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

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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 0.15*g* peak 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.

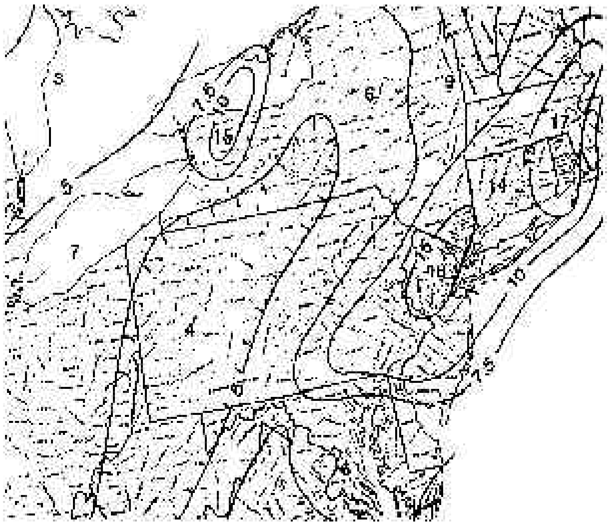

Figure 1. Part of AASHTO 1996 acceleration coefficient map – 80 - 90 percent probability of not being exceeded in 50 years.

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* (29.91*m*) - 77 *ft* (23.47*m*) - 85 *ft* (29.91*m*)) with four prestressed I-beam girders topped with composite reinforced concrete deck. Figure (2) shows partial elevation and a superstructure section for the bridge with square piers, while Figure (3) shows the partial bent elevation for a bent with pier height of 25 *ft* (7620*mm*) and a typical pier cross section (4 *ft* x 4 *ft* (1219*mm* x 1219*mm*)). The bridge has two double pier bents. The # 4 (12.7*mm*) bar peripheral hoop spacing in the lowest 6 *ft* (1829*mm*) of the pier is 6 *in*. (152*mm*) and in the rest of the pier 12 *in*. (305*mm*). The octagonal hoops (cross ties), also # 4 (12.7*mm*) bar, are spaced at 12 *in*. (305*mm*). The bridge with circular pier is a two-span bridge (101 *ft* (30.78*m*) – 125 *ft* (38.10*m*)) 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* (2896*mm*). 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 (1372*mm*)). The circular piers in this bridge have circular # 4 (12.7*mm*) hoops at 12 *in*. (305*mm*) spacing throughout the height. The hoops are closed with peripheral laps. The AASHTO [1] seismic provisions require a maximum

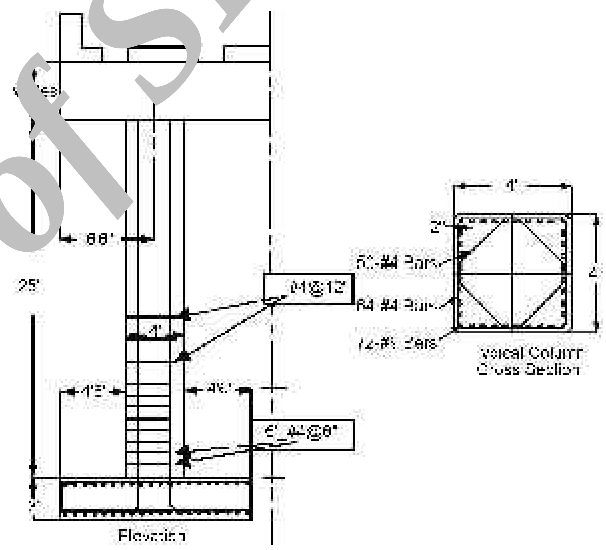

Figure 3. Partial bent elevation for one of the bents and typical pier cross section for the bridge with square piers. (Note: 1 ft = 304.8mm, 1 in. = 25.4mm).

