Assessment of Reinforced Concrete Bridge Piers under Low to Moderate Seismic Induced Loads

Ali M. Memari¹ and Andrew Scanlon²

- Department of Architectural Engineering, The Pennsylvania State University, 104 Engineering "A" Building, University Park, PA 16802, USA, email: memari@engr.psu.edu
- Department of Civil and Environmental Engineering, The Pennsylvania State University, 212 Sackett Building, University Park, PA, 16802

ABSTRACT: Two existing highway bridges in a low seismic area in Eastern U.S. are studied for seismic assessment. Transverse reinforcement spacing and detailing do not satisfy the 1996 AASHTO seismic design specifications. Transverse response of selected two column bents under the action of Nahanni and El Centro earthquakes scaled to 0.15g show the level of lateral force demand to be significantly below yield level. Preliminary seismic assessment based on pier analysis is presented.

Keywords: Bridges; Earthquakes; Assessment; Reinforced concrete; Low seismicity; Eastern U.S.; AASHTO

1. Introduction

Earthquakes of the recent past in the U.S., Japan and several other countries have demonstrated the vulnerability of highway bridges to seismic damage not only due to strong earthquakes, but also as a result of low level ground motions. The degree of damage observed has varied from total collapse in Kobe earthquake [4] to minor cracking and spalling of cover concrete in Nisqually earthquake [5]. With the assumption of 10% probability of having damaging earthquakes in 74% of the states in the U.S. (including central and eastern states) within the next 50 years, the 1996 American Association of State Highway and Transportation Officials (AASHTO) specifications [1] have adopted National Earthquake Hazards Reduction Program (NEHRP) horizontal acceleration maps [7], which have placed much of the Eastern States into higher seismic risk categories. Moreover, according to Federal Highway Administration (FHWA) guidelines [6], an importance factor for Seismic Performance Category (SPC) should be considered with the distinction between "standard" and "essential" bridges. Such requirements for seismic performance indicate a potential need for upgrading many existing bridges in low to moderate seismic regions. However, most of

the current guidelines for bridge retrofit at the U.S. national level is conservatively based on the experience of the West Coast and other high seismic regions. For the past several years, Eastern States have been in the process of dealing with these new requirements. With the large inventory of East Coast bridges on hand, implementation of conservative seismic retrofit schemes would impose a significant strain on many Eastern States' financial resources. This paper presents a preliminary evaluation of the need for retrofitting of bridge piers in Eastern U.S., which has low to moderate seismic regions. Two existing bridges were selected for evaluation, one with circular piers and one with square piers. Both bridges are located in an area that has an acceleration coefficient of 0.05g according to the AASHTO seismic provisions maps [1] as shown in Figure (1). Preliminary evaluation of the bridge piers indicates insufficient confinement reinforcement requirements based on current AASHTO design provisions. The pier bents were analyzed to determine force levels generated by earthquakes with 0.15gpeak ground acceleration to evaluate the response in a low to moderate seismic region. Results of this analysis and a discussion on seismic assessment of the piers is presented.



Figure 1. Part of AASHTO 1996 acceleration coefficient map - 80 - 90 percent probability of not being exceeded in 50 years.

2. Description of the Structures

Two existing bridges were selected for this study, one with square piers and one with circular piers to have a representation of most commonly used column types in short highway bridges. The bridge with square pier is a three-span bridge (85 ft (29.91m) - 77 ft (23.47m) - 85 ft (29.91m)) with forr prestressed I-beam girders topped with comparent to reinforced concrete deck. Figure (2) shows the partial bent elevation for a bent with square piers, while Figure (2) shows the partial bent elevation for a bent with piece section (4 $ft \times 4 ft$ (1219 $mm \times 1219mm$). The bridge has two double pier bents. The # 1 (12...m) bar peripheral



Figure 2. Partial elevation and superstructure section for the bridge with square piers. (Note: 1 ft = 304.8mm, 1 in. = 25.4mm).

