Seismic Evaluation of Reinforced Concrete Piers in Low to Moderate Seismic Regions

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ABSTRACT: Evaluation of reinforcement detailing with respect to the amount and spacing of transverse reinforcement, development length, and splice length based on seismic provisions for seven Pennsylvania bridges is discussed. The selection of prototype bridges as representative of the stock of all PA bridges for the study are explained. A case study of seismic evaluation of one of the bridges selected based on finite element analysis and ductility estimation is presented as representative of the approach used to evaluate all other selected bridges.

Keywords: Bridges, Reinforced Concrete, Seismic Evaluation, Low-Moderate Seismic Regions

1 INTRODUCTION

The damaging earthquakes of the 1990's in California underlined the vulnerability of existing bridges that have not been designed for seismic loads. This led to significant changes in the 15th and 16th editions of AASHTO seismic provisions (1992, 1996) with respect to detailing of reinforced concrete column confinement reinforcement. While based on the prior editions of AASHTO, designers specified transverse reinforcement of 13 mm at 305 mm spacing on centers, the 15th and 16th editions of AASHTO required the spacing of the transverse reinforcement for columns not to exceed 102 mm or 152 mm, depending on the Seismic Performance Category (SPC). Because most of the bridges in the eastern US, including Pennsylvania, with a variety of column geometry and bent types were built prior to the 1990's, the transverse reinforcement details in most existing bridges do not satisfy the 16th AASHTO seismic provisions, which was referred to at time of this study (Memari et al. 2001). The more recent AASHTO 17th edition (2002) also has the same seismic provisions (considered in this work) as that in the 16th edition with respect to the aspects considered in this paper. Faced with the question of possible need for seismic retrofitting of existing bridge columns, The Pennsylvania Department of Transportation (PennDOT), just as DOT's of some other states, was interested in determining whether reinforced concrete bridge columns with apparently deficient confinement reinforcement detailing should be retrofitted (FHWA 1995) or could they be regarded as acceptable in Pennsylvania.

The provisions for seismic design of bridges in AASHTO have been going through significant changes since the occurrence of the 1971 San Fernando Earthquake in California. A brief historical development of seismic provisions is discussed here. Notable changes occurred in the 1977, 1983, and 1992 AASHTO editions. While AASHTO 8th Ed. (AASHTO 1961) to 11th Ed. (AASHTO 1973) simply required consideration of a lateral seismic load equal to 2% to 6% (depending on foundation type and soil bearing capacity) of dead load "in regions where earthquakes may be anticipated," the 12th Ed. (AASHTO 1977) included seismic provisions much like those of the 1970's for buildings. The 12th edition of AASHTO prescribed the Equivalent Static Force Method, which took into account site seismicity (seismic risk map), soil dynamic properties, and structure dynamic response characteristics. For "complex structures" it recommended the Response Spectrum Method. The 13th Ed. of AASHTO (1983) included exactly the same seismic provisions as those in the 12th Ed. (AASHTO 1977), but it permitted the use of "*AASHTO Guide Specification for Seismic Design of Highway Bridges*" to be used as an alternative to the AASHTO specifications.

The largest departure in AASHTO 15th Ed. (1992) and later editions (AASHTO 1996, 2002) from the 13th Ed. (AASHTO 1983) is in detailing requirements, however. As an example, the 13th Ed. AASHTO requirements on ties for compression members state that "The spacing of ties shall not exceed the least dimension of the compression member or 12 inches." The seismic provisions for tie spacing in the 15th Ed. AASHTO and later editions, however, depend on Seismic Performance Category (SPC), which determines the level of sophistication of analysis required. According to the 15th Ed. AASHTO, the transverse reinforcement requirement for columns in bridges classified as SPC C or D should satisfy the following requirement: "The maximum spacing for reinforcement shall not exceed the smaller of one-quarter of the minimum member dimension or 4 in." For bridges classified as SPC B, it is permitted to increase the spacing to 152 mm. Such provisions impose very stringent requirements on column transverse reinforcement regardless of the design force level.

Given that most existing bridge columns do not satisfy the AASHTO detailing requirements, according to AASHTO, seismic response modification factors (larger than 1.0) cannot be used in the analysis of existing bridges for seismic vulnerability evaluation. This can result in bridge columns being thought of as overstressed when subjected to AASHTO prescribed seismic ground motions. This concern has been shared by many DOT's as is evident from similar studies carried out in other states (e.g., Mander et al. 1992, Hwang et al. 2000, DesRoches et al. 2003). The conventional solution approach is to retrofit bridge columns (e.g., using fiber reinforced polymers) for enhanced ductility capacity (e.g., Pantelides et al. 2004), whereby use of AASHTO-prescribed response modification factors would then be justified. Alternatively, if it can be shown that the apparently deficient existing columns possess sufficient ductility for earthquakes expected in low to moderate seismicity regions, then the use of some calculated values of response modification factor can be justified. This latter approach was taken to study the bridge columns in Pennsylvania (Memari et al. 2001).

Seismic evaluation of the selected bridges in this study included evaluation of reinforcement detailing in the substructure based on the 16th Ed. of AASHTO (1996), three-dimensional finite element modeling and analysis of the superstructure and the substructure, and static pushover analysis of the substructure. In the following sections, the process of selecting several prototype bridges, bridge descriptions, and pier reinforcement evaluation is discussed. This is followed by a discussion of the seismic evaluation of one of the bridges as an example case study based on finite element analysis as well as moment-curvature and pushover analysis to estimate response modification factor.

2 DESCRIPTION OF BRIDGES SELECTED FOR THE STUDY

The task of selecting prototype structures for detailed study involved using a screening approach to select seven bridges from the inventory of PennDOT bridges. The inventory of PA bridges at the time of the study was stored in the Bridge Management System (BMS), which included structure inventory, inspection, and appraisal data for every highway related structure in PA. Thus the task required selecting seven bridges from a total of 6132 bridges reported in the BMS inventory. Several criteria were considered to initially narrow down the selection to 50 bridges. The criteria considered are as follows: seismicity, importance, age, number of spans, type of pier, pier foundation types, type of superstructure, vertical clearance, and pier column cross-section type.

