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Effect of Plastic Hinge Properties in Nonlinear Analysis of Highway Bridges

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ABSTRACT

Current design practice of highway bridges is moving towards an increased emphasis on nonlinear static analysis methods. Modelling for such analysis requires the determination of the nonlinear properties of the bridge elements that are expected to behave nonlinearly. Nonlinear static analysis is carried out for either user-defined nonlinear hinge properties or automated-hinge properties, as available in the software SAP2000. User defined hinge properties can be obtained using the recommendations of the Seismic Retrofit Manual by the Federal Highway Administration. Automated-hinge properties in SAP2000 are computed automatically from the element material and section properties according to Caltrans criteria. The bridge designer needs to be aware of that the majority of old bridges were built with little or no consideration to seismic forces and plastic hinge detailing requirements. Therefore, the use of automated-hinge properties for old bridges in nonlinear static analysis may lead to unrealistic displacement capacities. In this study, pushover analysis of two highway bridges built with little attention to seismic forces was performed in an effort to evaluate the difference in global response predicted by using the user-defined nonlinear hinge properties or automated-hinge properties in the software SAP2000. The results demonstrated that user-defined hinge model is capable of capturing the effect of local failure mechanisms, in the plastic hinge region, on the global response of the bridge; while the automated-hinge model can not capture this effect. Therefore, automated-hinge properties should be used with a lot of care, especially for old bridges that might include local failure mechanisms in the plastic hinge region.

KEYWORDS: Seismic, Bridges, Nonlinear, Pushover, Hinges, Earthquake engineering.

INTRODUCTION

The American Association of Highway and Transportation Officials (AASHTO) provides guide specifications for seismic design of new highway bridges under the document Guide Specifications for LRFD Seismic Bridge Design (AASHTO, 2009). For old or existing bridges, the Seismic Retrofitting Manual (SRM) for Highway Structures: Part 1 by the Federal Highway Administration (FHWA) provides the most current state-of-practice in assessing the vulnerability of highway structures to the effects of earthquakes and implementing retrofit measures to improve performance (FHWA, 2005). The guidelines permit the bridge engineer to utilize a variety of methods for seismic evaluation, from simple connection forces and seat width checks (Method A1/A2) to complex nonlinear dynamic analysis (Method E2).

For regular and irregular bridges, it is common practice to utilize Method D2 (structure capacity/demand method) for seismic evaluation of existing bridges (FHWA, 2005). Method D2 which is also known as Pushover Method employs elastic methods such as the multi-mode response spectrum

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method, or an elastic time history method, to determine the seismic demand on the bridge. Capacity assessment is based on the displacement capacity of individual piers as determined by a pushover analysis.

Pushover analysis should be able to track the nonlinear relationship between load and deformation of the bridge as the lateral load is monotonically increased from an initial elastic condition to failure. This requires the estimation of the capacity of each of the critical structural members, from first yield until collapse and at intermediate limit states. Therefore, member performance is expressed in terms of force *versus* deformation, moment *versus* rotation or shear force *versus* distortion.

The deformation capacity of a component depends on the curvature capacity of the plastic hinge and its length. The use of different criteria for estimating the curvature capacity of a plastic hinge may result in different deformation capacities of the structure. The FHWA-SRM, (FHWA, 2005), provides detailed procedures for calculating the plastic curvatures of the structural members based on potential local failure mechanisms within the plastic hinge. The governing limit state is the state that has the least plastic curvature capacity. Plastic curvatures for the following limit states are discussed in detail in the FHWA-SRM, they are: compression failure of unconfined concrete; compression failure of confined concrete; compression failure due to buckling of the longitudinal reinforcement; longitudinal tensile reinforcing bar fracture; low cycle fatigue of the longitudinal reinforcement; failure in the lap-splice zone and shear failure of the member that limits ductile behavior. In practical use, most often the default properties provided in the FEMA-356 (FEMA, 2000) and ATC-40 (ATC, 1996) documents are preferred due to convenience and simplicity (Shatarat et al., 2007).

Mehmet and Hayri (2006) studied the effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings. The authors studied the possible differences in the results of pushover analysis due to default and user-defined nonlinear component properties. Four- and seven-storey buildings were considered to represent low- and medium- rise buildings for this study. The results of their works showed that the misuse of default-hinge properties may lead to unreasonable displacement capacities for existing structures. The observations clearly showed that the user-defined hinge model is better than the default-hinge model in reflecting nonlinear behavior compatible with the element properties.