hoop spacing in the lowest 6 ft (1829mm) of the pier is 6 in. (152mm) and in the rest of the pier 12 in. (305mm). The octagonal hoops (cross ties), also # 4 (12.7mm) bar, are spaced at 12 in. (305mm). The bridge with circular pier is a two-span bridge $(101 \ ft \ (30.78m) - 125 \ ft \ (38.10m))$ with five prestressed I-beam girders and composite reinforced concrete deck. The bent in this bridge also has two piers with height 9.5 ft (2896mm). Figure (4) shows partial elevation and a superstructure section for the bridge with circular piers, while Figure (5) shows its partial bent elevation and a typical pier cross section (4.5 ft diameter (1372mm)). The circular piers in this bridge have circular # 4 (12.7mm) hoops at 12 in. (305mm) space r throughout the height. The hoops are ci ed with peripheral laps. The AASHTO [1] 'smic rov.sions require a maximum



Figure 3. Partial bent elevation for one of the bents and typical pier cross section for the bridge with square piers. (Note: 1 ft = 304.8mm, 1 in. = 25.4mm).



Figure 4. Partial elevation and superstructure section for the bridge with circular piers. (Note: 1 ft = 304.8mm, 1 in. = 25.4mm).

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Figure 5. Partial bent elevation and typical pier cross section for the bridge with circular piers. (Note: 1 ft = 304.8mm, 1 in. = 25.4mm).

spacing of 4.0 *in*. (102*mm*) or 6.0 *in*. (152*mm*) (depending on the SPC for the site, i.e., 6 in. (152*mm*) for SPC B and 4 *in*. (104*mm*) for SPC C and D) for the hoops over the entire column height. Obviously, both bridges violate this requirement.

3. Assessment Earthquake

In order to consider a more critical situation fo. these bridges than the 0.05g acceleration coefficient assigned (per AASHTO maps as shown in F gure (1)) to where they are located, it was $2ch^{-1}$ to apply the higher contour acceleration c flicient of 0.15g, which can be considered as $x \neq x$ to moderate seismic region for East Co \cdot t o the J.S. These prescribed maximum ground ce. ations define design earthquake motions such , at the probability of the elastic design f rc levels being exceeded in 50 years is in the ... ze of 5 to 20%. In order to develop spectra coelera ons, the May 19, 1940 El Centro (Imperia V., Station 9, Magnitude (M_{a}) 6.7, Orientation 180°, and peak ground acceleration 0.35g) and the December 23, 1985 Nahanni (Western Northwest Territories of Canada (Slide Mountain), Magnitude (M_{e}) 6.9, Orientation 240°, and peak ground acceleration 0.54g) records were chosen from Strongmo Database System) [13]. These two earthquakes represent, respectively, a large magnitude interplate earthquake for Western U.S. and a large magnitude intraplate earthquake for Eastern North America, according to Saadeghvaziri and Jones [11]. Both earthquake records were scaled to 0.15g to represent appropriate assessment earthquakes for low to moderate seismic regions. Figures (6) and (7) show







Figure 7. Acceleration-time history record for December 23, 1985 Nahanni earthquake with peak ground acceleration scaled to 0.15g.

respectively, the scaled acceleration time history for El Centro and Nahanni earthquakes. The El Centro record will serve as a basis for comparison. Response spectra were developed for the records using the step-by-step integration method described in Paz [9]. Figures (8) and (9) show the derived acceleration spectra for El Centro and Nahanni earthquakes, respectively. It should be noted that there are very limited choices for real earthquake records (particularly larger magnitude events) useful for Eastern North America. The Nahanni Earthquake has been identified as an appropriate record for this area (e.g., [11]). Atkinson and Boore [2] provide further information on the appropriateness of this event for Eastern North America from the seismological perspective. For application of the record in this paper, although scaling down the record to 0.15gwill result in most significant peaks (other than the largest one) to be less than 0.05g, the use of El Centro record in the study will compensate any such shortcoming.



Figure 8. Derived response spectra for scaled (to 0.15g) ⊟ Centro earthquake.



Figure 9. Derived response spectra for scaled (to 0.15g) Nahanni earthquake.

