Pushover Analysis of Curved Steel Bridges for Evaluating Seismic Performance and Unseating Prevention

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ABSTRACT :
Demand on existing bridges for seismic capacity evaluation becomes more crucial and imperative. On the other hand, the performance-based philosophy has been accepted as a more reasonable design concept for engineering structures. For this purpose, capacity evaluation, as a result, and demand prediction procedures for civil engineering structures under earthquake excitations are of great significance. This paper describes a displacement-based seismic performance verification procedure including capacity and seismic unseating predictions for curved steel bridges and investigates its applicability. Prevailing pushover analyses procedure is employed as a primary tool to investigate the bridge's behaviors under seismic input. A failure criterion for steel members incorporated with the effect of local buckling is considered in the analytical model and an equivalent single-degree-of-freedom system with a simplified bilinear hysteretic model formulated using pushover analyses results is introduced to estimate the displacement capacity and maximum potential unseating mechanism of curved steel bridges under major earthquakes. To check the accuracy of the proposed method, seismic capacities and demands from multi-degree-of-freedom time-history analyses with artificial design earthquake inputs modeling major earthquakes are used as benchmarks for comparison. By a case study, it is clarified that the proposed prediction procedure can give accurate estimations of displacement capacities and demands of the curved steel bridge in the transverse direction, while insufficient tally with for the longitudinal direction, which confirms the conclusion drawn in other structure types about the applicability of pushover analyses.

KEYWORDS:
seismic evaluation procedure, seismic performance, curved steel bridge, seismic capacity, pushover analyses

1. INTRODUCTION

The Chien Kou Viaduct was designed and construction completed in 1980s. The project site is located in Taipei main district and being a major transportation line. The 1980 Chien Kou Bridge currently carries eastbound traffic on Interstate 80 from the greater San Francisco Bay Area to the Sacramento Valley. This 1022-meter long bridge spans the Chien Kou Strait, a major shipping channel for the Sacramento River. Both the shipping channel and the vehicular artery that the Chien Kou Bridge provides are considered important routes of commerce for the State of California. The bridge has three spans consisting of two 45-meter anchor spans, one 55-meter center span.

All analysis in support of the retrofit design was performed using the finite element program, SAP2000. SAP2000 was specified by the Taipei City Government Bridge Seismic Retrofit Program because it permits the user to evaluate important non-linear behaviors for the bridge structures, such as, expansion joints, damping devices, bearings, contact and non-linear axial force-bending moment interaction. This paper presents the results of the pushover analysis. A discussion of the development of moment-curvature properties for the non-linear members is presented. The procedures used to perform the collapse analysis and determine the behavior of individual members is also discussed. Results of the collapse analyses for the as built, prototype retrofit and final retrofit designs are presented and discussed.

The purpose of the dynamic non-linear analysis effort was to determine the structural response for the bridge steel members due to the postulated safety evaluation earthquake. As part of this effort, the capacities of the as-built and retrofit s were determined via collapse, or "pushover," analyses. These analyses investigated the mechanisms that could lead to collapse of the s, including local and global buckling of members, axial tension failure, and plastic hinge formation due to moment yield or rupture. The collapse analyses were also used to determine the overall ductility of the structures and ductility of individual structural elements. Several prototype retrofit designs for the s were evaluated and compared to the as-built capacities to determine the improvements in ductility realized through various retrofit configurations.

Prototype retrofit s with all of the structural elements characterized as non-linear were later included in a full bridge model, shown in Figure 1, where the superstructure was modeled as elastic. The design earthquake motion was applied to the system, and the system was analyzed. Finally, superstructure members considered essential to the primary load-path, as well as members with significantly high demand-capacity ratios, were characterized using non-linear properties for subsequent earthquake analyses.