The software SAP2000 is (Computers and Structures, Inc., 2009) the most commonly used software for performing pushover analysis for highway bridges (Shatarat, 2007). In SAP2000, pushover analysis can be carried out for either user-defined nonlinear hinge properties, default-hinge properties or automated hinge properties. User defined hinge properties can be obtained using the recommendations of the FHWA-SRM. Automated-hinge properties in SAP2000 are computed automatically from the element material and section properties according to Caltrans criteria (Caltrans, 1994). The bridge designer needs to be aware of that different hinge properties will result in different force-deformation capacities of the structure. Therefore, the main objective of this work is to investigate the difference between the use of user-defined hinge properties and the use of automated-hinge properties on the force-deformation capacity of old highway bridges.

PUSHOVER ANALYSIS

Pushover analysis in SAP2000 assumes that nonlinear behavior occurs within frame elements at concentrated plastic hinges with automated or user-defined hinge properties being assigned to each hinge (Shatarat, 2007). Different types of plastic hinges can be defined in SAP2000: uncoupled axial P, shear V2, shear V3, torsion T, moment M2 and moment M3 and interacting P-M2-M3 frame hinge types. In this study, a coupled axial force and biaxial bending moment hinge (P-M2-M3 hinge) was assigned to the upper and lower ends of the columns of the piers model.

Hinge properties are defined through the definition of the moment–curvature relation, plastic hinge length and an interaction surface. For the case of user-defined plastic hinge properties, the plastic curvature capacity of columns is determined in accordance with FHWA-SRM. Potential plastic hinge locations and local deformation limit states such as compression failure of concrete, buckling of longitudinal reinforcement, low-cycle fatigue, lap splice failure and shear failure are identified for each column within a pier. The limit state resulting in the least plastic curvature of a member is considered to be the controlling limit state. The plastic rotational capacity of the plastic hinge is directly proportional to the column curvature capacity through the plastic hinge length which can be determined from equations available in the FHWA-SRM. Curvature capacity of a plastic hinge depends on the axial load level in the column. Therefore, a number of moment curvature relationships at different axial load levels are created to accurately capture the behavior of the bridge. Three axial load levels that are of an interest are: point of pure compression (Po), point of pure bending (P0) and point of expected load level from dead and seismic loads (Pe). Figure 1 shows typical moment-rotation relationships for a plastic hinge at the three axial load levels.

For the case of automated plastic hinge properties, hinge properties are defined in accordance with Caltrans hinge specifications. Hinge properties are defined through the definition of the material properties and cross-section properties of the columns. Definition of the material properties includes defining stress strain curves for the core concrete inside the transverse reinforcement, the unconfined concrete outside the core and the longitudinal reinforcement rebars. Cross-section properties include the size of the section and longitudinal and transverse reinforcement details.

Pushover analysis can be performed using force controlled pattern or displacement controlled pattern. A lateral force distribution or displacement pattern is applied in an incremental fashion while monitoring the occurrence of nonlinear behavior and the displacement of a control node. In this study, a displacement pattern was used as the loading pattern for pushover analysis. This is most useful for structures that become unstable and may lose load-carrying capacity during the course of the analysis (Computers and Structures, Inc., 2009).

PLASTIC CURVATURE CAPACITY OF AN RC MEMBER

The plastic curvature capacity of a reinforced concrete member is based on the governing limit state for that member. The governing limit state is the state that has the least plastic curvature capacity. Plastic curvatures for the following limit states are discussed in details in the FHWA-SRM (FHWA, 2005), they are: compression failure of unconfined concrete; compression failure of confined concrete; compression failure due to buckling of the longitudinal reinforcement; longitudinal tensile reinforcing bar fracture; low cycle fatigue of the longitudinal reinforcement; failure in the lap-splice zone and shear failure of the member that limits ductile behavior. Two limit states relevant to the scope of the study are discussed in detail in the following part:

Compression Failure of Unconfined Concrete

According to the FHWA Seismic Retrofit Manual, the plastic curvature corresponding to compression failure in unconfined concrete is given by:

$$\phi_p = \frac{\varepsilon_{cu}}{c} - \phi_y \qquad (1)$$

where ε_{cu} is the ultimate concrete compression strain for concrete, which should be limited to 0.005 for unconfined concrete, and c is the depth from the extreme compression fiber to the neutral axis.