4. Preliminary Seismic Evaluation

AASHTO [1] defines bridges in areas with acceleration coefficients in the range of 0.09 to 0.19 as belonging to Seismic Performance Category (SPC) B regardless of the Importance Classification (IC). This is illustrated in the top part of Table (1). SPC C and Dare used for higher acceleration coefficients. For regular bridges, a single mode (e.g., first mode) spectral analysis is sufficient, according to AASHTO. Furthermore, for SPC B., it prescribes seismic d ϵ si, n forces to be determined by dividing elastic reis ic forces obtained from two orthogona' cor 'rin tions by the recommended response modific tion ctor, which for multiple column bent 5° The jesulting modified seismic design force shud then be combined with other applicative inds, e.g., dead loads, etc. If the two selected bridges were to be

Table 1. Definition c⁺ Seismic . rformance Category (SPC).

Seismic Performance	recry (SPC) -	- AASHTO [1]				
Acceleration Coefficient	importance Classification (IC)					
A	I	II				
$A \leq 0.09$	А	А				
$0.09 < A \le 0.19$	В	В				
$0.19 < A \le 0.29$	С	С				
0.29 <a< td=""><td>D</td><td>С</td></a<>	D	С				
Seismic Performance Category (SPC) – FHWA [6]						
Acceleration Coefficient	Importance Classification					
	Essential	Standard				
$A \leq 0.09$	В	А				
$0.09 < A \le 0.19$	С	В				
$0.19 < A \le 0.29$	С	С				
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designed as r. brices, design forces would then be obtaine bas, on he described procedure. For assessment purpos however, an FHWA report [6] suggests or \int of C instead of B for the above ac eleration range if the bridge can be considered ess, tial as opposed to standard, as shown in the lower art of Table (1). The difference between the two recommendations stems from the way nport cce classification is applied. Whereas AASHTO considers this factor applicable only for zones with an acceleration coefficient greater than 0.29, the FHWA report applies it to all zones, including Eastern U.S., where due to the nature of Eastern U.S. earthquakes, the maximum credible earthquake is believed to be much larger than the design earthquake. For the purpose of this preliminary study, it was decided to evaluate these two bridges assuming they belong to SPC C.

The analysis procedure for SPC C bridges includes consideration of the formation of plastic hinges, following the capacity design approach [10], in addition to the conventional linearly elastic analysis. First, the analysis based on the formation of plastic hinges is discussed. The cap beam is conservatively assumed to be rigid, as are the pier to footing connections. It is further assumed that plastic hinges form simultaneously at top and bottom of piers. Of course, whether in an actual earthquake simultaneous plastic hinges will form at top and bottom of piers or a single plastic hinge at top or bottom, instead, depends on many factors, notably the single pier versus multiple pier bents. The assumption of simultaneous top and bottom hinge formation is conventional in multi-column bents for push-over analysis, particularly when the cap beam is stiffer than the columns and the foundation can be assumed as rigid. Example results of momentcurvature analysis by *SEQMC* software [12] for circular and square pier types are shown in Figures (10) and (11), respectively. In order to use more realistic material properties, $1.5f_c$ ' for concrete strength and $1.1f_y$ for reinforcing steel yield strength were used, as recommended by Priestley et al [10].



Figure 10. Example moment-curvature diagram for circular pier. (Note: 1 Kip-ft = 1.356kN-m, 1 k = 4.448kN, 1 ft = 304.8mm, 1 in. = 25.4mm)





For the initial moment curvature analysis, the dead loads were used to obtain axial loads (P_G) in the piers. Using the yield moments (M_{ij}) at top and bottom of piers, corresponding shears were determined, i.e., shear = (column top yield moment + column bottom yield moment)/column length. The lateral bent force $(E_{\rm v})$ was then determined using pier shears, and when it was applied at the center of mass of the superstructure, tension and compression axial forces $(P_{\rm F})$ in the piers were determined from static equilibrium of the bent free body diagram. This simplified analysis is based on the assumption that the bent along with its tributary superstructure can be considered as an isolated single degree of freedom system viously, for more accurate analysis, a three dimensional finite element modeling should be used. The calculated axial forces were then comp. I'd w i' the dead loads to get more accurate mon. It curvature results. Recalculation of pier hear force and subsequent lateral bent force after several iterations eventually establishes the lateral yic^{1,1} force level. This will be further discussed bsequently. The results are summarized in Table (2) The lateral bent forces E_{y} , calculated as per xpla ned procedure, are listed in Table (3). In Table (2), EI_{eff} refers to the effective flexural stiffness, where E is the modulus of elasticity and Iis the moment of inertia.

