SEISMIC ASSESSMENT ANALYSIS OF BRIDGES IN MEXICO

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ABSTRACT:
In spite of the high seismicity of the country, a large number of existing bridges in Mexico were designed with non seismic provisions. Then, a methodology for the seismic evaluation of these structures is proposed. The assessment procedure is divided in two stages: the first involves a general screening study for the identification of the bridges in the worst conditions of vulnerability; the second stage is a displacement based procedure based on a capacity curve obtained with a non-linear static analysis. The results obtained with the proposed methodology have been compared for existing bridges that suffered different damage levels in past earthquakes and good correlation was observed. It is expected that the procedure will contribute to mitigate the seismic hazard posed by existent bridges.

KEYWORDS:
Seismic assessment, damage levels, damaged bridges

1. INTRODUCTION

More than 70% of the existing bridges in Mexico were constructed before 1970. In spite of the high seismicity of the country, most of these bridges were designed with non seismic provisions, or by using codes with seismic specifications that no satisfied the recommendations of present-day knowledge. These conditions make necessary the assessment of the seismic capacity of the existent bridges population, especially for those bridges located in the more seismic active areas of the country.

As a contribution to the evaluation of the seismic capacity of bridges in Mexico, an assessment methodology is proposed. The procedure is divided in two phases: the first one is a screening method for the preliminary assessment of seismic vulnerability of bridges, which allows the identification of bridges in a specific region for a detailed evaluation. The second stage is a displacement base assessment procedure, applicable to the bridges in the worst conditions of vulnerability according to the screening procedure. The capacity curve of the structure is obtained with a nonlinear static analysis, and the plastic hinge properties are obtained for the different pier conditions that can be encountered in existing bridges. Of course, these conditions do not correspond to a well designed pier and non ductile response could be expected. The structure displacement is obtained from an acceleration-displacement response spectra modified to take into account the equivalent damping of the system. The demand displacement is compared to the displacement (rotation) limit states.

2. GENERAL SCREENING AND PRIORIZATION METHOD

The procedure begins with a simplified evaluation that assigns a vulnerability index (Iv) to each bridge with the purpose of determine which bridges are most likely to pose the greatest risk in the study area, so that, and optimum allocation of resources can be made. The index Iv takes into account nine factor of risk, combined in a multiplicative relationship (Eqn. 2.1). The nine contributory factors for assessing relative risk were identified from the typical causes of damage in bridges during recent earthquakes, from bridges numerical studies and from expert's opinions. Equation 2.1 was obtained from a regression analysis of data related to seismically damaged bridges (Landa, 2006). The Iv index is limited between 0 and 1.0. Iv = 1.0 is assigned to a non vulnerable bridge, while an Iv = 0.0 means that the bridge is highly vulnerable.

$$Iv = \left[\frac{C_1 * C_2 * \dots\dots\dots * C_9}{\left(\bar{C}_i\right)^7} \right] \tag{2.1}$$

where C_i is the value assigned to the contributory factor i, ($\bar{C}_i$) is the average of the nine contributory factors, and $0 \leq Ci \leq 1.0$. As each factor of risk has different influence on the general stability of the bridge, the weight (values) of the factors of risk Ci is not the same.

2.1. Factors of risk

The way for obtaining the contributory factors of risk can be seen in Landa, 2006. Each factor is briefly described as follows:

a) C_1 is given by equation 2.2 and is proposed for evaluating the influence of transversal stiffness configuration on risk.

$$C_1 = 1.0 - \frac{k_M - k_m}{10\,k_m} \tag{2.2}$$

where k_M and k_m are the great and the minor stiffness of two adjacent supports respectively. The support´s stiffness determination must consider shear and bending deformations, especially for wall type piers, and also the bearings´ flexibility.

b) C_2 represents the risk associated to the lack of seat potential. The risk is evaluated in terms of the actual and recommended seat length comparison.

c) C_3 takes into account the common practice and code recommendations at the time the bridge was designed.

d) C_4 is a measure of the effect of bridge plan configuration on risk. Skewed spans develop larger displacements than right spans as a tendency of the skew span to rotate. The skew angle (α) is considered in C_4 and the curvature of the bridge.