Buckling of Longitudinal Bars

According to the FHWA-SRM, if a compression member has inadequate transverse reinforcement with a spacing, s, in potential plastic hinge zones that exceeds six longitudinal bar diameters (i.e., $s > 6d_b$), then local buckling at high compressive strains in the longitudinal reinforcement is likely to happen. The plastic curvature of this failure mode can be determined from:

$$\phi_p = \frac{\varepsilon_b}{(c-d')} - \phi_y \quad (2)$$

where d' is the distance from the extreme compression fiber to the center of the nearest compression reinforcing bars, and ε_b is the buckling strain in the longitudinal reinforcing steel. If $6d_b < s < 30d_b$, the buckling strain may be taken as twice the yield strain of the longitudinal steel; i.e.,

$$\varepsilon_b = \frac{2f_y}{E_s} \quad (3)$$

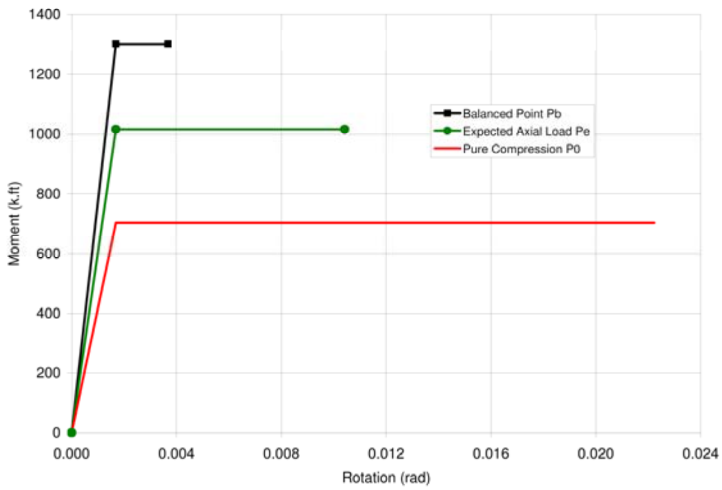

Figure 1: Typical Moment-Rotation Relationship for a Plastic Hinge at Different Axial Load Levels

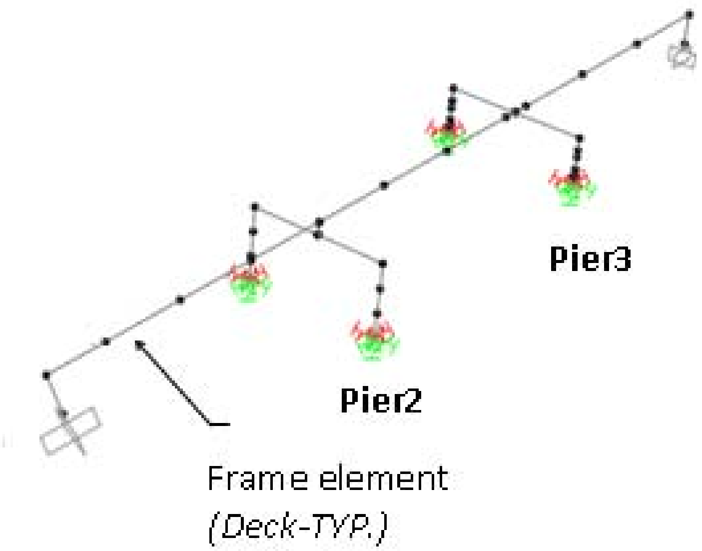

Figure 2: Three-dimensional Model of the Bridge

DESCRIPTION OF SELECTED BRIDGES

Two old bridges that have potential local failure mechanism in the plastic hinge zone are described briefly in the following.

Bridge #1

Bridge # 1 which was built in 1940 consists of three equal-length simply supported spans comprised of precast concrete girders totalling 230 feet in length. The intermediate piers (Pier #2 and Pier #3) are comprised of two circular 36-in. diameter columns founded on spread footings. The compressive strength of the concrete is 4.0 ksi and the yield strength of the longitudinal and transverse bars is 40 ksi.

The columns at Pier #2 and Pier #3 have a clear length of 20.0 ft. The columns are longitudinally reinforced with eleven #9 bars and transversely with #3 hoops spaced at 12 in. The longitudinal rebars of the columns are spliced within the plastic hinge zone with a splice length of 36.0 in.