Figure 1 Curved Steel Bridge for Analysis

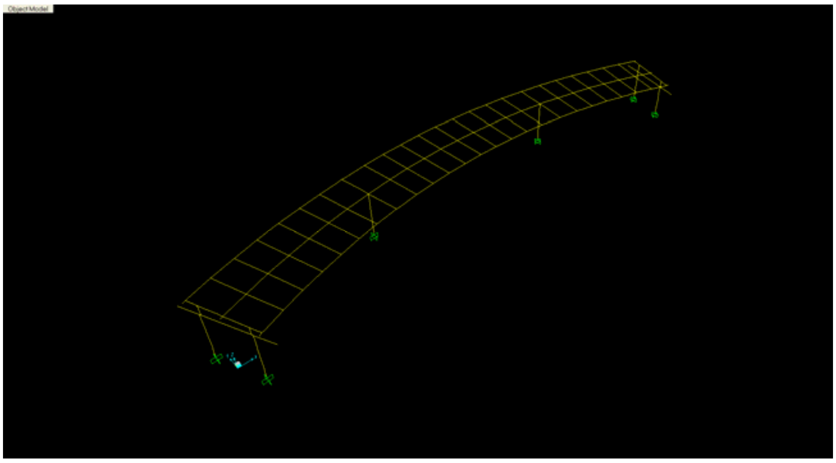

Figure 2 Global SAP2000 Finite Element Model

2. MOMENT-CURVATURE BEAM ELEMENT

One of the latest features implemented in the SAP2000 program is a non-linear beam defined through moment-curvature and axial force-strain interaction. The development of this class of elements is an important advancement in the available tools for estimating the structural behavior in large bridge structures because of the size of such bridge models and the efficiency of these elements. For the non-linear analysis activity, most of the bridge truss and members that exhibited inelastic behavior were defined using non-linear moment-curvature SAP2000 beam elements.

The inelastic members were characterized using four sets of non-linear curves. These curves defined axial force-strain, bending moment-curvature for both bending axes, and torsion-twist. The axial force was coupled separately to each of the two bending moments and torsion, resulting in a family of curves for each principal direction. These families of curves thus defined the axial force-moment interaction for each element. In SAP2000, axial force-strain curves must be symmetric for tension and compression. However, a different set of moment-curvature relationships may be defined for axial tension versus axial compression. In other words, moment-curvature curves need not be symmetric with respect to the sign of the axial force.

3. NON-LINEAR MEMBER PROPERTIES

Perhaps one of the most important issues for the analysis of the 1980 Chien Kou Bridge was the establishment of reliable member properties, that is, moment-curvature and axial force-stain properties that properly represented the behavior of such members. Much effort was invested establishing these member properties. Derivations of the axial force-strain and moment-curvature curves are based on established engineering mechanics principles that assume that strains can be linearly interpolated over the cross-section and curvatures, when combined with uniform axial strains, result in a non-linear stress distribution that can be integrated to determine resisting force and moment. Therefore, moment-curvature yield points and ultimate curvatures can be determined for any state of axial force-strain using the member's material yield stress and ultimate strains. Moment-curvature and axial force-strain curves were simplified to bilinear curves in a manner that maintains the total energy of the theoretical curves, i.e., the total area under the theoretical curves and the simplified curves were equal. The moment-curvature curves contain, as a minimum, a yield point and a rupture, or ultimate curvature, point. Each moment-curvature curve corresponds to a specific member axial force. The range of axial forces in combination with the family of moment-curvature curves defines the interaction surface for the element. Figure 3 shows a typical set of these moment-curvature curves. The axial force curves were developed using weighted average section properties for the bridge's perforated members. Above the critical local buckling compressive load, the bending resistance is considered to be zero. For both axial tension and flexure, the bending curvature rupture points correspond to either the ultimate tensile strength of the material, or the point at which bending will cause local buckling in one of the extreme fibers.

4. PUSHOVER ANALYSIS METHODOLOGY

For the pushover analyses, the geometry of the s was extracted from the global SAP2000 model and each element in the structures was modified to capture its expected non-linear behavior. Before any non-linear analysis was implemented, rigorous tests including material properties, beam moment-curvature elements, and non-linear buckling behavior were conducted.