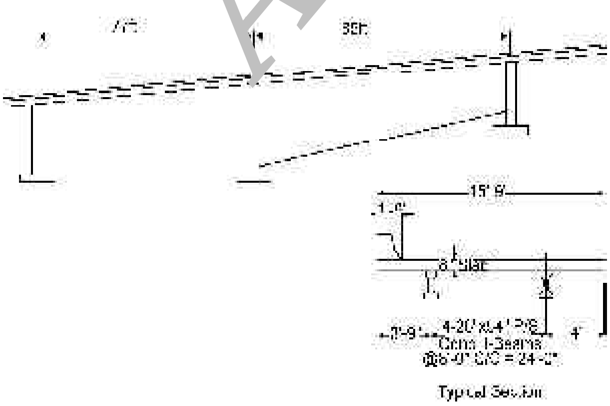

Figure 2. Partial elevation and superstructure section for the bridge with square piers. (Note: 1 ft = 304.8mm, 1 in. = 25.4mm).

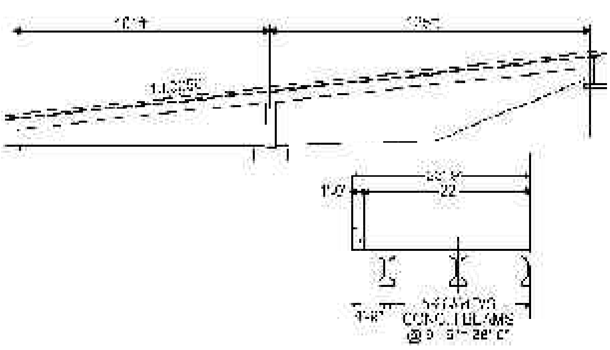

Figure 4. Partial elevation and superstructure section for the bridge with circular piers. (Note: 1 ft = 304.8mm, 1 in. = 25.4mm).

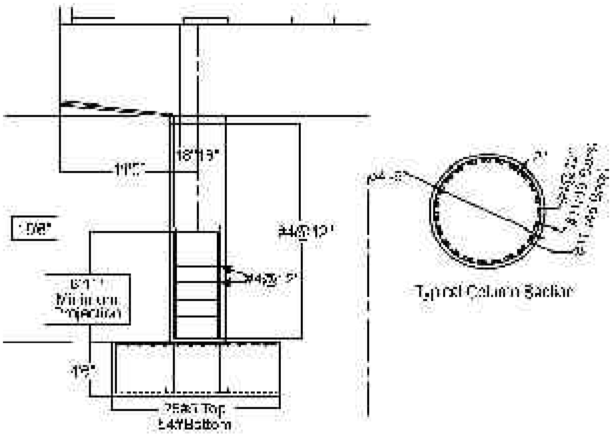

Figure 5. Partial bent elevation and typical pier cross section for the bridge with circular piers. (Note: 1 ft = 304.8mm, 1 in. = 25.4mm).

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 for these bridges than the 0.05g acceleration coefficient assigned (per *AASHTO* maps as shown in Figure (1)) to where they are located, it was decided to apply the higher contour acceleration coefficient of 0.15g, which can be considered as low to moderate seismic region for East Coast of the *U.S.* These prescribed maximum ground accelerations define design earthquake motions such that the probability of the elastic design force levels being exceeded in 50 years is in the range of 5 to 20%. In order to develop spectral accelerations, the May 19, 1940 *El* Centro (Imperial Valley Station 9, Magnitude (M_s) 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_s) 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

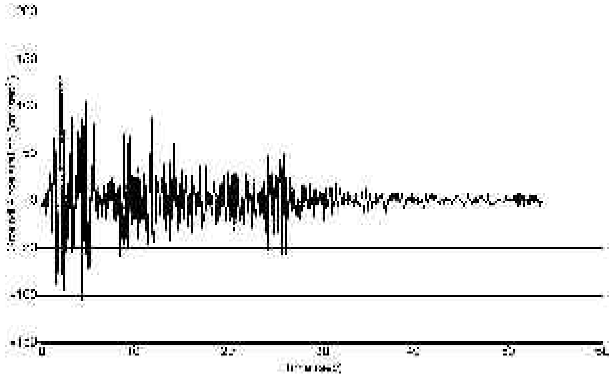

Figure 6. Acceleration-time history record for May 19, 1940 El Centro earthquake with peak ground acceleration scaled to 0.15g.