Bridge #2

Bridge #2 which was built in 1945 consists of three equal-length spans comprised of multi-cell cast-in-place concrete box section. The intermediate piers (Pier #2 and Pier #3) are comprised of four square 2' 6" x 2' 6" columns founded on spread footings. The compressive strength of the concrete is 4.0 ksi and the yield strength of the longitudinal and transverse bars is 40 ksi.

The columns at Pier #2 and Pier #3 have a clear length of 24.0 ft. The columns are longitudinally reinforced with eleven #9 bars and transversely with #3 hoops (2 legs in each direction) spaced at 12 in. The longitudinal rebars of the columns are spliced within the plastic hinge zone with a splice length of 48.0 in.

MODELING APPROACH

A three-dimensional model utilizing the software SAP2000 was created for each bridge, as shown in Figure 2. Based on the FHWA-SRM recommendations, a spine-type model was used to represent the bridge superstructure. Superstructure was represented by a single line of multiple three-dimensional frame elements, which passes through the centroid of the superstructure. Rigid elements were provided between the centroid of the superstructure and the centroid of the crossbeams.

Since the bridge columns are expected to respond inelastically under the input ground motions, effective column properties were used to reflect concrete cracking and reinforcement yielding. FHWA-SRM, (Table 7-1) was used to determine the effective rigidities for different components of the bridge. Figure 3 shows a typical bridge pier and the associated stiffness properties of the frame elements. Bridge foundations, superstructure and crossbeams are assumed to behave elastically for the purpose of this analysis.

Each bent is supported by a spread footing that is modeled using spring elements. The soil springs were generated using the method for spread footings outlined in Caltrans bridge design specifications (Caltrans, 1994).

In Version 14 of SAP2000 (Computers and Structures, Inc., 2009). concrete and reinforcement nonlinear material properties for Caltrans sections are defined in the material definitions themselves. The strain at Unconfined Compressive Strength, f'c and the Ultimate Unconfined Strain Capacity are set to the values required in Section 8.4.4 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO, 2009). These unconfined properties are parameters used in defining the Mander confined concrete stress-strain curve of the column core (Mander et al., 1988). Figure 4 shows a typical stress-strain curve for confined concrete as defined by Mander et al. (1988).

For the longitudinal reinforcement, the expected yield stress was taken as, Fye = 1.1Fy = 44.0 ksi and the expected ultimate tensile stress was taken as, Fue = 1.5 Fu = 66 ksi as required by the FHWA-SRM.

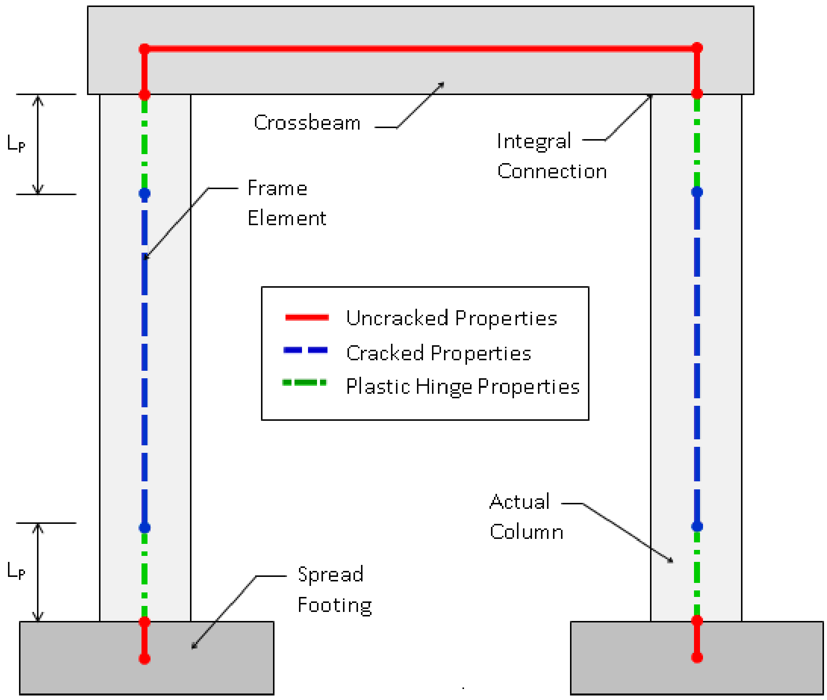

Figure 3: Locations of Section Properties in a Typical Pier (LP: Equivalent Plastic Hinge Length)