e) C_5 takes into account the influence of the bearing in the stability of the bridge by means of the following coefficients: a) for isolating and energy dissipating devices C_5 = 1.0; b) for elastomeric bearings C_5 = 0.9; c) for roller bearings C_5 = 0.8; d) for rocker bearings C_5 = 0.7

f) C_6 is a measure of the bridge's deterioration that considers: scour condition under piers or abutments, bearing deterioration, damage on structural elements, damage on joint or connections, and maintenance condition.

g) C_7 is a measure of soil liquefaction potential.

h) C_8 relates the bridge's fundamental period (T_s) to the periods that define the corner ordinates of the design response spectra of the site (T_a and T_b).

i) C_9 depends on the importance of the bridge. For important bridges C_9 = 1.0/1.5 = 0.67, according to the importance factor recommended by Mexican codes (1.5). C_9 = 1.0 for normal bridges.

2.2. Evaluation of the screening procedure

The Iv index was evaluated for thirteen bridges that have been shocked by the 1994 Northridge earthquake (Maldonado, 2000). Table 2.1 shows the Iv and the damage level for each bridge. As it can be seen, there is a good correlation between damage level and the Iv index.

Table 2.1 Iv and damage level for 13 bridges damaged by the 1994 Northridge earthquake

No.	Bridge	Damage level	*Iv*
1	Ruta SR-14/I-5, North Connector Overcrossing	Collapsed	0.17
2	Fairfax – Washington Undercrossing	Collapsed	0.17
3	La Cienega – Venice Undercrossing	Collapsed	0.17
4	Ruta SR-14/I-5, Separation and Overhead	Collapsed	0.31
5	Old Road	Medium damage	0.54
6	Gavin Canyon Undercrossing	Collapsed	0.58
7	Mission – Gothic Undercrossing	Collapsed	0.58
8	Ball Creek Canyon Channel	Major damage	0.58
9	Santa Clara River	Medium damage	0.72
10	Pico – Lyons Overcrossing	Medium damage	0.74
11	Valencia Boulevard Overcrossing	Light damage	0.76
12	Mc Bean Parkway Overcrossing	No damage	0.80
13	Balboa Boulevard Overcrossing	Light damage	0.83

3. DISPLACEMENT BASED ASSESSMENT PROCEDURE

The method is intended for bridges whose lateral resistance is provided by the vertical supports, that is, piers or abutments. The assessment is based on the ratio of the seismic rotation demands and the accepted rotation limit states. The idealized stress-strain relationships commonly used for design are not suitable for an assessment procedure; rather, more realistic curves must be used. The steel stress-strain curve must include the strain hardening and the stress-strain relationship of the concrete must consider the resistance and ultimate strain increments due to confinement. Appropriate stress-strain relationships can be found in the literature (Caltrans, 2004, Mander 1986, or others). The ultimate concrete strain (ε_{cu}) should be determined by Eqn. 3.1 (Jara, 2004),

$$\varepsilon_{cu} = \frac{110\rho_s + 3.4\rho_l + 0.017\gamma\sqrt{f'_c}}{\rho_l\left(0.94 f_{yl} + 302\right) - \gamma\left(0.015 f'^2_c - 1.1 f'_c - 8\right)} \quad \left(f'_c \text{ y } \quad f_{yl} \text{ en MPa}\right) \tag{3.1}$$

where $\gamma = 0.45 + 0.5\sqrt{\dfrac{P}{f'_c A_g}}$ for rectangular sections and $\gamma = 0.45 + 0.7\sqrt{\dfrac{P}{f'_c A_g}}$ for circular sections.