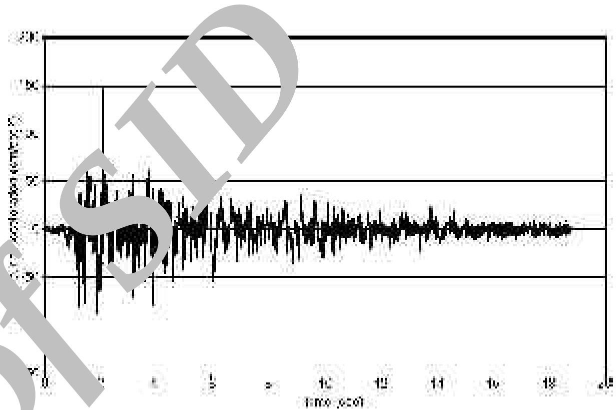

Figure 7. Acceleration-time history record for December 23, 1985 Nahanni earthquake with peak ground acceleration scaled to 0.15g.

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 0.15g will 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.

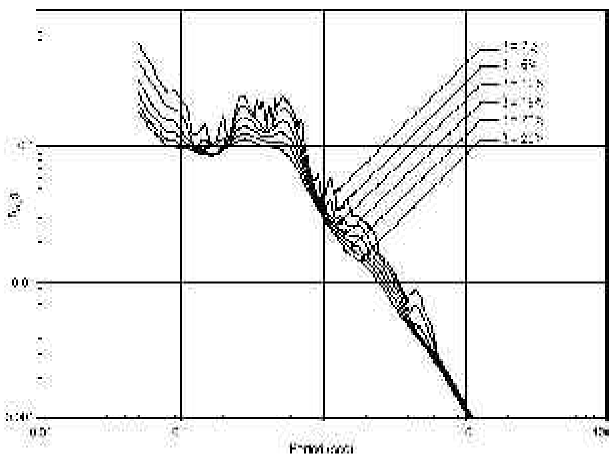

Figure 8. Derived response spectra for scaled (to 0.15g) El Centro earthquake.

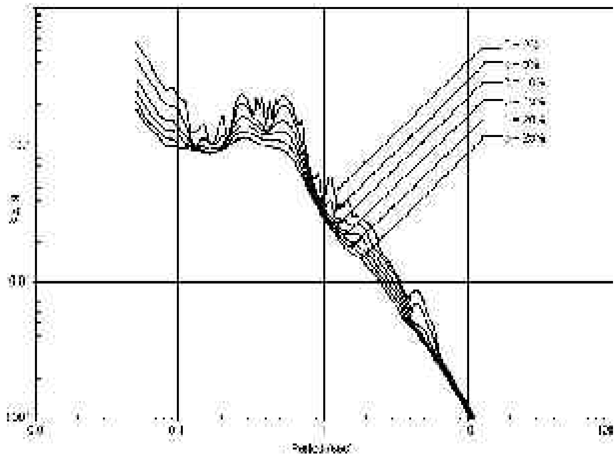

Figure 9. Derived response spectra for scaled (to 0.15g) Nahanni earthquake.

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 *D* are 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 design forces to be determined by dividing elastic seismic forces obtained from two orthogonal combinations by the recommended response modification factor, which for multiple column bents is 5.0. The resulting modified seismic design forces should then be combined with other applicable loads, e.g., dead loads, etc. If the two selected bridges were to be designed as new bridges, design forces would then be obtained based on the described procedure. For assessment purposes, however, an *FHWA* report [6] suggests an *SPC* of *C* instead of *B* for the above acceleration range if the bridge can be considered essential as opposed to standard, as shown in the lower part of Table (1). The difference between the two recommendations stems from the way importance 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.*

Table 1. Definition of Seismic Performance Category (SPC).

Seismic Performance Category (SPC) – AASHTO [1]		
Acceleration Coefficient	Importance Classification (IC)	
A	**I**	**II**
A $\leq$ 0.09	A	A
0.09 < A $\leq$ 0.19	B	B
0.19 < A $\leq$ 0.29	C	C
0.29 < A	D	C
Seismic Performance Category (SPC) – FHWA [6]		
Acceleration Coefficient	Importance Classification	
	Essential	Standard
A $\leq$ 0.09	B	A
0.09 < A $\leq$ 0.19	C	B
0.19 < A $\leq$ 0.29	C	C
0.29 < A	D	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 moment-curvature 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].