Table 1. Curvature Capacity of Intermediate Piers, Bridge #1

Limit State	Ultimate Curvature (rad/in.)
Compression Failure	0.0005374
Buckling of Longitudinal Rebars	0.000616
Fracture of Longitudinal Rebars	0.004101
Low-cycle Fatigue of Longitudinal Rebars	0.002503
Failure in the Lap-splice Zone	0.001740

RESULTS AND DISCUSSION

An eigenvalue analysis was performed to identify natural periods and mode shapes. The eigenvalue analysis resulted in the longitudinal and transverse fundamental natural periods and associated modal participating mass ratios (i.e., effective modal mass to total mass ratios). Preliminary seismic analysis was then carried out to identify the potential for inelastic response. Results from the seismic analyses (i.e., the moments and axial forces in the columns due to combined dead and seismic loads) revealed that columns inelastic behavior is likely to occur for the expected acceleration levels where bridges are located. Consequently, pushover analysis was performed for the bridge models to determine the capacity curves of the bridge. For brevity, pushover analysis of the highway bridge in the transverse direction will be discussed.

Pushover analysis starts with defining the moment curvature capacity, plastic hinge length and interaction surface of the plastic hinge zone. For the case of user-defined plastic hinge properties, the curvature capacity of the plastic hinges was determined using the recommendations of the FHWA-SRM (Shatarat, 2007).

Table 1 shows the curvature capacity of the intermediate piers corresponding to each local failure mechanism expected in the plastic hinge zone for Bridge #1 and for an axial load level corresponding to Pe. The controlling limit state of the plastic hinge was found to be buckling of longitudinal bars. Table 2 shows the curvature capacity of the intermediate piers corresponding to each local failure mechanism expected in the plastic hinge zone for Bridge #2 and for an axial load level corresponding to Pe. The controlling limit state of the plastic hinge was found to be compression failure of the unconfined concrete. It is worth to note that the controlling limit state of the plastic hinge might change with the axial load level on the columns. For this study, it was assumed that the controlling limit state remains the same under the effect of different axial load levels.

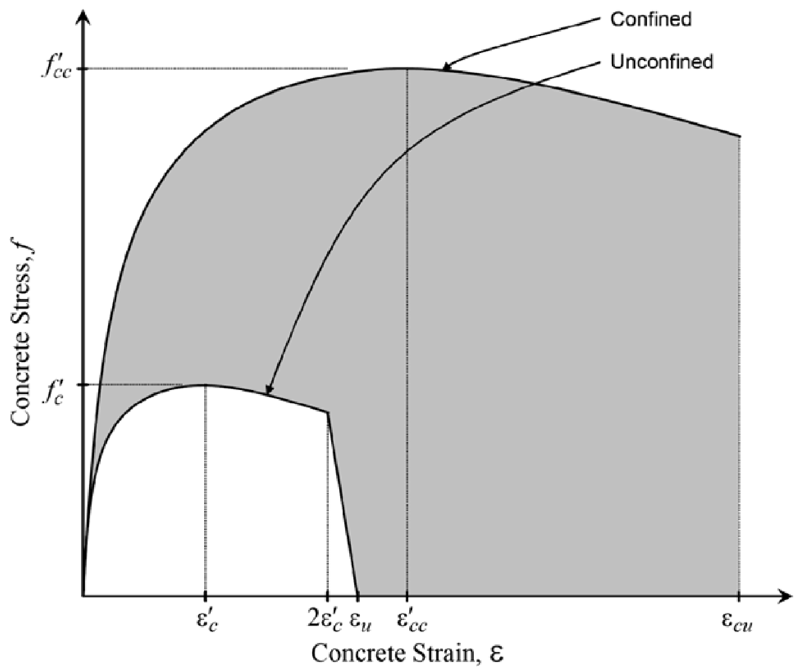

Figure 4: Mander Confined and Unconfined Stress-Strain Curves (Mander et al., 1988)

The curvature at the first yield of longitudinal bars is given by the following equation (FHWA, 2005):

$$\phi_y = \frac{2\varepsilon_y}{D'} \qquad (4)$$

where ε_y is equal to Fye divided by the elastic modulus of elasticity of the longitudinal rebars (29000 ksi) and *D'* is the cross-sectional dimension measured between the center line of the transverse reinforcement. The moment corresponding to the ultimate curvature is called the flexural moment overstrength capacity, Mpo. Details of the procedure and equations used to calculate Mpo at different axial load levels can be found in FHWA-SRM.