Each structural member with a high slenderness ratio was subdivided into four finite elements each defined using five integration points along their lengths. Components comprising individual members of v-brace systems, with probable out-of-plane collapse modes, were subdivided into two non-linear finite elements of equal length. For members with low slenderness ratios where collapse was considered improbable, no element subdivision was implemented. Each element was offset from the work point of the connection. The length of the offset was taken from the work point as defined in the elastic model, to the last line of bolts in the gusset plate connecting the member to the joint. This offset of the end of each member from the connection work point was represented using rigid links. This implementation of rigid links implies that the retrofit design details would ensure that connection capacities exceeded member capacities by a minimum of 30 percent, thereby forcing any yielding to occur in the member rather than within the region of the connection. Where connections controlled, such as, for several members in the as-built structure, axial tension-strain curves were defined, such that the tension rupture point corresponded to the capacity of the connection.

5. BRIDGE COLLAPSE ANALYSES

A key step in the evaluation of the behavior of the 1980 Chien Kou Bridge structure was the individual collapse, or "push-over," analyses. These analyses provided valuable insight to the structural characteristics of the as-built s, as well as the effects of various retrofit alternatives on their displacement capacities. collapse analysis models for the as-built and prototype retrofit configurations were extracted from the elastic SAP2000 global model. Boundary conditions at the bases of the s were defined consistent with the details of their support upon their respective caissons or pile caps. Where retrofit details permit the column legs freedom to uplift, contact surfaces were placed at the bottom of the legs with elastomeric bearing pads modeled using linear or non-linear spring elements. Shear keys and restrainer beams were modeled per the design details, with end moment and axial force releases properly represented.

The top of each was constrained to a "push" node located at the plan center with the constraints defined consistent with details of force transfer between the bridge and superstructure. The constraint details included fixity of the top of each leg to the bridge, and shear transfer at the apex of top chevron bracing members and shear keys. The dead weight of the structure was applied as the first step. The mass of the portion of the bridge superstructure that was supported by the was also applied as a vertical load to the push node in this step. Next, a displacement or rotation was applied to the push node in the direction of study and the displacement was increased until P-δ collapse of the occurred. Typically, the was "pushed" to collapse in the longitudinal and transverse directions, as well as in three intermediate directions at 30, 60, and 75 degree angles to the centerline of the bridge. A global twist (or rotation) was also applied to the "push" node in order to determine the torsional ductility of the "Collapse" was defined to occur, and the solution was typically terminated, when the reaction at the push node peaked then dropped to less than 80% of the maximum value. Several steps were required for post-processing results of collapse analyses. For all non-linear members, axial force, bending moments, curvatures, and member distortions, as well as total and plastic axial strains were evaluated.

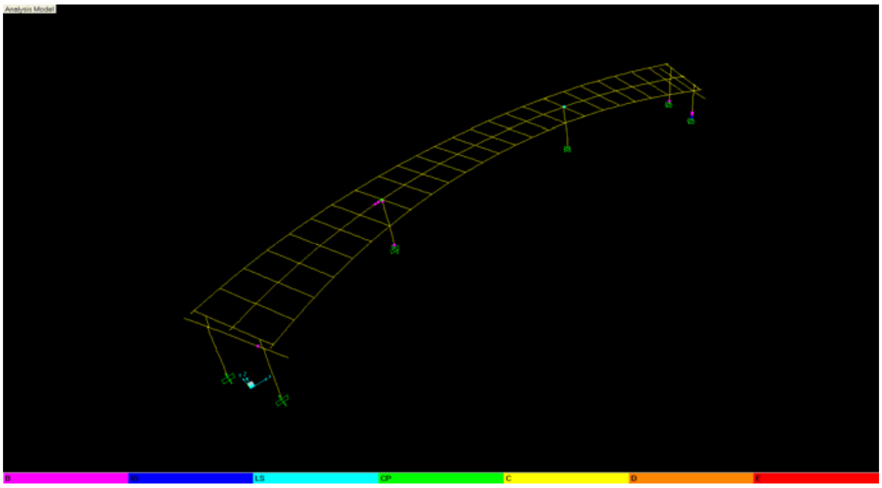

Figure 3 Global SAP2000 Analyzed Collapse Sequence

A collapse scenario was constructed and displayed for the active frame in each push direction. The active frame is defined as the frame that is in the plane of the push load and thus active in resisting this load. A load-deflection curve was plotted for each collapse analysis, and important events along the collapse history were noted. For each significant event, a "snapshot" of the displaced shape of the structure is developed, and yield and/or rupture states of the elements in the frame are indicated along with the displacement associated with each member non-linear activity.