Table 2. Results of moment-curvature analysis.

	M_y	$\phi_y * 10^{\text{-5}}$	$\mathrm{EI}_{\mathrm{eff}}$ *10 ⁶	\mathbf{P}_{E}	$\mathbf{P}_{\mathbf{G}}$
	(Kip-ft)	in/in	(Kip-ft ²)	(Kip)	(Kip)
Circular (tens-top)	3429	6.45	4.43	554	843
Circular (tens-bottom)	6055	6.97	7.24	554	890
Circular (comp-top)	4865	7.26	5.58	554	843
Circular (comp-bottom)	7334	7.63	8.01	554	890
Square (tens-top)	1618	6.16	2.19	290	335
Square (tens-bottom)	6044	7.32	6.88	290	395
Square (comp-top)	2576	6.77	3.17	290	335
Square (comp-bottom)	6808	7.71	7.36	290	395

(Note: 1 Kips-ft=1.356kN-m, 1 k = 4.448kN, 1 ft = 304.8mm, 1 in. = 25.4mm)

Table 3. Assessment earthquake demand forces (Horizontal bent forces).

	Ey	\mathbf{K}_{eff}	W	T_0	$S_a(g)$	$\mathrm{E}_{\mathrm{Demand}}$		
	Kips	Kips/ft	Kips	sec	El Centro	Nahanni	El Centro	Nahanni
Circular	1112	20456	1733	0.32	0.30	0.17	520	295
Square	681	7534	730	0.35	0.26	0.15	190	110

(*Note:* $1 \ k = 4.448 \ kN$, $1 \ ft = 304.8 \ mm$)

As mentioned before, according to the AASHTO [1] seismic design provisions, the hoop spacing should be 4.0 *in*. (104*mm*) or 6.0 *in*. (152*mm*), which for SPC C, the former is the maximum. The requirements are intended for piers to provide sufficient ductility capacity during strong earthquakes. In low to moderate seismicity regions, however, the behavior under design earthquake could well be in the elastic range. Although for design of new bridges it can be quite justified to use the required small hoop spacings at minimal extra cost, the issue of upgrading existing bridges with deficient hoop spacing certainly deserves much analytical efforts for assessment.

One approximate analysis approach for seismic assessment of bridges with piers not satisfying AASHTO hoop spacing requirements involves determination of the level of seismic force demand (E_{Demand}) according to the assessment earthquake [10]. The first step involves determining the lateral force demand using the derived response spectra for assessment earthquakes. Using the results of momentcurvature analysis, the effective flexural stiffness can be obtained as $EI_{eff} = M_{y} / \phi_{y}$, see Table (2). It should be noted that EI_{eff} value is obtained by the SEQMC program using the values of theoretical yield moment and corresponding curvatures as shown in Figures (10) and (11). The values listed in Table (2) are the results of the 3rd iteration of the process described previously. The values of gross moment of inertia and the effective moment of inertia corresponding to the 3^{rd} iteration is shown in Figures (10) and (11).

Based on the conservative assumption of rigid cap beam, the lateral stiffness of the bent can be obtained as $K_{eff} = 24 E I_{eff} / H^3$. This effective stiffness can then be used to estimate the fundamental period of the bent, considering the tributary dead load of the superstructure, the cap beam and half of the piers, as follows: $T_0 = 2\pi (W/gK_{eff})^{1/2}$. The tributary weight of the bridge (W) is listed in Table (2). Using the calculated value for the period in the lateral direction, we can obtain the spectral acceleration from Figures (8) and (9) assuming a damping ratio of 5% for each bridge and the corresponding lateral force demand, $E_{Demand} = S_a W/g$. The results of such calculations are presented in Table (3). It should be noted that the calculated lateral force demand is the result of applying only one component of each of the earthquakes.