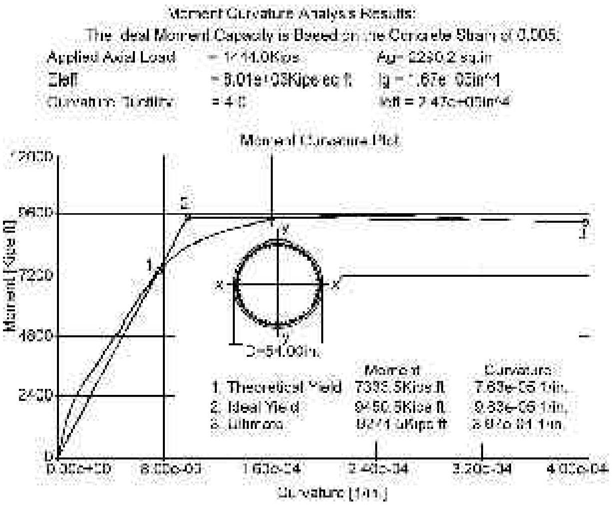

Figure 10. Example moment-curvature diagram for circular pier. (Note: 1 Kip-ft = 1.356kN-m, 1 k = 4.448kN, 1 ft = 304.8mm, 1 in. = 25.4mm)

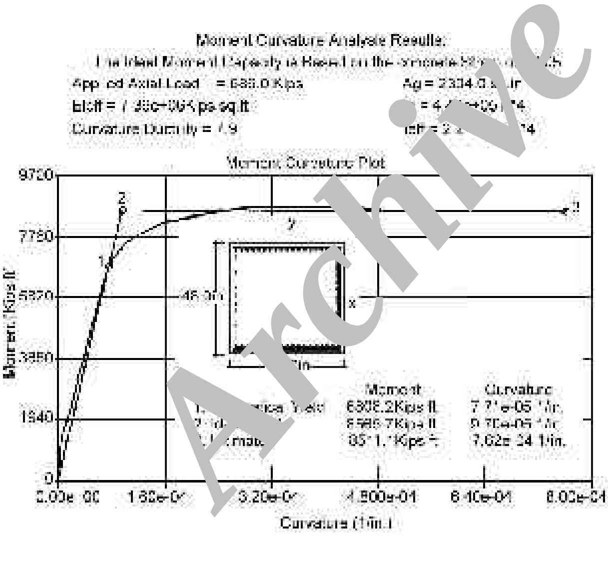

Figure 11. Example moment-curvature diagram for square pier. (Note: 1 Kip-ft = 1.356kN-m, 1 k = 4.448kN, 1 ft = 304.8mm, 1 in. = 25.4mm)

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_y) 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_y) 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_E) 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. Obviously, for more accurate analysis, a three dimensional finite element modeling should be used. The calculated axial forces were then combined with the dead loads to get more accurate moment curvature results. Recalculation of pier shear forces and subsequent lateral bent force after several iterations eventually establishes the lateral yield force level. This will be further discussed subsequently. The results are summarized in Table (2). The lateral bent forces E_y, calculated as per explained 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 I is the moment of inertia.

Table 2. Results of moment-curvature analysis.

	M_y	ϕ_y*10^{-5}	EI_{eff} *10^6	P_E	P_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).

	E_y	K_{eff}	W	T_0	S_a (g)		E_{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 moment-curvature 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 3rd 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 $K_{eff} = 24\,EI_{eff}/H^3$. 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_b = 21,151 *k-ft* (28,680 *kN-m*) and P_b = 4,132 *k* (18,379 *kN*). Considering the balanced axial force level of 4132*k* (18,379*kN*) 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

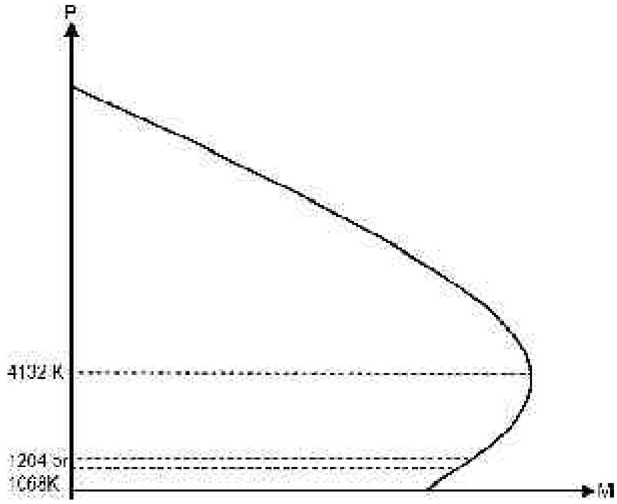

Figure 12. Column Interaction Diagram for Circular Piers. (Note: 1 k = 4.448 kN).