At a second level of screening, PennDOT engineers evaluated the suggested 50 bridges and, using their knowledge of the conditions of the bridges and other input, a short list of 12 bridges was determined. After studying the drawings for the 12 bridges, the research team, in consultation with PennDOT engineers, finally selected seven bridges as prototypes for detailed study. The selected bridges, which are representative of the majority of bridges in PA, are described briefly next. It should be noted that depending on the objectives of the study and preference of a given DOT, different screening approaches may be used. For example, Hwang et al. (2000) used NBIS/FHWA recording and coding guide (FHWA 1998) to classify the stock of 452 bridges. This classification defines bridge types in accordance to their superstructure type, material, and the continuity of supports. Hwang et al., however, added bent or pier information to the classification as well.

The selected bridges are described briefly in the following and some details are provided in Table 1, while a photograph of each bridge is shown in Figure 1:

- *Bridge No. 1: 28-Span Steel Plate Girder Bridge in Philadelphia County*: This bridge is 662.94 m long and has 27 spans for the southbound lanes and 28 spans for the northbound lanes. Due to the discontinuity of the superstructure, the bridge is divided into three segments referred to as Area A, Area B, and Area C. Area A starts at the south end abutment with two individual parallel spans carried on reinforced concrete bents. The two side-by-side spans then merge into one continuous span with steel bents (cap beams) and reinforced concrete columns to carry both northbound and southbound lanes. The last portion of this bridge, Area C, reverts back to parallel side-by-side span, which are similar to those in Area A.

Table 1. Description of Selected Bridges.

Bridge #	Girder Number, Type	Type of Column/Bent	Column Size mm	Column Reinforcement		Foundation Type	Soil Type
				Longitudinal	Transverse		
1-A	9 Steel Girders	Conc. Cap Beam/2- & 3-col. Bents	Tapered: 914x914 T; 1524x914 B	29 mm	13 mm @ 305 mm spacing	Spread Footing on Steel Pile	Silty Clay
1-B	9 Steel Girders	Steel Cap Beam/2-col. Bents	Tapered: 1067x1524 T; 1524x1524 B	≥35 mm	16 mm @ 305 mm spacing	Spread Footing on Steel Pile	Silty Clay
1-C	9 Steel Girders	Conc. Cap Beam/2- & 4-col. Bents	Tapered: 914x914 T; 1524x1524 B	29 mm/36 mm	13 mm @ 305 mm spacing	Spread Footing on Steel Pile	Silty Clay
2	2 Deep Steel Girders	Conc. Cap Beam/2-col. Bents	Stepped: 1829x1829 T; 3658x3658 B	36 mm	13 mm @ 305 mm spacing	Spread Footing	Sandy Clay/Gravel to Sandstone
3	Precast Concrete I-Beams	Conc. Cap Beam/3-col. Bents	Square: 1067x1067	29 mm/36 mm	13 mm @ 305 mm spacing	Spread Footing	Sily Sand/Clay to Very Stiff Sily Clay/Sandstone
4	Precast Concrete I-Beams	Conc. Cap Beam/3-col. Bents	Square: 1067x1067	32 mm/36 mm	13 mm @ 305 mm spacing	Pile Caps on Steel Pile	Silt and Clayey Sand
5	Steel I-Beams	Conc. Cap Beam/5-col. Bents	Circular: 762 dia.	29 mm	13 mm @ 305 mm spacing	Continuous Spread Footing	Sandy, Silty Clay w/ Rock Fragments
6	Precast Concrete I-Beams	Conc. Cap Beam/1-col. Hammerhead	Circular: 2438 dia.	36 mm	13 mm @ 305 mm spacing	Spread Footing on Steel Pile	Silty Sand/Gravel to Sandstone
7	Precast Concrete I-Beams	Conc. Wall Piers	Wall Pier: 914x7315	16 mm vert.	13 mm horiz. @ ≥305 mm spacing	Spread Footing	Sandy, Silty Clay w/ Rock Fragments

- *Bridge No. 2: Seven-Span Steel Deck Girder Bridge in Somerset/Cambria Counties*: The structure is a 422.15 m long seven-span steel girder bridge. The concrete deck is supported by longitudinal steel stringers, which in turn are supported on 1524 mm deep floor beams, where floor beams are supported on two main longitudinal steel girders 3810 mm deep.
- *Bridge No. 3: Dual Five-Span Prestressed Concrete I-Beam Bridge in Mercer County*: This bridge is a dual five-span structure consisting of separate northbound and southbound lanes supported on precast concrete I-beams and reinforced concrete diaphragms.
- *Bridge No. 4: Five-Span Three-Column Bent Pretressed Concrete Bridge in Bucks County*: This is a straight five-span precast prestressed concrete bridge with each pier consisting of two adjacent bents.
- *Bridge No. 5: Four-Span Steel Plate Girder Bridge in Lehigh County*: This bridge is a skewed and straight four-span steel I-beam supported on three piers consisting of five-column bents.
- *Bridge No. 6: Nine-Span Prestressed Concrete I-Beam Bridge in Huntingdon County*: This is a 382.22 m long, nine-span bridge straight over four spans and horizontally curved over the remaining five spans. The superstructure, which consists of precast prestressed I-beams, is supported on single-column hammerhead piers. Of the total eight piers, two of them at one end of the bridge are wall type piers.
- *Bridge No. 7: Curved Seven-Span Prestressed Concrete I-Beam Bridge in Tioga County*:This bridge has a curved seven-span superstructure consisting of a concrete slab supported on eight precast prestressed concrete I-beams. The substructure includes six wall piers of hammerhead type besides the two abutments.

Site visits of the prototype bridges revealed the existing physical conditions of the substructure and superstructures. Most bridges inspected were in good physical condition, while some had various degrees of deterioration such as spalling of concrete and exposed reinforcement, corrosion of main reinforcement, and fracture/spalling of pedestals on top of cap beams supporting girders. Although the existing bearings have performed well for temperature purposes, some of the bridges with deteriorated seat conditions were identified as deserving special attention for possible retrofit.