3.1 Moment-curvature analyses for existing piers

The moment- curvature expressions must consider actual conditions of existing bridges in Mexico and the expected type of failure. The damage mechanisms proposed for moment-curvature analyses are: a) premature shear failure; b) axial compression coupled with bending of unconfined elements; c) axial compression coupled with bending of elements with different levels of confinement; d) premature failure due to bar splices located in the plastic hinge region.

a) Shear failure
When a shear failure occurs, the flexural capacity of the pier is not achieved and the ductility is drastically reduced as it can be seen in figure 3.1. The shear resistance is the sum of the contribution of the resistance provided by concrete V_c, shear reinforcing V_s and axial force V_p, defined by equations 3.2 to 3.4 (Priestley et al, 1996),

$$V_c = k\sqrt{f'_c}\,0.80A_g \tag{3.2}$$

$$V_s = \begin{cases} \dfrac{\pi}{2}\dfrac{A_h f_y D' \cot 30}{s} & \text{circular section} \\ \dfrac{A_v f_y \cot 30}{s} & \text{rectangular section} \end{cases} \tag{3.3}$$

$$V_p = P\tan\alpha \tag{3.4}$$

where A_g is the gross area, k a ductility factor (Priestley et al, 1996), D' the core diameter, A_h the area of one leg of shear reinforcement, A_v the shear area in the analysis direction, s is the spacing of stirrups and α the angle between the centroid of the compression zone and the point of application of the axial force.

b) Unconfined elements

For unconfined elements, without bar splices in the plastic hinge region, the ultimate strain ε_{cu} is assumed as 0.005 (Priestley et al, 1996), and f'c is taken as the unconfined concrete resistance.

c) Confined elements

The volumetric ratio of transverse reinforcement, the axial load ratio and the longitudinal reinforcement ratio are the most significant variables for the moment-curvature expressions (Landa, 2006). Moment-curvature relationships for existing piers with different levels of confinement (ke) where obtained and displayed in figure 3.2. As it can be seen, the confinement effectiveness is very important for definition of the plastic hinge properties for an inelastic analysis.

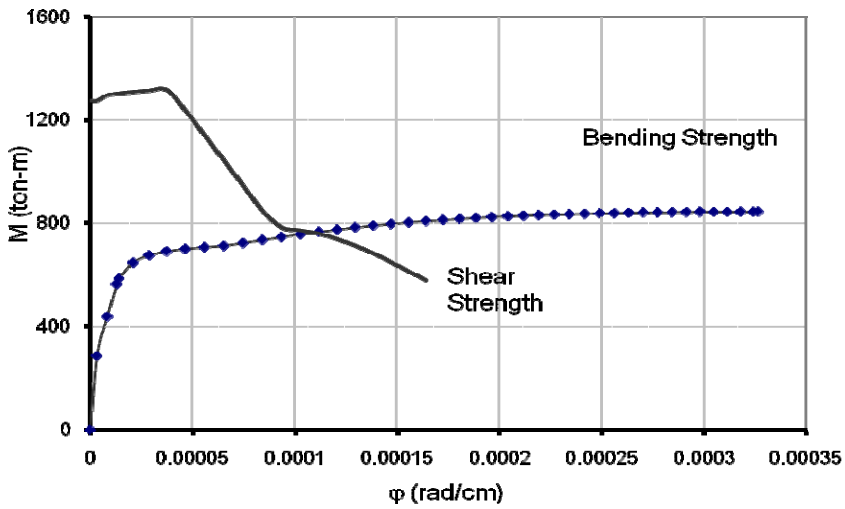

Figure 3.1 Premature shear failure

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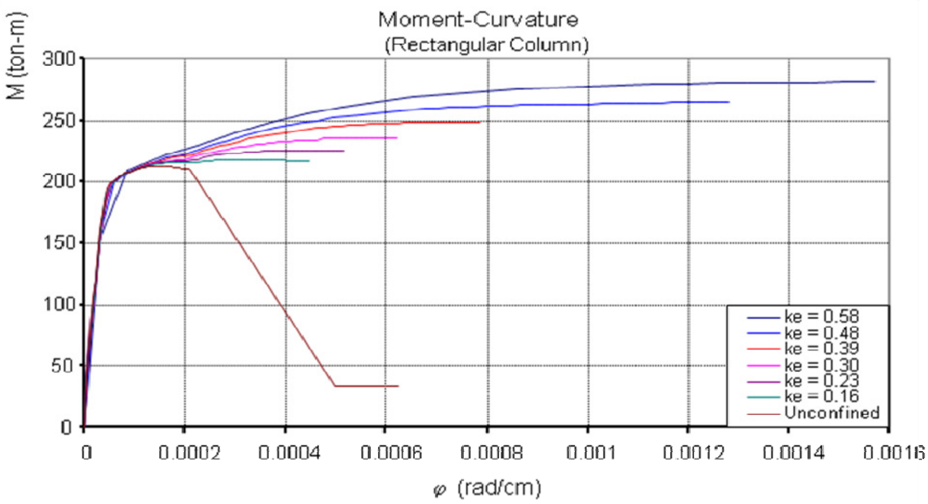