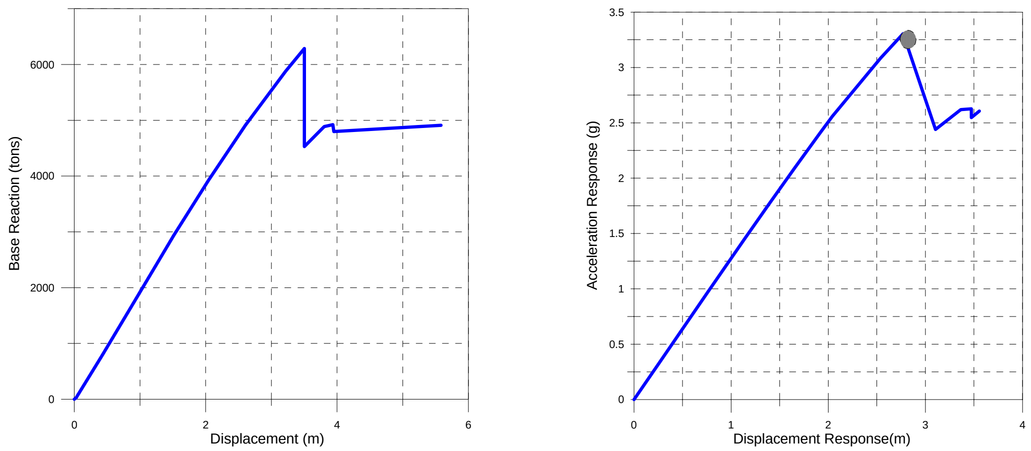

Figure 4 Load versus Displacement curves (longitudinal)

The collapse scenario, in conjunction with the strain and distortion time histories, provided a good physical understanding of the behavior of the structure on both a component level and a global structural level. The collapse scenario showed clearly the yielding sequence and re-distribution of forces, which occurred when various elements yielded, ruptured in tension, or buckled in compression. The collapse analyses were also useful for establishing displacement capacity envelopes for each structure. By applying push loads at various angles to the centerline of the bridge s, the maximum displacements for each loaded direction were plotted on a polar graph, thereby defining the displacement capacity envelope. This envelope was useful when compared to maximum displacements of the s from the global time history analyses.

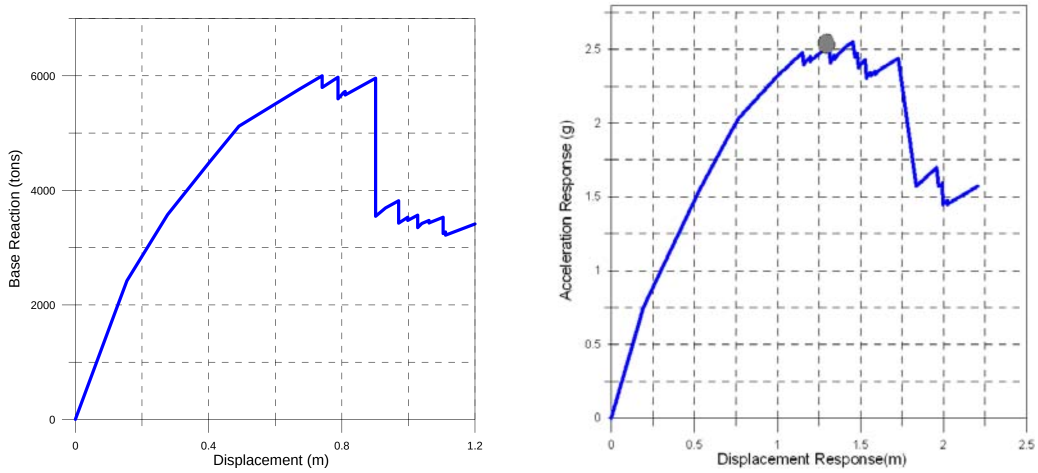

Figure 5 Load versus Displacement curves (transversal)

