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Seismic Behavior and Capacity/Demand Analyses of a Simply-Supported Multi-Span Precast Bridge

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Abstract**—**This paper presents the results of an analytical study on the seismic response of a Multi-Span-Simply-Supported precast bridge in Washington State. The bridge was built in the early 1960's along Interstate 5 and was widened the first time in 1979 and the second time in 2001. The primary objective of this research project is to determine the seismic vulnerability of the bridge in order to develop the required retrofit measure. The seismic vulnerability of the bridge is evaluated using two seismic evaluation methods presented in the FHWA Seismic Retrofitting Manual for Highway Bridges, Method C and Method D2. The results of the seismic analyses demonstrate that Method C and Method D2 vary markedly in terms of the information they provide to the bridge designer regarding the vulnerability of the bridge columns.

Keywords**—**Bridges, Capacity, Demand, Seismic, Static pushover, Retrofit.

I. Introduction

A COMMONLY used type of bridges in Washington State (WA) in the 1950's and 1960's is Multi-Span-Simply-Supported (MSSS) precast system. This system was proven at the time to be easy to be designed and to be constructed. The designers used a pre-designed tabulated set of bridge components, where the superstructure and the substructure components are selected based on certain global characteristics and dimensions of the bridge such as, height of the columns, roadway width, bent skewness and span length. The pre-designed bridges always hold the same assumptions in terms of material strength. Then, modification to footings or other components might be introduced based on site conditions.

The superstructure of these bridges consist of prestressed concrete girders that are supported by concrete bents, which usually consist of a cap beam that is 3ft x 4.5 ft (0.91 x 1.37 m) supported by a number of circular columns, typically 3 ft (0.91 m). The number of the columns depends on the bridge skewness and the roadway width. Spread footings or footings on timber piles were the common footing systems used.

The MSSS prestressed concrete bridges built in Washington State in the 1960's were designed with little or no attention to seismic forces. Seismic provisions that guarantee appropriate ductility and acceptable seismic performance such as confinement, rebar splice length and girder seat-length were not considered. Only couple of girder stops where used to support the bridge deck against accidental lateral movement. Longitudinal restrainers were not commonly provided, as the contact length between the girders and the top of the cap beam was assumed to be sufficient to accommodate longitudinal displacement. Abutments were typically seat-type supported on strip footing or timber piles.

Recently, Washington State Department of Transportation (WSDOT) has commenced a comprehensive seismic analysis program to evaluate the vulnerability of the exiting bridges under a seismic event and to determine the most effective retrofit alternative.

The Bridge investigated in this study is one of the representative MSSS prestressed bridges built in WA. The bridge was built in the 1960's and widened twice from its both sides, in 1979 and 2001. Apparently, the last widening was conducted based on more conservative seismic design and detailing provisions, where confinement and lap-splice requirements were improved.

The primary objective of this research project is to determine the seismic vulnerability of the bridge in order to develop the required retrofit measure. The seismic vulnerability of the bridge is evaluated using two seismic evaluation methods presented in the Federal Highway Administration (FHWA) Seismic Retrofitting Manual for Highway Bridges [1], namely, Method C and Method D2.

II. Seismic Evaluation Approach

The seismic evaluation of the bridge was performed using the FHWA Seismic Retrofitting Manual for Highway Bridges Part 1 - Bridges published by the Multidisciplinary Center for Earthquake Engineering Research (MCEER) [1] for the Federal Highway Administration. Two methods of analysis were used: The first method is a linear elastic force based method of evaluation named Method C, which is used to find out the elastic demand. The second method of analysis is a non-linear static pushover analysis named Method D2 in the seismic retrofitting manual, which is used to find out the nonlinear plastic capacity of the structure.

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The Capacity/Demand ratios are then calculated for all relevant components, such that a Capacity/Demand ratio value less than one indicates a potential need for seismic retrofit of a structural element. The following is a brief discussion of the two methods:

A. Method C Analysis

Method C is used to determine the seismic demand by elastic analysis. For bridges with regular configuration multi-mode elastic response spectrum analysis is recommended. Time-history analysis is recommended for irregular bridge configuration; however the MCEER manual recommends that the multi-mode elastic response spectrum analysis could be used as minimum as well. Using this method the following components are to be investigated: seats, connections, columns, walls and footings.

The elastic response spectrum function used to find the demand is based on a 475-year design level earthquake with 5 percent damping. AASHTO LRFD [2] is used to construct the spectrum function with 0.3g Peak Ground Acceleration (PGA) and 1.2 site coefficient.