Figure 3.2 Moment – curvature relationships for different levels of confinement

a) Bar splices in the plastic hinge region

If splices of the longitudinal reinforcement are located in the critical region of the element, extensive cracking would appear for low curvature levels reducing drastically the ductility, and the force transmitted by the steel reinforcement could be less than the yield force in some cases. As longitudinal micro-cracks appear at low strain levels, reducing the tension capacity of the concrete, the ultimate strain ε_{cu} is assumed as 0.002 (Landa, 2006). Figure 3.3 shows the moment-curvature graphics obtained for elements with bar splices in the plastic hinge region. The reduction of ductility can be seen if these curves are compared to the curves of figure 3.2.

3.2 Displacement demand

A non linear static analysis is suggested for obtaining the capacity curve of the bridge. The plastic hinge properties are given as idealized moment curvature expressions representative of the type of failure expected of the bridge's piers. Next figure shows the capacity curve of the same bridge, but considering the piers: confined, unconfined and with bar splices in the critical region.

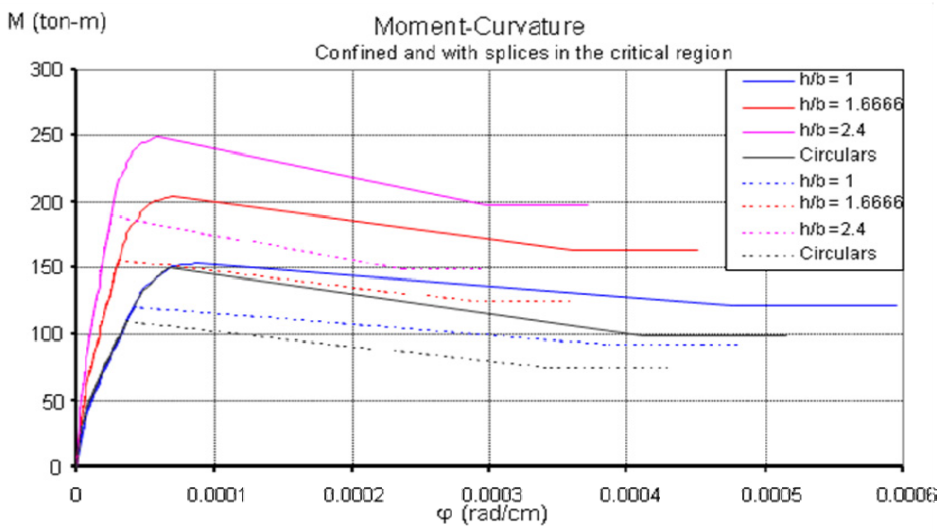

Figure 3.3 Moment–curvature relationships for elements with bar splices in a critical region

Following the ATC-55 (2005) procedure an Acceleration-Displacement Response Spectra (ADRS) is constructed for the site and for the return period assumed for the limit state under analysis. The effective period and equivalent damping of the structure are dependent on the displacement level as it is shown in equations 3.5 to 3.10. So, an iterative methodology is needed as the demand displacement is unknown at the beginning of the analysis. Once the demand displacement is obtained, the pier's displacement capacity must be checked.