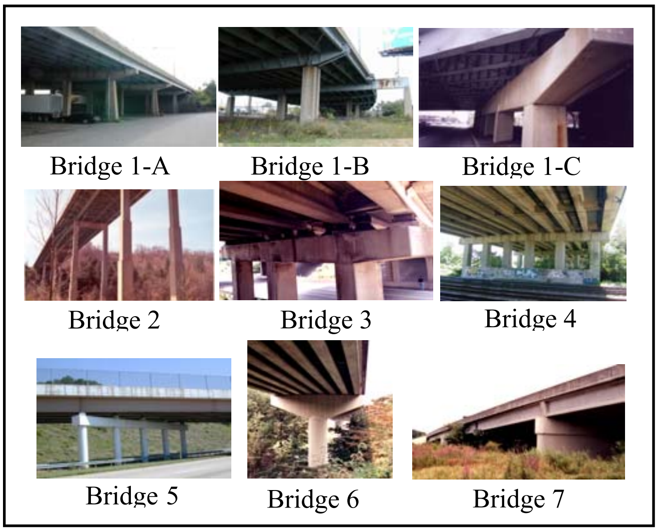

Figure 1. Photographs of Bridges Studied.

3 EVALUATION OF EXISTING REINFORCEMENT

Seismic performance of bridge columns would depend on the detailing of reinforcement, in particular, with respect to column tie spacing, development length and splice length. Following the discussion of AASHTO seismic provisions of the three detailing aspects, the corresponding existing lateral and longitudinal reinforcement in typical columns in the selected bridges was evaluated. Section 6 of AASHTO Division I-A (1996) for seismic design specifies the design requirements for reinforced concrete bridges based on Seismic Performance Categories (SPC). For bridges in SPC B, C, and D, the same minimum area for hoop reinforcement that should be provided in the columns of the bents is specified. The total cross-sectional area of hoops including supplementary cross ties, A_{sh}, within a maximum vertical spacing of "a" and crossing a section having a core dimension of h_c, should be obtained using the following equations, whichever gives a larger value:

$$A_{sh} = 0.30\, a\, h_c\, (f_c'/f_{yh})\, (A_g/A_c - 1) \tag{1}$$

$$A_{sh} = 0.12\, a\, h_c\, (f_c'/f_{yh}) \tag{2}$$

where "a" is the vertical spacing of stirrups in inches, A_c is the area of column core measured to the outside of the transverse spiral reinforcement, A_g is the gross area of column, f_c' is the specified compressive strength of concrete in psi, f_{yh} is the yield strength of hoop or spiral reinforcement in psi, and h_c is the core dimension of tied column in the direction under consideration. The maximum spacing, however, is different for different SPC categories – 152 mm for SPC B and 102 mm for SPC C and D.

For bridges in SPC B with circular columns, the volumetric ratio of spiral reinforcement (ρ_s) shall be the larger of the following two equations:

$$\rho_s = 0.45\, [A_g/A_c - 1]\, (f_c'/f_{yh}) \tag{3}$$

$$\rho_s = 0.12\, (f_c'/f_{yh}) \tag{4}$$

where A_c is the area of column core measured to the outside of the transverse spiral reinforcement, A_g is the gross area of column, f_c' is the specified compressive strength of concrete in psi, f_{yh} is the yield strength of hoop or spiral reinforcement in psi. Based on recommendations by Priestley et al. (1996), to account for more realistic material properties in the analysis, a factor of 1.1 was used for f_y and a factor of 1.3 for f_c'.

AASHTO Division I-A (1996) Section 6 does not mention any special provisions with respect to development length for SPC B bridges, the implication being that the normal development length provisions of Section 8.25 in Division I are acceptable. For SPC C and D, Section 7.6.4 in Division I-A requires the use of $1.25f_y$ instead of f_y in the development length equations of Section 8.25 in Division I. For bridges in SPC B, there is no need for the 1.25 factor. According to Section 8.25.1 in Division I, the basic development length (L_d) for 36 mm bar and smaller is

$$L_d = 0.04\, A_b f_y / (f_c')^{1/2} \quad \text{(for straight bars) and}$$

$$0.02\, A_b f_y / (f_c')^{1/2} \quad \text{(for hooks)} \tag{5}$$

The development length for 43 mm and 57 mm bars is $0.085 f_y / (f_c')^{1/2}$, and $0.11 f_y / (f_c')^{1/2}$, respectively.

AASHTO Division I-A (1996) Section 6 does not mention special requirements for splice length for SPC B bridges. However, Section 7, which applies to SPC C and D, requires that lap splices be used only within the center half of the height of columns with a minimum length of 406 mm or 60 d_b, whichever is greater. Furthermore, it requires a maximum tie spacing of 102 mm or ¼ of the minimum member dimension (whichever is smaller) over the length of the splice. In the case of bridges with SPC B, we need only consider the lap splice length required in Section 8.32.3 in Division I. Accordingly, the required lap splice length in tension for the most critical Class C splice is 1.7 l_d.

Based on these criteria, the result of the evaluation of typical columns in the selected bridges is listed in Table 2 and summarized as follows: 1) The amount and spacing of transverse reinforcement in all the bridges considered do not meet the AASHTO requirements; 2) The embedment lengths of longitudinal reinforcement into cap beams are found to be generally sufficient for most bridges; 3) The splice length of longitudinal reinforcement at

Table 2. Evaluation of Reinforcements for Selected Bridges.

Bridge #	A_{sh} (mm^2) or ρ_s		Hoop Spacing (mm)		Development Length (mm)		Lap Splice Length (mm)	
	Required	Provided	Required	Provided	Required	Provided	Required	Provided
1-A	5239mm^2	258mm^2 $\therefore$NG	152	305$\therefore$NG	613	533 $\therefore$NG	1042	864 $\therefore$NG
1-B	2897mm^2 to 3458mm^2	413mm^2 $\therefore$NG	152	305$\therefore$NG	956	1448$\therefore$OK	1625	N/A
1-C	3329mm^2 to 5281mm^2	258mm^2 to 400mm^2 $\therefore$NG	152	305$\therefore$NG	345 to 432	533$\therefore$OK	1042 to 1625	864 $\therefore$NG
2	6413mm^2	387mm^2 to 516mm^2 $\therefore$NG	152	305$\therefore$NG	762	2286$\therefore$OK	1295	1219 $\therefore$NG
3	3955mm^2	258mm^2 $\therefore$NG	152	305$\therefore$NG	483 to 762	914 to 1016$\therefore$OK	813 to 1295	1321 to 1549
4	3884mm^2	258mm^2 $\therefore$NG	152	305$\therefore$NG	778 to 956	914 to 1067$\therefore$OK	1323 to 1762	991 to 1092 $\therefore$NG
5	0.0156	0.0029$\therefore$NG	152	305$\therefore$NG	490	914$\therefore$OK	1041	1905$\therefore$OK
6	0.0095	0.0007$\therefore$NG	152	305$\therefore$NG	1448	2972$\therefore$OK	2464	2972$\therefore$OK
7	6742mm^2	258mm^2 $\therefore$NG	152	305$\therefore$NG	254	838$\therefore$OK	432	406$\therefore$NG

the potential plastic hinge region in the column footing connection zone is sufficient for some of the bridges studied.