From Table (3), it can be seen that the lateral force demand level (E_{Demand}) for the circular pier is 47% and 26% of the yield level force (E_{y}) , respectively, for

El Centro and Nahanni earthquakes. The respective percentages for the square pier are 28 and 16. This means that under assessment earthquakes, plastic hinges will not likely form, as the response is significantly below the yield level. Obviously, if the assessment earthquakes were to be chosen at higher levels of acceleration coefficients, such a conclusion would not be valid. However, to mark these piers as seismically deficient for 0.15g level assessment earthquakes and requiring seismic retrofit seems overly conservative.

The axial load demand (gravity plus earthquake effect) in the circular pier is approximately 1204 k(5355 kN) and 1068 k (4750 kN), respectively, under El Centro and Nahanni earthquakes. The balanced P-M pairs for the circular column bottom section is $M_{h} = 21,151 \ k$ -ft (28,680 kN-m) and $P_{h} = 4,132 \ k$ (18,379 kN). Considering the balanced axial force level of 4132k (18,379kN) for this pier, the demand level is respectively, 29% and 26% of the balance level, respectively for El Centro and Nihhani earthquakes as illustrated in Figure (12). The respective percentages for the square pier are 14 and 12 as shown in Figure (13). It is generally understood [8] that closely spaced hoops are necessary if the axial force level is larger than 40% of the balanced level. Under the selected 0.15g assessment earthquakes, the demand axial force levels are below 30% of the balanced force. Therefore, from this aspect, too, it does not seem justified to require any retrofit measures on the basis of hoop spacing requirements. It would be necessary to perform more detailed analysis of a three dimensional model under the action of several ultimate level earthquakes to study the deformation capacity and demand more accurately and obtain a more detailed seismic assessment of the selected bridges. In particular, for a more complete vulnerability analysis, it would be necessary to also evaluate shear capacity and demand. However, this latter aspect was not an objective of this paper.

As a follow up to this study, Chendana [3] undertook the task of performing three-dimensional pushover analysis of the two bridges. He compared the performance of the square and circular piers in these two existing bridges with 12 *in*. (305 *mm*) hoop spacing and hypothetical cases of 4.0 *in*. (102*mm*) hoop spacing. He developed estimates for displacement ductility factors for the two cases of hoop spacing. One of his results is that the behavior of the circular and square piers would be the same for the two different hoop spacing as long as the behavior is in the



Figure 12. Column Interaction Diagram for Circular Piers. (Note: 1 k = 4.448 kN).

elastic range. Another result is that for 12 *in*. (305*mm*) hoop spacing, both the square piers and circular piers will have approximately the same ductility capacity on the order of 3.5, based on three-dimensional analysis of the bridges. Readers interested in further results of the pushover analysis of these bridges are referred to the work of Chendana [3].

5. Conclusions

Preliminary results presented by this study in ate that many of the apparently inudeque bridges (with respect to hoop spacing) in lder oridges, with an acceleration coefficient in the large of 0.05 to 0.15, will perform within the ela c range under assessment earthquakes scilled to bese peak ground accelerations. The analysi results indicate that for the two bridges studie under me action of two 0.15g level earthqua is the isomand level lateral force \sim % of the yield level, and that would be less than the axial force level cemand would be smaller than about 30% of the balanced axial force level. No significant difference as to the behavior of circular versus rectangular piers under this level of seismic input was observed. This turned out to be the case even though the considered pier with square section had a height 2.5 times that of the pier with circular section. Under the seismic load level considered, it is overly conservative to require any retrofit measures on the basis of transverse reinforcement spacing. Further detailed analysis under ultimate level earthquakes would be necessary if any justification for upgrading the piers in these bridges is to be found. Of course, other force (e.g., shear) and deformation



adequacy aspendent these bridges should be done as well pop spacing.

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