For $\mu < 4$

$$T_{eff} = \left[0.20(\mu-1)^2 - 0.038(\mu-1)^3 + 1\right]T_0 \tag{3.5}$$

$$\beta_{eff} = 4.9(\mu-1)^2 - 1.1(\mu-1)^3 + \beta_0 \tag{3.6}$$

For $4.0 \le \mu \le 6.5$

$$T_{eff} = \left[0.28 + 0.13(\mu-1) + 1\right]T_0 \tag{3.7}$$

$$\beta_{eff} = 14 + 0.32(\mu-1) + \beta_0 \tag{3.8}$$

For $\mu > 6$

$$T_{eff} = \left[0.89\left(\sqrt{\frac{\mu-1}{1+0.05(\mu-2)}} - 1\right) + 1\right]T_0 \tag{3.9}$$

$$\beta_{eff} = 19\left[\frac{0.64(\mu-1)-1}{\left[0.64(\mu-1)\right]^2}\right]\left(\frac{T_{eff}}{T_0}\right)^2 + \beta_0 \tag{3.10}$$

3.3 Limit states

It is difficult to determine the optimum number of limit states that should be checked for assessment purposes. In some cases, only one check may be needed, but in other three or more limit states may be required. For completeness, the four limit states recommended by the Vision 2000 Committee (Bertero and Bertero, 2002) are considered. Based on the RC mechanical behavior and experimental results Jara (2004) proposed the following pier rotation limit states:

For a serviceability limit state it is expected that only minor cracks can appear and there would be no interruption of normal operations. The maximum rotation for this limit state is (Jara, 2004):

$$\phi_{\max} = \frac{0.7 f_y + 0.0015 E_s}{3 E_s d} L \tag{3.11}$$

For an operational limit state maximum cracks 1 mm wide should be formed, and the steel stress lie well below the yield stress. The rotation limit state for this case is,

$$\phi_{\max} = \frac{0.7 f_y L}{E_s h} + 0.0025 \tag{3.12}$$

The life safe limit state is derived from the consideration that the steel stress reaches the yield stress. Spalling of the cover concrete also occurs and initiation of plastic hinge formation can be observed. Based on this behavior ϕ_{max} = 0.015 is proposed (Jara, 2004).

In the collapse prevention limit state, spalling of the cover concrete in the plastic hinge region may occur and the concrete core can be damaged without fracture of the transverse reinforcement. For this limit state a maximum rotation ϕ_{max} = 0.025 is adopted (Jara, 2004).

Once the rotation limit states have been satisfied the resistance of the pier must be also checked. Then, a final decision about the safety of the bridge could be taken.

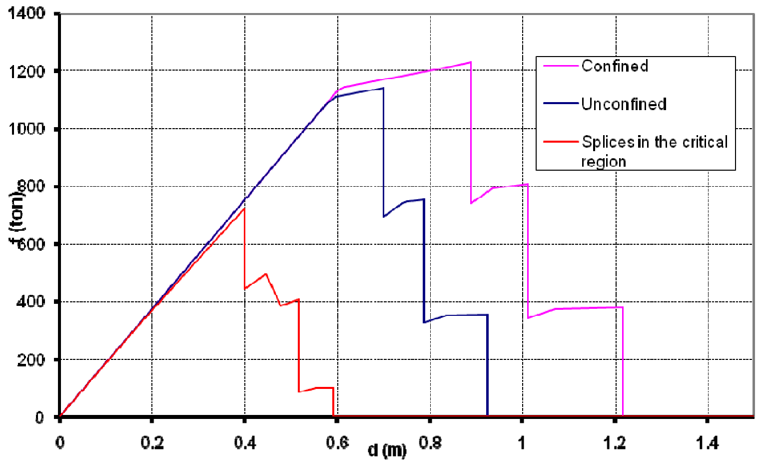

Figure 3.4 Capacity curves for the same bridge with different pier failure condition

4. CONCLUSIONS

A bridge seismic assessment procedure for bridges is proposed. The methodology is divided in two stages: the first is a screening method to identify the bridges in the worst conditions of vulnerability. The second is a displacement based procedure that considers the type of failure expected for existing bridges.

An ensemble of thirteen bridges that have been damaged by the Northridge earthquake (1994) was used to calibrate and evaluated the screening method. A good correlation between the vulnerability index and damage level for the thirteen bridges was obtained.

The final displacement demand is converted to rotation on the plastic hinges and it is compared to the four proposed rotation limit states to decide the safety level of the structure.

The procedure allows checking the seismic capacity of an important number of bridges that have been designed without present-day seismic design provisions.

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