4 AVAILABLE DUCTILITY IN APPARENTLY DEFICIENT BRIDGES

In general, large amounts of transverse reinforcement would be necessary for plastic hinges to fully develop and provide large displacement ductility capacity for the column. For designs in low to moderate seismic regions where seismic input is not large enough to form plastic hinges, such requirements are overly conservative. Test results available in the open literature (mentioned subsequently) show that most of these types of columns can provide a displacement ductility of greater than 1.5. The implication of such test results for low seismic region should be considered in seismic evaluation of older bridges.

The question of how a bridge column with 13 mm transverse reinforcement at 305 mm spacing can be considered acceptable in a low to moderate seismic region such as Pennsylvania then is addressed by considering the background for the development of seismic provisions in AASHTO. The seismicity maps in the 16th Ed. of AASHTO (the same as 17th Ed.) indicate ground acceleration in Pennsylvania varying from 0.05*g* to 0.15*g*, with a 10% probability of being exceeded in 50 years, which corresponds to a return period of approximately 475 years. According to AASHTO, if the column transverse reinforcement spacing is larger than 102 mm (for SPC C), then that column can conservatively be assumed to possess no ductility capacity. A structural element (such as a column) or structural system such as a bent is considered to have a desirable seismic performance when, under cyclic loading, the load-displacement relationship shows stable hysteresis loops without degradation in strength and stiffness. Tested reinforced concrete columns with short lap splice length of longitudinal bars at the column-footing connection area, short development length of longitudinal reinforcement at the column-cap beam connection area, large spacing of transverse reinforcement and lap ends have generally indicated inferior load-displacement relationships, characterized by strength and stiffness deterioration. In other words, if a column has splice length that is not sufficient for full development of longitudinal bar strength, the column will have bond failure (pullout of bars) regardless of the spacing of ties when subjected to cyclic lateral force. Such a column then offers little ductility capacity. Based on the results of such experimental studies, seismic provisions recommend different response modification factors for different structural systems. To be on the conservative side, the 16th Ed. of AASHTO recommends an R-factor equal to one to be used if the required detailing is not provided. However, if it can be shown that even a column with 13 mm at 305 mm hoop spacing does possess some ductility capacity, then it can be shown that R-values larger than one may be used for the analysis.

Literature review (Chai *et al.*, 1991; Priestley *et al.*, 1994; Saadatmanesh *et al.*, 1996; Sexsmith *et al.*, 1997; Seible *et al.*, 1997; Jaradat *et al.*, 1998; Xiao et al., 1999; and Daudy and Filiatrault, 2000) shows that if a column has 13 mm at 305 mm hoops and insufficient lap splice, that is 20 d_b (according to older editions of AASHTO) or less, then that column will have a very small displacement ductility capacity, on the order of 1.5. However, if the same column has continuous longitudinal reinforcement, i.e., no splice at potential plastic hinge region, then the column will show displacement ductility factors even on the order of 4 to 6. The length of lap splice in the AASHTO provisions is based on the ability of the bar to develop yield strength without bond failure. If the columns satisfy the current AASHTO requirements for lap splice length, then one can assume that the behavior of the column would be similar to the one with continuous longitudinal reinforcement at potential plastic hinge region. Therefore, based on available experimental evidence, one can deduce that such columns with 13 mm at 305 mm hoop spacing will have reasonable ductility capacity. Other similar studies seem to point to similar conclusions. For example, the study by Mander et al. (1992) states that although bridges in the eastern U.S. may have poor detailing for seismic, nonetheless, the gravity-load designed bridges can still have considerable lateral load resistance and ductility capacity. Based on the results of cyclic lateral testing of a ¼ scale model bridge pier with hoops at 305 mm spacing, their study showed that the pier behaved in a ductile manner even with such large hoop spacing.

To illustrate the next step in the evaluation process, one of the bridges introduced is subjected to the AASHTO prescribed seismic loading to determine the modal superposition force and displacement responses. The bridge is then further studied by estimating the displacement ductility and response modification factor through pushover analysis.

5 FINITE ELEMENT MODELING AND ANALYSIS OF BRIDGE NO. 5

This section describes an example of the analytical study carried out for one of the selected bridges. Specifically, the finite element modeling and elastic analysis of Bridge No. 5 is discussed. Designed in 1984, this bridge is a straight four-span steel I-beam and girder bridge with a skew angle of approximately 34°46'37.2". It has a 11.58 m wide deck consisting of a 203 mm thick reinforced concrete slab that is carried on five steel I-beams and steel plate girders. The span lengths are approximately: 24.69 m, 45.72 m, 45.72 m, and 16.15 m. The superstructure is carried on three piers consisting of five round column bents on combined footing foundations and two concrete abutments. Figure 2 shows plan and elevation views of the bridge, while Figure 3 shows details of one bent.

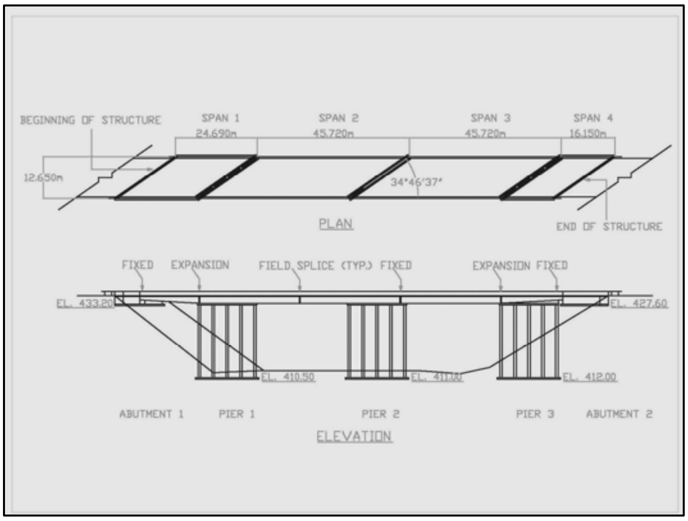

Figure 2. Plan and Elevation of Bridge No. 5.

