the portion of a segment which is modeled by "fiber" springs is concerned, it may be based on the geometric characteristics (cross-section depth, wall thickness, segment length) of the segment and the level of nonlinear behavior considered and/or expected at the segmental joints of interest. Thus, for longer segments, the length of the "fiber" springs may equal the depth of the cross-section, whereas, for shorter segments, the "fiber" element length may be taken such that the contact elements at the perimeter of the cross-section are not activated (or not significantly activated) during the analysis. For short segments, it is also suggested that shear deformations be considered for the beam-column element.

Due to the nature of the proposed modeling approach, three generalized section forces can be generated at the ends of the super-element; an axial force N_S and two moments, $M_{S,y}$ and $M_{S,z}$, defined as:

$$F_S = \sum_{i=1}^{N} F_i \quad , \quad M_{S,y} = \sum_{i=1}^{N} z_i F_i \quad and \quad M_{S,z} = -\sum_{i=1}^{N} y_i F_i \tag{3}$$

where F_i is the force of the i-th "fiber" spring located at (y_i, z_i) on the cross-section. Shear and/or torsional resistance may be introduced either by considering a shear springs between every pair of nodes of a "fiber" spring, or by considering one shear/tortional spring between the two end nodes of two adjacent beam-column elements. The properties of these springs are determined by the properties of the segments in contact and the segment-to-segment interface.

Regarding the tendons of the post-tensioning system, each of them is modeled by several axial "tension-only" bilinear springs in series. Each node of this spring assembly is laterally constrained to the adjacent segmental joint. Post-tensioning is achieved by considering initial element stressing, either as a thermal effect, or as pre-stressing.

Model Definition

A 2-D numerical model for the bridge specimen in the lateral direction is developed using the structural analysis software Ruaumoko (Carr 2004) to illustrate the proposed multi-element modeling approach. The model, which is shown in Figure 7, utilizes "compression-only" bilinear hysteretic springs with slackness as "fiber" springs, beam-column elements incorporating moment-axial force interaction diagrams, hertzian contact elements, and "tension-only" axial bilinear springs with slackness to model the post-tensioning system The pier segment cross-section is divided into 9 "fiber" spring areas along its depth, 5 of which are located in the webs, whereas the remaining 4 are located in the two flanges (2 in each flange). The portion of the segments (longitudinally) modeled by "fiber" springs is 6 inches at each end, resulting in "fiber" elements of total length of 12 inches. Although the cap beam and the foundation block are assumed to be rigid compared to the pier segments, short "fiber" springs (3 inches in length) are considered in the area of contact in both the cap beam and the foundation block to account for the existing local flexibility. The properties of the "fiber" elements are derived by taking into account the effect of both the concrete and the steel reinforcement, while the moment-axial force interaction diagrams are determined using appropriate computer codes. To avoid penetration of a segment into the adjacent ones, contact springs are placed at the edges of the cross-section and in the middle of the cross-section depth.

The post-tensioning system, which consists of 8 tendons, is modeled by three series of "tension-only" axial bilinear springs. The two series closer to the exterior surfaces of the segments model the tendons located at the flanges (3 at each flange), whereas the one series in the middle of the segment depth models the two tendons located at the webs.

No sliding is considered at the segment-to-segment joints and at the interface between the cap beam and the deck.