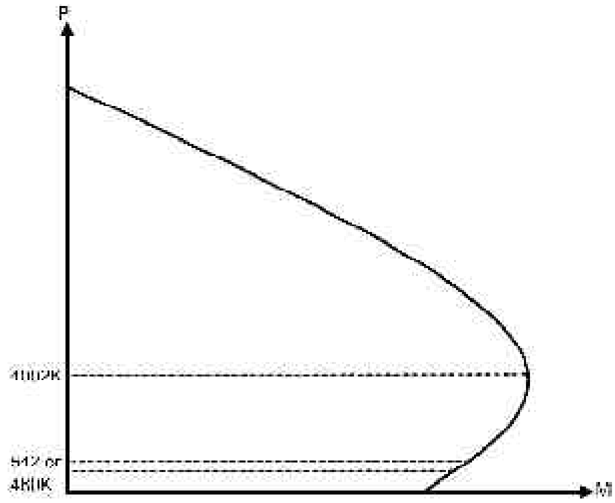

Figure 13. Column Interaction Diagram for Square Piers. (Note: 1 k = 4.448 kN).

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 indicate that many of the apparently inadequate bridges (with respect to hoop spacing) in older bridges, with an acceleration coefficient in the range of 0.05 to 0.15, will perform within the elastic range under assessment earthquakes scaled to these peak ground accelerations. The analysis results indicate that for the two bridges studied under the action of two 0.15g level earthquakes the demand level lateral force would be less than [illegible]% of the yield level, and that the axial force level demand 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 aspects of these bridges should be done as well as the hoop spacing.

Acknowledgment

The contribution of Aditia Darbari and Serdar [illegible]sta[illegible]oglu, who were graduate students at the time the study is acknowledged.

References

1. AASHTO (1996). "Standard Design Specifications for Highway Bridges", American Association of State Highway and Transportation Officials, Washington, D.C.

2. Atkinson, G.M. and Boore, D.M. (1995). "Ground Motion Relations for Eastern North America", *Bulletin of the Seismological Society of America*, 85(1), 17-30.

3. Chendana, J. (2003). "Static Nonlinear Pushover Analysis of Typical Pennsylvania Bridge Piers", Master of Science Thesis, Department of Civil and Environmental Engineering, The Pennsylvania State University, University Park, PA.

4. EERI (1995). The Hyogo-Ken Nanbu Earthquake January 17, 1995 Preliminary Reconnaissance Report, Earthquake Engineering Research Institute, Oakland, CA.

5. EERI (2001). The Nisqually, Washington, Earthquake February 28, 2001 Preliminary Reconnaissance Report, Earthquake Engineering Research Institute, Oakland, CA.

6. FHWA (1995). "Seismic Retrofitting Manual for Highway Bridges", Buckle, I.G. and Friedland, I.M. (Editors), FHWA-RD-94-052.

7. NEHRP (1988). "Recommended Provisions for the Development of Seismic Regulations for New Buildings", *National Earthquake Hazards Reduction Program*, Building Seismic Safety Council, Washington, D.C.

8. Park, R. and Paulay, T. (1975). "Reinforced Concrete Structures", John Wiley and Sons.

9. Paz, M. (1985). "Structural Dynamics: Theory and Computation", 3rd ed. Chapman and Hall, New York.

10. Priestley, M.J.N., Seible, F., and Calvi, G.M. (1996). "Seismic Design and Retrofit of Bridges", John Wiley and Sons.

11. Saadeghvaziri, MA. and Jones, LC. (1993). "Nonlinear Response of SDOF Systems to Intraplate Earthquakes", *Proceedings, Structural Engineering in Natural Hazards Mitigation*, ASCE Structures Congress, Irvine, CA.

12. SEQMC (1996). "SEQAD Moment Curvature Analysis Program", SC Solutions, Inc. Santa Clara, CA.

13. Strongmo Database System, http://www.ldeo. Columbia.edu/res/data/nceer/strongmo.