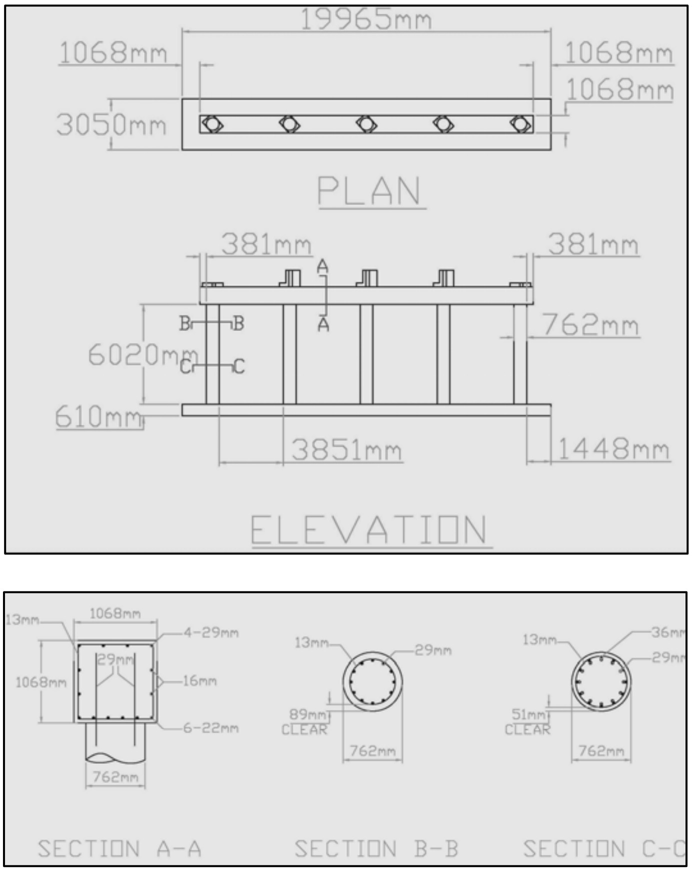

Figure 3. Details of Piers 1 and 3 in Bridge No. 5.

At Piers 1 and 3, expansion bearings allow longitudinal horizontal movement but restrain transverse motion. Fixed bearings at Pier 2 restrain the transverse and longitudinal motion but not the rotation. The concrete for the piers is Class A cement concrete with a 28-day strength of 22754 kPa. An increase in strength with age for the concrete can be expected as discussed in Bowles (1996). The plausible compressive strength of the concrete in the piers is approximately 30338 – 31027 kPa, according to Neville (1996).

Most of the references that incorporate computer modeling of highway bridge structures for seismic analysis use different finite element modeling techniques. Some of the recent references relevant to modeling aspects of this study include Caner et al. (2002), Abeysinhe et al. (2002), DesRoches et al. (2004), Bolten et al. (2005), and Moroni et al. (2005). Different computer programs have been used in the cited works. In the work presented here, the SEISAB (SEISmic Analysis of Bridges) computer program, Version 4.3, developed by Imbsen & Associates, Inc. (SEISAB 1999) was used to perform a multimode spectral analysis in line with the requirements of AASHTO Division I-A recommended procedures. The superstructure and substructure input data are listed in Table 3, while the finite element model is shown in Figure 4. The moment of inertia of the column sections were computed for the SEISAB model using I_g (gross section properties). Additional runs were made using the effective section properties I_{eff} (ATC 1996) to compare the results since the stiffness of columns reduces substantially after cracking develops.

Table 3. Section Properties for Bridge No. 5.

	Superstructure			Substructure
	Span 1	Spans 2 & 3	Span 4	Columns
L or H (m)	24.690	45.720	16.150	6.02
A (m^2)	4.517	5.434	4.216	0.456
$I^{1-1}(m^4)$	0.072	0.072	0.072	0.033
$I^{2-2}(m^4)$	76.535	89.503	72.589	0.017
$I^{3-3}(m^4)$	1.000	2.392	0.725	0.017
f'_c(kPa)	22753	22753	22753	22753
E_c(kPa)	2152400	2152400	2152400	21512400

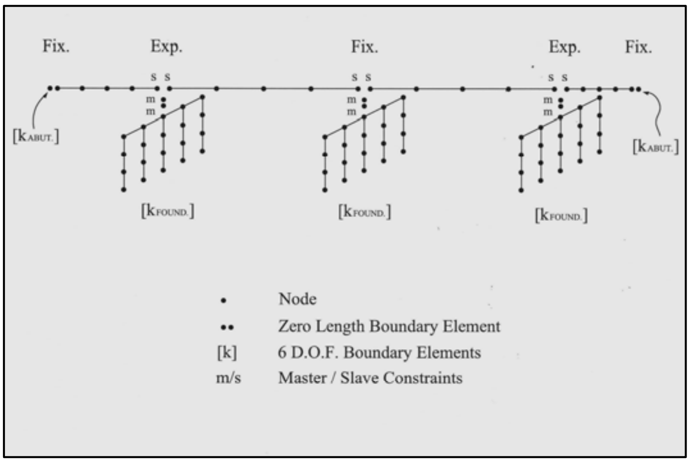

Figure 4. Finite Element Model of the Bridge.

The values of friction coefficient for the steel/bronze interface at expansion bearings were assumed to be 0.2 in the longitudinal direction and 0.5 in the transverse direction. These values were based on a study by Mander et al. (1993) who obtained coefficient of friction at the steel/bronze bearing interface from test results on a 30-year old bridge and associated laboratory tests of the original bridge components. A component analysis was performed to simulate the central bent with two full spans with different support conditions using SEISAB. This determined the maximum displacement at the expansion joint under a single longitudinal mode loading. The friction load was determined from the static analysis. Other input requirement included number of mode shapes to be specified, soil type, acceleration coefficient, and damping coefficient. The recommended number of vibration modes contributing to the response is usually taken as three times the number of spans. However, it is recommended that the number of modes be increased such that the percent modal mass is as close to 90% as possible for both the longitudinal and transverse directions.

The ground acceleration used was 0.15 *g* based on the AASHTO specifications (1996). Based on the ground acceleration coefficient and the importance classification, this corresponds to a seismic performance category (SPC) B bridge. The soil profile is Type I based on the soil boring data provided in the drawings. The present bridge with its high skew angle and irregular geometry induces coupling in the three orthogonal coordinate directions. These coupling effects make it difficult to separate the modes of vibration into simple longitudinal and transverse modes. The SEISAB computer program allows for the determination of the various modes that contribute to the total response of the structure within the choices discussed above.