The equivalent plastic hinge length is given by the following semi-empirical equation (FHWA, 2005):

$$L_p = 0.08L + 4400\varepsilon_y d_b \qquad (5)$$

where d_b is the diameter of the longitudinal tension reinforcement and L is the shear span or effective height (i.e., L = M/V).

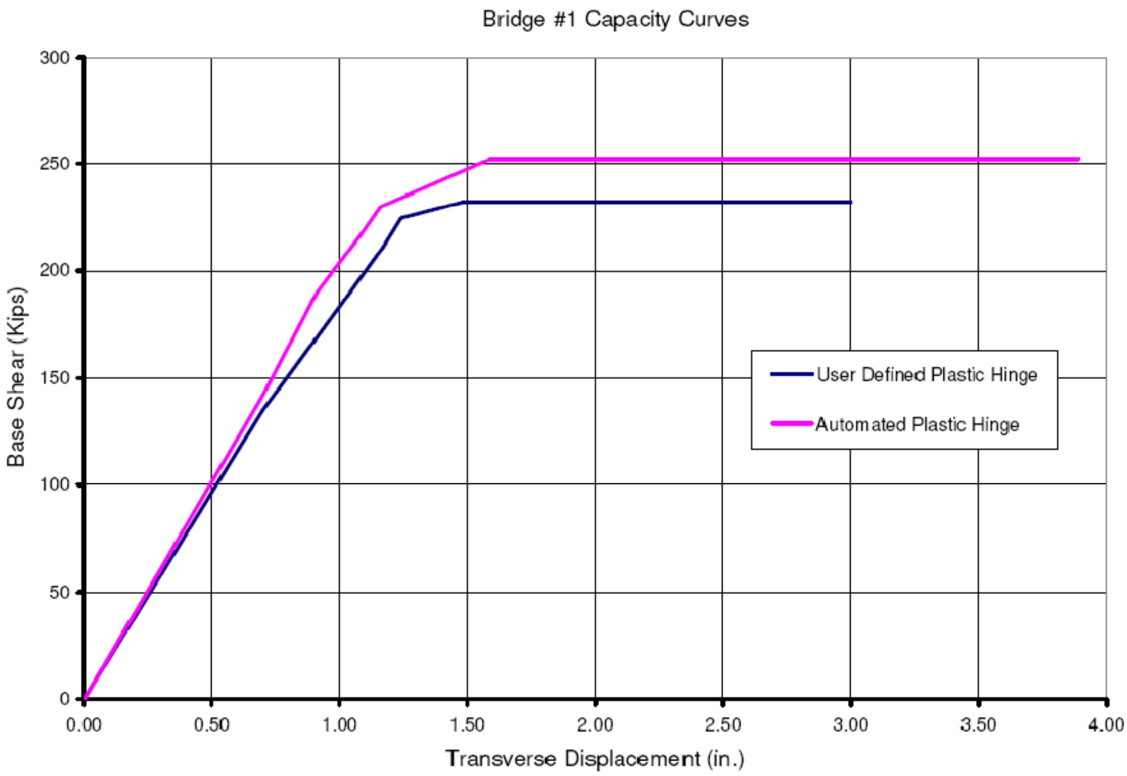

Figure 5: Transverse Pushover Curves for Bridge #1

For the case of automated hinge properties, the cross-section size and reinforcement details were assigned through the section designer in SAP2000 (Computers and Structures, Inc., 2009). Unconfined concrete stress-strain curve was assigned to the concrete cover, Mander stress-strain curve was assigned for the column core concrete and a typical mild steel stress-strain curve was assigned for the longitudinal rebars.

The length and the relative location of the plastic hinge for the case of automated plastic hinge properties and for the case of user-defined hinge properties were kept the same.

The pushover curve was obtained by first analyzing the bridge under the effect of dead load and then pushing the bridge until any column plastic hinge within the bridge piers reaches its maximum inelastic curvature capacity. The pushover curve was obtained by performing a displacement-controlled analysis wherein the bridge was subjected to a prescribed displacement pattern. The resulting base shear *versus* control node displacement represents the pushover curve.