6. UNSEATING LENGTH DEMANDS DISCUSSION

Restrainers, or unseating prevention devices in a broader sense, were first developed and implemented in bridges after the 1964 Niigata, Japan, Earthquake. Extensive damage of bridges that resulted from excessive relative displacements between superstructures and substructures created an inspiration for developing unseating prevention devices. They include steel plate connectors, bar connectors, cable restrainers and chains that tie two decks or a deck and a substructure. Providing sufficient seat length is an important unseating-prevention measure. Today unseating prevention devices have become an important component of bridges worldwide.

However, because unseating prevention devices are not expensive structural components, little attention has been given to develop a comprehensive design procedure after the pioneering research at EERC. The seismic design force has been crudely obtained by multiplying a static seismic coefficient by a reaction force. Effect of pounding has also been considered a secondary importance in seismic design, since pounding results in only local damage at the end of superstructures. Because of the widespread damage, which occurred in the recent earthquakes in the USA, Japan, Taiwan and Turkey, the importance of unseating prevention devices and pounding effects on the total response of a bridge system is becoming widely accepted. Although pounding causes only local damage at the contact face, it transfers large seismic lateral forces from one deck to another, which results in a significant change in the seismic response of the entire bridge system. Unseating prevention devices also affect the total response of a bridge system. A good example for this is the collapse of an approach span of the Nishinomiya Bridge system, Hanshin Expressway in the 1995 Kobe, Japan, Earthquake. Main bridge, Nishinomiya Bridge, was a Nielsen Lohse bridge with a mass of 12,000 t, while the approach span was a steel plate girder bridge with a mass of 1,900 t. These two structures were tied together by plate-type restrainers. The damage was initiated by failure of fixed-bearings of the main bridge. This allowed large response displacement of the main bridge to take place, and the main bridge pulled the approach span, which resulted in failure of the fixed-bearings in the approach span. As a consequence the approach span dislodged from its support when the decks moved in the other direction. The unseating prevention devices were not strong enough to support the approach span once it dislodged from the support. Skewed bridges exhibit a unique structural response under strong excitation either when pounding occurs between decks or when unseating prevention devices restrain deck response. Because there is a horizontal eccentricity between a line of action created by pounding and unseating prevention devices, and the mass center, the decks twist as well as the laterally displaced during an earthquake. If the skew angle is larger than a certain value, the rotation results in collapse of a skewed bridge without contact of the deck ends with its abutments or adjacent decks.

Extensive analyses and experiments are being conducted on the effect of unseating prevention devices and pounding effects. Based on experiments and analyses, it is now known that tying together two adjacent spans is not appropriate if the masses or natural periods of the two spans are very much different, as was the case of the

Nishinomiya Bridge. In such an instance, enough seat length should be provided for the prevention of the spans from dislodging from their supports. Various new devices are continuously developed for unseating prevention devices.

7.DISCUSSION AND CONCLUSIONS

In addition to the overall unseating prevention retrofit goals for the bridge, certain behavioral characteristics of the retrofitted strategies sought, namely:
(1) Increased ductility of the bridge pier members.
(2) Provide for unseating length to resist all of the longitudinal seismic reactions of the superstructure.
(3) Significantly increased displacement capacity for the bridge without increase in force resistance. Figure 10 is a comparison of the load-deflection response of the as-built bridge structure. Through examination of the changes in behavior between as-built configurations, the pushover analyses demonstrated that the retrofit measures greatly improved displacement ductility. Most notably, displacement capacity in the longitudinal direction for bridge increased over 650%.

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