Three analysis models that have significance to the seismic analysis are discussed. A summary of the three analysis models discussed here is given in Table 4. Model 1 considers the bridge as a whole (global model). Under the Spectrum Analysis section, the ground acceleration for this model is based on the seismicity contour map of PennDOT (DM-4 1994) for Lehigh County with the value of 0.15*g*. The properties of the frame elements are computed based on the gross section properties. The type of soil is taken as Type I. Model 1 uses com-

Table 4. Description of SEISAB Analysis Models.

Model Number	Description
1	General Criteria with Fixed Supports, Gross Properties, and Ground Acceleration of 0.15g
2	Same as Model 1, Except for Effective Section Properties (I_{eff}=0.6 I_g)
3	Same as Model 1, Except for Effective Section Properties (I_{eff}= 0.4 I_g)

puted spring coefficients based on realistic soil properties at the abutments and pier foundations. The Response Spectrum Analysis used in Model 1 and all other subsequent dynamic analyses considers the following four load cases: 1) Load Case 1 – Longitudinal; 2) Load Case 2 – Transverse; 3) Load Case 3 - 1.0 * Longitudinal + 0.3 * Transverse; and 4) Load Case 4 - 0.3 * Longitudinal + 1.0 * Transverse.

In analysis Model 2, the 0.15 *g* ground acceleration is maintained and all other properties are as in Model 1. However, the properties of the columns change from gross section to effective moment of inertia because of cracking, which is a function of the level of axial load. Values ranging from 40% to 60% of the gross inertia have been recommended by Paulay and Priestley (1992). A 40% reduction in gross moment of inertia has been used in Model 2 (i.e. $I_e = 0.6\ I_g$). The resulting fundamental periods for all three models are listed in Table 5. The results show an increase in the longitudinal fundamental period from 1.072 sec. in Model 1 to 1.259 sec. in Model 2 as expected. In analysis Model 3, a 60% reduction in gross moment of inertia at all columns is used (i.e., $I_e = 0.4\ I_g$). An increase in fundamental period from 1.259 sec. to 1.436 sec. is shown in the result. Another step of moment reduction in columns can also be found in the result of Model 3.

The extreme case at the expansion joints is the condition where the friction is so high that the bearing is "frozen" and no sliding at the interface plates can occur. In an earthquake strong enough to cause sliding at the interface of steel and bronze plates, friction forces develop during the motion and the resistance to motion modifies the structural characteristics. To model the influence of friction on the dynamic response, we can assume that friction increases the stiffness and determine an "equivalent" stiffness coefficient for the bearing. For linearly elastic analysis, the equivalent stiffness coefficient is estimated by dividing the friction force at the bearing by the displacement. For a coefficient of friction of 0.2, the breakaway friction force is 431 kN. The displacement 64 mm gives an equivalent stiffness coefficient of 6755 kN/m. For older bridges, the coefficient of friction has been found to be higher (Mander et al. (1993), Xanthakos (1994)) and thus the friction forces generated at the bearings is expected to be higher with the resulting equivalent stiffness coefficient also higher.

The results of absolute value (combined modal response) maximum displacements for each of the runs are given in Table 5. These include the maximum displacements at abutments and at the bents. The values of displacement listed are the maximum values based on the four different load cases considered. The maximum longitudinal displacement at the expansion joints was 73 mm in Model 3 (Table 4). This is less than the maximum design joint opening of 76 mm shown on the drawings. Therefore, based on the analysis results, superstructure displacements are not critical for the ground acceleration considered.

SEISAB provides axial load and torsion as well as shear and bending moment in both the longitudinal and transverse directions. Here, the focus is mainly on axial loads, bending moments and shear in the columns. Initially, static analysis was carried out to obtain the gravity effects followed by dynamic analysis. Comparison of column moments in different analysis models (Table 6) indicates that the results of Model 3, which accounts for the largest reduction of moment of inertia in columns, are the most plausible values for this bridge. Based on these considerations, the maximum moments are: $M_{long.}$ = 618 kNm occurring in bent 3, all columns, and $M_{trans.}$ = 681 kNm occurring in bent 4, column 2. The maximum axial force for Model 3 is P_{axial} = 688 kN in bent 4, column 2.

With the consideration of bending moments and axial loads from static gravity loading analysis, the new maximum moments and axial loads will be the summation of the results from gravity loading case and Model 3, which shows the M_{long} = 945 kN-m and M_{trans} = 1024 kN-m, both occurred at bent 4, column 2 with P_{axial} = 187 kN. The computed shear capacity of the columns (516 kN) is greater than the maximum shear force computed in Models 1-3 (225kN). Figure 5 shows the axial force vs. moment interaction diagram developed for the 762 mm diameter columns and for a concrete with f_c' = 30338 kPa and a reinforcement ratio of 1.7%. Based on the developed interaction diagram, the values for pairs of M_{long} - P_{axial} and $M_{trans.}$ - P_{axial} fall outside the ten-

Table 5. Summary of Results – Displacements; Abutment Forces (Bent Numbers Shown in Parenthesis).

Model Number	Found Period (sec.)	Abutment Displacement (mm)	Max. Longitudinal Bent Displacement (mm)	Max. Transverse Displacement (mm)	Abutment Foundation Spring Force		
					Longitudinal (kN)	Vertical (kN)	Transverse (kN)
1	1.072	47.9 (1)	48.2 (3)	41.8 (3)	834.9 (5)	166.8 (1)	907.4 (1)
2	1.259	60.0 (1)	60.4 (3)	48.8 (3)	919.8 (5)	157.9 (1)	952.8 (1)
3	1.436	72.5 (1)	73.2 (3)	54.9 (3)	994.6 (5)	162.4 (1)	999.5 (1)

Table 6. Summary of Results – Forces and Moments in Columns and Bents (Bent Numbers Shown in Parenthesis).