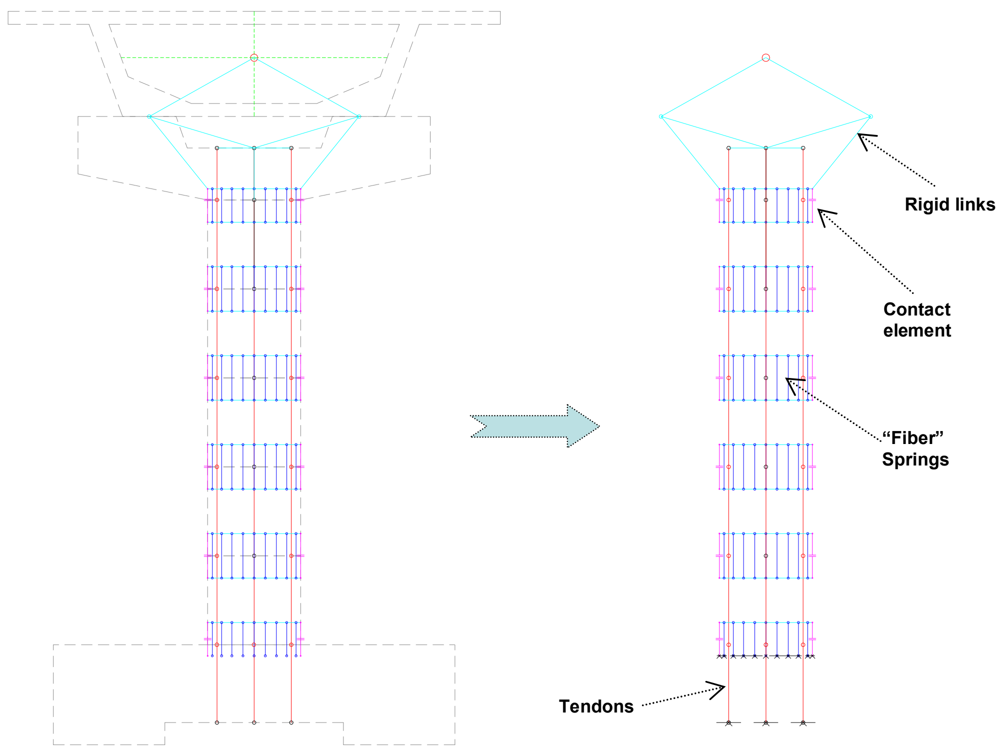

Figure 7. Numerical model for bridge specimen in the lateral direction

Modal and Dynamic Analyses

By performing a modal analysis for the bridge specimen using the model presented in Figure 7, the first few modes of the response are computed. Thus, the first natural period of the system is found to be 0.224 seconds, while the 2^{nd} and the 3^{rd} natural periods are found to be 0.018 seconds and 0.009 seconds, respectively. In the context of the similitude analysis performed for the design of the test specimen, the natural periods of the prototype may be approximated as 0.535 seconds, 0.043 seconds and 0.021 seconds for the 1^{st}, 2^{nd} and 3^{rd} mode, respectively.

Dynamic analyses are performed with 200% amplitude of the North-South component of the 1940 El Centro record. To abide by the assumption of the similitude analysis, the earthquake accelerogram is also scaled both in time (×1/2.4) and amplitude (×2.4), resulting in a peak ground acceleration of 1.62g (compared to the original 0.34g). Rayleigh damping is considered for the numerical model with a 3% damping ratio assigned to its two first modes. From the data obtained from the time history analysis of the system, observations may be made regarding the dynamic behavior of the bridge specimen in the lateral direction. Thus, the ground acceleration applied to the system and the total acceleration computed at the deck are shown in Figure 8. From these time histories, it can be seen that the absolute deck acceleration does not exceed an upper bound of 0.5g to 0.6g. Such behavior may be attributed to the joint opening at the base of the system (pier segment-to-foundation block joint) which controls/limits the maximum base moment that can be developed at the joint, and, consequently, controls the maximum seismic force applied to the system as well. An illustration of the self-centering capabilities of the bridge specimen considered in this study is presented in Figure 9 which shows the relative displacement response of the deck. According to this figure, the residual deformation of the system is negligible, which may be attributed to the fact that the tendons did not yield during the analysis, while the concrete segments experienced negligible inelastic deformations.

Conclusions

The proposed precast segmental bridge system with internal unbonded tendons and shear resistance at the joints provided only by friction between adjacent segments in contact seems to perform satisfactorily under earthquake excitation. Preliminary numerical analyses show that the system can be designed to exhibit high ductility and enhanced self-centering capabilities. These analyses followed the concepts presented above for modeling of segmental systems with 2-node elements, and they seem to have provided reasonable results.

Significant information on the seismic behavior of precast segmental systems following the design concepts described earlier is expected to be obtained from quasi-static and shake table testing of the bridge specimen described earlier. The tests will be conducted in the SEESL of the State University of New York at Buffalo, U.S.A.

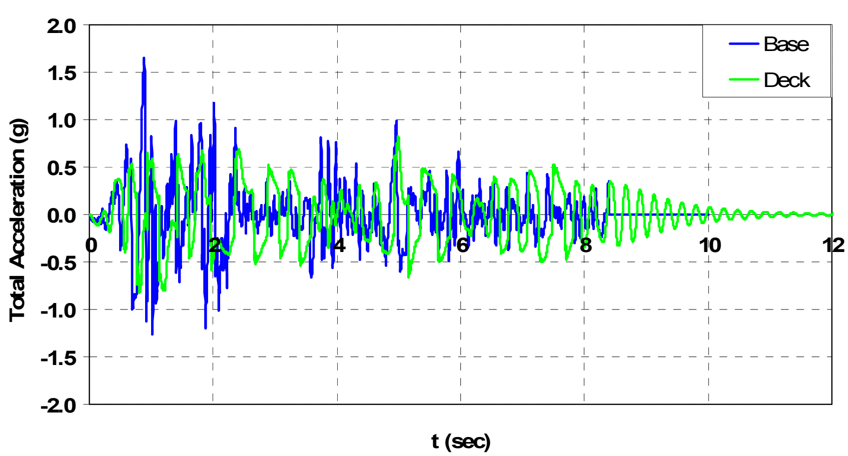

Figure 8. Total acceleration time histories

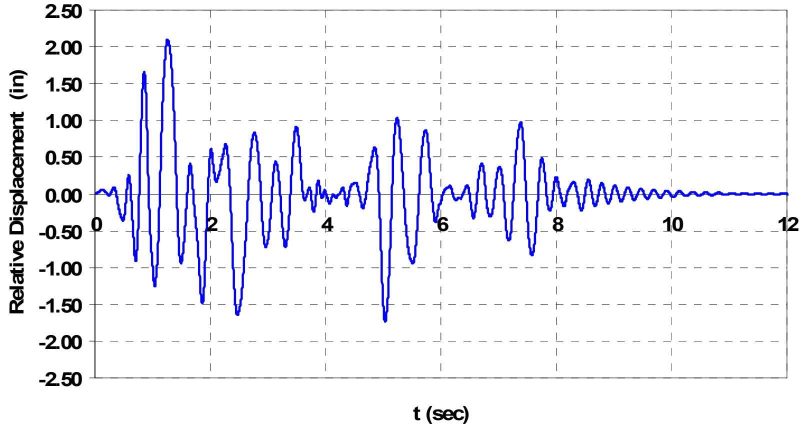

Figure 9. Relative displacement time history of deck (measured at its center of mass)

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