Figure 5 shows the capacity curves for Bridge #1 in the transverse direction for user-defined and automated hinge properties. Figure 6 shows the capacity curves for Bridge #2 in the transverse direction for user-defined and automated hinge properties. It is clear from the Figures that different capacity curves were obtained using different plastic hinge properties. This difference is due to the fact that user hinge properties and automated hinge properties result in different moment curvature capacities of the plastic hinge. Therefore, it is recommended that plastic hinge zones that might have local failure mechanisms be modelled using the user-defined plastic hinge properties.

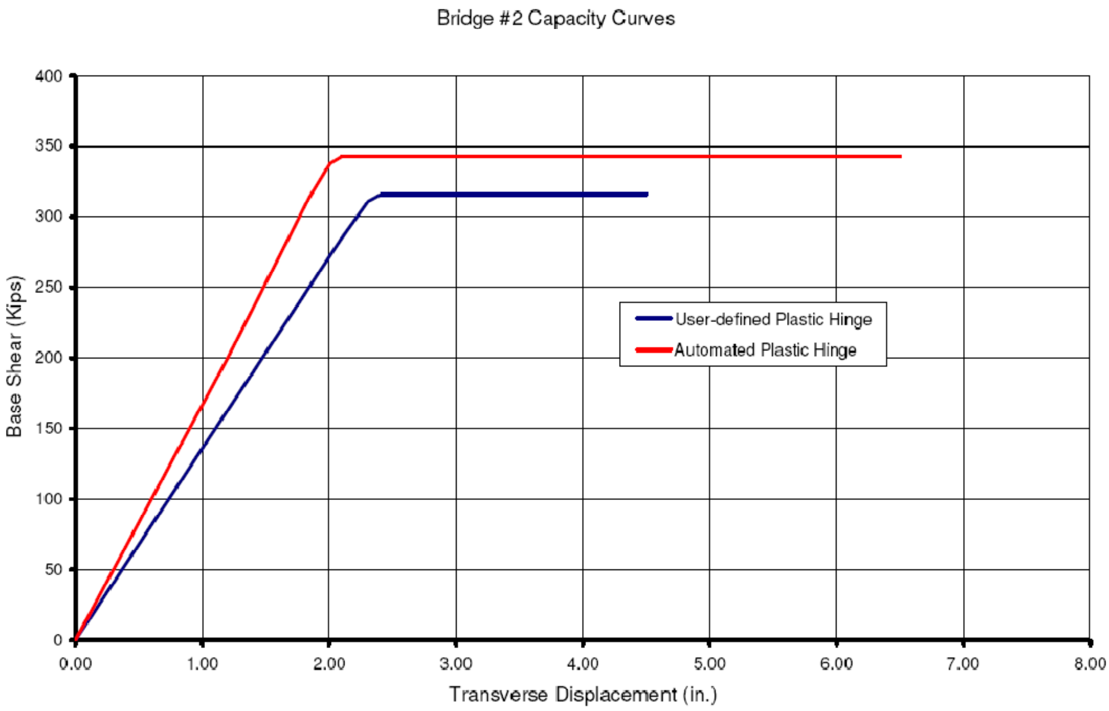

Figure 6: Transverse Pushover Curves for Bridge #2

Table 2. Curvature Capacity of Intermediate Piers, Bridge #2

Limit State	Ultimate Curvature (rad/in.)
Compression Failure	0.000728
Buckling of Longitudinal Rebars	0.000983
Fracture of Longitudinal Rebars	0.004207
Low-cycle Fatigue of Longitudinal Rebars	0.002320
Failure in the Lap-splice Zone	0.002170

CONCLUSIONS

The conclusions of the study may be summarized as follows:

- Local failure modes in the plastic hinge zone and thus the corresponding curvature capacity of the plastic hinge zone can be determined from equations provided in the FHWA-SRM.
- Automated hinge properties in SAP2000 are defined through the definition of the cross-section details and the nonlinear stress-strain properties of the unconfined concrete, core concrete and longitudinal reinforcement. Therefore, some local failure mechanisms that might occur within the plastic hinge zone cannot be evaluated through automated plastic hinge properties.
- For old highway bridges that were built and detailed with little or no attention to seismic forces,

it is recommended to obtain the plastic hinge properties through the equations provided in the FHWA-SRM and to utilize user-defined plastic hinge properties to obtain the capacity curve of the structure.

- Automated plastic hinge properties result in a capacity curve that is different from the capacity curve obtained using user-defined hinge properties especially when local failure mechanisms are present in the plastic hinge zone.

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