Model Number	Maximum Longitudinal Column Moment kNm	Maximum Transverse Column Moment kNm	Maximum Column Shear kN	Column Axial Force kN	Maximum Longitudinal Bent Force kN	Maximum Transverse Bent Force kN
1	1079 (3)	885 (3-2)	225.5 (3-2)	1132.5 (3-2)	1030.2 (3)	1098.7 (4)
2	800 (3)	750 (3-2)	203.7 (3-2)	1125.7 (3-2)	915.8 (3)	1032.5 (4)
3	618 (3)	681 (4-2)	193.0 (4-2)	687.7 (4-2)	840.7 (3)	1167.6 (4)

sion control region of the interaction diagram. Therefore, based on elastic dynamic analysis results (considering the moment-interaction diagram), the bent is overstressed, i.e., unacceptable performance.

However, the analysis indicates the high sensitivity of the resulting moment to the assumed moment of inertia of the columns. Of course, it should be noted that an R value of 1.0 was assumed for the analysis. If it can be shown that an R value greater than one is justified, the analysis results will likely be acceptable.

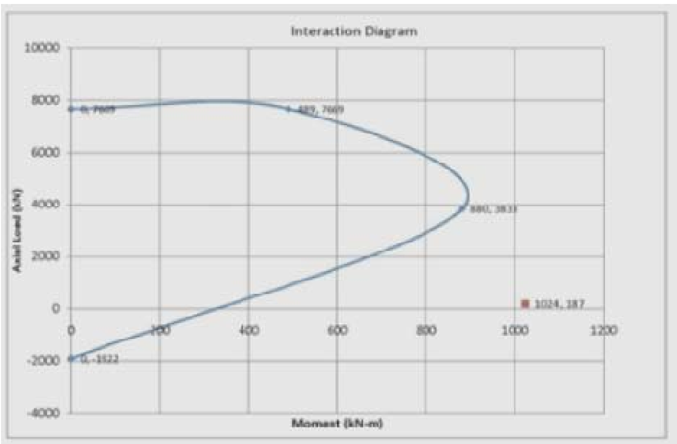

Figure 5. Column Interaction Diagram for a Typical Column of the Bridge.

6 EVALUATION OF DUCTILITY AND FORCE REDUCTION FACTOR

In order to determine a response modification factor R, the structure displacement ductility factor μ_Δ, which can be related to the member curvature ductility factor, must be estimated. There are different approaches to estimate a value for R. In this study (Memari et al. 2001), the approach proposed by Priestley et al. (1996) was followed. An overview of the approach is given here followed by presentation of the calculated results. Member plastic rotation capacity depends on the yield curvature (φ_y) and ultimate curvature (φ_u) values, which are defined as the ratio of corresponding compressive strain to neutral axis depth. An extensively used model for ultimate compressive strain capacity (ε_{cu}) is that developed by Mander et al. (1988), which was used in this study and expressed as follows:

$$\varepsilon_{cu} = 0.004 + 1.4\ \rho_s\ f_{yh}\ \varepsilon_{su} / f_{cc}' \tag{6}$$

where ρ_s and ε_{su} are the volumetric ratio of confinement reinforcement and its strain at maximum tensile strain, respectively.

The displacement ductility factor for a given bent depends on the plastic deformation at potential plastic hinge locations. The plastic deformation (e.g., rotation) can be expressed in terms of curvatures over the plastic hinge region. The plastic hinge length (L_p) can be expressed as follows (Priestley 1996):

$$L_p = 0.08\ L + 0.15\ f_{ye}\ d_{bl} \geq 0.3\ f_{ye}\ d_{bl} \tag{7}$$

where L is the length of column from the critical section to the point of zero moment and d_{bl} is the diameter of longitudinal reinforcement. The plastic hinge rotation capacity θ_p can then be determined as $\theta_p = L_p\ \varphi_p$ where the plastic curvature capacity φ_p, which is assumed to be constant over the plastic hinge length, is defined as the difference between the ultimate and the yield curvature values, i.e., $\varphi_p = \varphi_u - \varphi_y$. A moment-curvature analysis provides the necessary information. Based on the moment-curvature analysis results for critical sections (e.g., Figure 6), a bilinear approximation can be used to obtain the plastic rotation capacity. Since for a given section there is no well-defined yield point on the M-φ diagram, an equivalent yield point can be obtained by extending the line that connects the origin to the first-yield point (Point 1 (Figure 6)) to the point corresponding to the nominal moment capacity (M_n- φ_y) obtained by taking into account realistic material properties (Point 2). The (M_u- φ_u) point corresponds to the extreme fiber ultimate compression strain ε_{cu} (Point 3). The displacement ductility capacity of a bent μ_Δ can be obtained by relating the bent displacement to plastic hinge rotation capacity as follows (Priestley 1996):

$$\mu_\Delta = M_u/M_n + 3(\mu_\varphi - 1)(L_p/L)(1 - 0.5L_p/L) \tag{8}$$

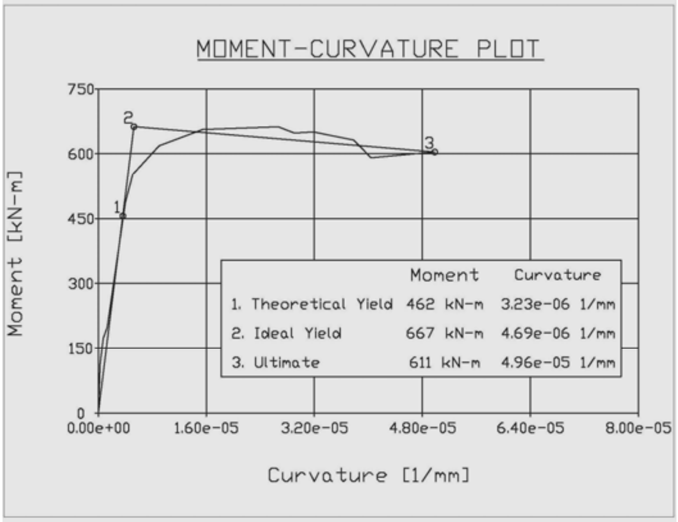

Figure 6. Moment Curvature Relation for the Center Column of the Bridge.

where the curvature ductility factor is defined as $\mu_\varphi = \varphi_u / \varphi_y$. Using the information obtained from *M-φ* analysis along with the plastic hinge length, one can estimate the displacement ductility capacity for the simple case of single-column bent. For multi-column bents, however, a collapse mechanism analysis is needed before the ductility factor can be estimated. The difference between the two cases is that in a single column bent, the collapse mechanism forms upon the formation of a plastic hinge at the bottom of the column. The sequences of plastic hinge formation must be determined by a series of linear elastic analyses to identify the critical collapse mechanism.

The procedure to determine the relationship between lateral load and corresponding displacement of a frame through the collapse mechanism conditions is based on a certain static push-over analysis procedure suggested by Priestley (1996). It should be noted that pushover analysis can also be performed by using a nonlinear static analysis software package that will perform the entire process (including P-Δ effect) automatically. However, the procedure used in this study (Memari et al. 2001) followed the approach suggested by Priestley (1996) and used a combination of hand calculation and linear elastic analysis. In short, in this procedure, gravity load analysis is carried out separately from lateral "unit" load analysis. Based on moment-curvature analysis for critical sections of the bent, the first section that reaches yield is identified by comparing capacity (reduced by gravity load moment) over demand ratios. The smallest ratio indicates where the first hinge will form. The moments from lateral unit load analysis are then multiplied by this smallest ratio and subtracted from capacity to obtain the "remaining" capacities. The lateral load and corresponding lateral displacement of the bent is then obtained and plotted. The bent computer model is then modified by placing a hinge where the first plastic hinge formed. Another lateral unit load analysis is performed and the process is repeated until all plastic hinges that yield a mechanism are identified; corresponding lateral load and displacements are also plotted. The ultimate bent displacement is then determined by adding the yield displacement to the plastic displacement which is a function of plastic rotation capacity (the smallest capacity chosen). The ultimate displacement is then plotted at the same lateral load as the yield load. Depending on the relative moment strength of the cap beam and columns, a beam side-sway or a column side-sway mechanism can form. Once the push-over analysis results and displacement ductility capacity for the bent have been obtained, a more refined seismic evaluation of the bridge can be made by determining a seismic response modification factor *R*.

The seismic response modification factor *R* can be obtained from consideration of the relation between a linearly elastic response and an elasto-plastic response of a single degree of freedom system. For very long period structures (say periods larger than 5 seconds), the relative displacement of the superstructure with respect to the ground is approximately of equal magnitude to that of ground displacement regardless of elastic or inelastic behavior. This is referred to as the equal displacement principle (EDP). For such cases, the force reduction factor can be assumed to be equal to the displacement ductility capacity or $R = \mu_\Delta$. For bridges with intermediate periods (say periods between 0.2 and 5 seconds), the maximum displacement in an elasto-plastic response will be usually larger than the elastic response. In such cases, the area under the two curves can be assumed to be equal to conserve the strain-energy. This is known as the equal energy principle (EEP). From the equality of areas under the two force-displacement diagrams, the force reduction factor can be obtained as $R = (2\,\mu_\Delta - 1)^{1/2}$. The force reduction factor determined by EEP is always smaller than that obtained by EDP.

For the five-column-bent bridge, a pushover analysis was performed to determine the displacement ductility and load reduction factor. Moment-curvature relations for the columns and beam cap were acquired using the SEQMC program (1989). Figure 6 shows a typical moment-curvature relation for one column of the bent. The static, non-linear analysis was performed using the STAAD/Pro program. The results of the static, non-linear push-over analysis and the moment-curvature capacities determined previously can be plotted as shown in the load-deflection curve (Figure 7). Relevant results of the push-over analysis are as follows: yield displacement = 17 mm; plastic hinge length = 593 mm; plastic hinge rotation = 1.16 %; plastic displacement = 61 mm; total displacement = 78 mm; and displacement ductility = 4.57. While based on

Figure 7. Load-Displacement Relationship for the Five-Column Bent.

EDP, the R-factor can be assumed 4.57, the more realistic estimate is based on EEP, which gives 2.85. Using an R-factor of 2.85 will reduce demand forces discussed earlier sufficiently to make the bridge be assessed as acceptable. Following this procedure, the R-factor was determined for all bridges except for Bridges number 4 and 7 since the elastic finite element analysis showed that the capacity exceeded demand. The R-factor determined for all other bridges are as follows: #1-A: 3.65; #1-B: 4.51; #1-C: 2.77; #2: 2.15; #3: 2.53; #5: 2.85; and #6: 1.88. Therefore, the r-factor for the bridges determined were all over 2.15, except for the single-column hammerhead type bridge (No. 6) that resulted in a value of 1.88. Details for all hand calculations and analysis results can be found in Memari et al. (2001).

7 CONCLUDING REMARKS

Although the focus of this study presented in this paper was on confinement effect on ductility, nonetheless, it should be emphasized that the even if the confinement is not critical, still the shear force carrying capacity is critical. Column shear failure is characterized by inclined cracking, concrete cover spalling, and rupture or opening of ties, followed by buckling of longitudinal reinforcement and disintegration of the core concrete (Seisble et al. 1997). In particular, in cases where squat shear-dominated bridge columns are used, it is likely that shear failure will precede the flexural failure, and thus, the ductility capacity up to the shear failure point can be expected to be smaller than the ductility in a bridge column with proportions leading to a flexural failure. Past studies have indicated that for columns with a relatively small aspect ratio, say height-to-diameter ratio on the order of 3, failure is mostly characterized as shear failure. In such cases, confinement reinforcement is not the critical issue.

Based on the results of this study, several specific conclusions can be stated:

- The transverse reinforcement in typical existing bridge column in PA violates AASHTO provisions on transverse reinforcement detailing requirements.
- The embedment lengths of longitudinal reinforcement into cap beams generally satisfy the AASHTO requirements.
- The splice lengths of longitudinal reinforcement at potential plastic hinge regions are not sufficient in some bridges.
- When bridges are analyzed dynamically based on AASHTO modal superposition requirements and the assumption of R=1.0 (because of transverse detailing non-conformance), the columns in most bridges will likely show unacceptable performance per axial load-moment interaction diagram.
- Literature review of experimental studies shows that columns with 13 mm hoop at 305 mm spacing and less than 20 d_b lap splice length have at least a displacement ductility capacity of 1.5.
- Pushover analysis of R/C bridge bents studied shows that response modification factors on the order of 2.0 and higher can be obtained. This conclusion is based on the analysis of a few bridges chosen for this study and it cannot be general conclusion for all PA bridges.
- The results of this study shows that retrofitting PA bridges on the basis of insufficient hoop spacing alone is not justified since by detailed analytical modeling it can be shown that the structures will not be overstressed under AASHTO prescribed seismic loading.

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