According to Method C Analysis, a rigors analysis is conducted to find the Capacity/Demand ratios of each component. The procedure starts by finding the elastic forces in columns and footings and comparing them with the corresponding ultimate capacity, such that the strength reduction factor is set to one. If plastic hinging occurs in any of these elements, splice length, anchorage and/or confinement failures shall be checked. The overall capacity-demand ratio is determined based on the lowest ratio calculated. Fig. 1 illustrates the procedure of finding the C/D for columns. The Capacity/Demand ratio for joints are also computed by comparing the elastic displacement demand to the corresponding displacement capacity as computed per FHWA Seismic Retrofitting Manual for Highway Bridges manual [1]. This mainly includes the C/D of seat-length.

B. Method D-2 Analysis

Method D-2 is a structural capacity-demand method, rather than a component capacity-demand method as in Method C. The capacity of the structure is determined by the means of non-linear static analysis, that is pushover analysis, where a detailed analysis of individual piers is carried out. The inelastic displacement capacity of a pier is directly related to its columns' ability to survive plastic hinging and to accommodate the plastic rotational demands within potential plastic hinge locations. Displacement demands could be estimated using elastic multi-mode response spectrum analysis.

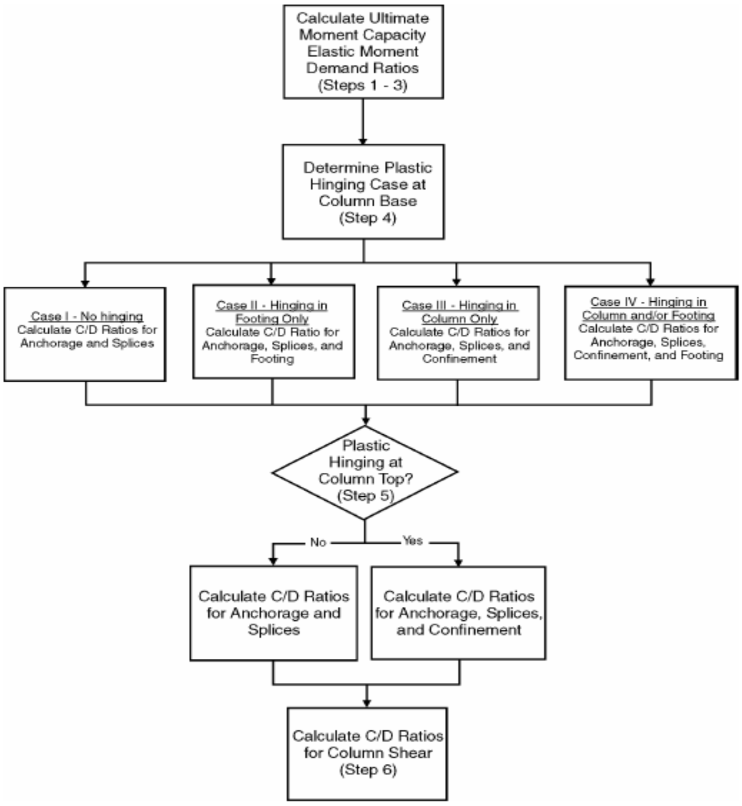

Fig. 1: Method C Column Analysis Flow Chart, Adapted from [1]

According to FHWA Seismic Retrofitting Manual for Highway Bridges [1], section 7.8.2, plastic hinging is formed due to certain local deformation limit states specifically, compression failure of concrete, buckling of longitudinal reinforcement, low-cycle fatigue, lap splice failure, and shear failure.

These limit states has been identified for each column within a pier. The limit state resulting in the least plastic rotation of a member is determined to be the controlling limit state. The potential plastic hinge location is also determined and implemented in the finite element model.

Since the plastic rotational capacity of the plastic hinge is proportional to the column curvature capacity, which is sensitive to the axial load on the column, five moment-rotation relationships are defined. These points are selected to be on the failure surface of the column interaction diagram, and they are: minimum axial capacity (pure tension), maximum axial capacity (pure compression), zero axial force (pure flexure), the maximum moment capacity (balanced failure), and the moment capacity corresponding to the axial load equals to the applied dead load. Constructing more than one moment-rotation relationship is very important when pushover analysis is conducted in the transverse direction, which is when frame action takes place. This issue becomes less important for MSSS bridges when the bent is pushed in the longitudinal direction, where frame action is insignificant and only the axial force due to dead load always prevailing.

Each bridge pier is pushed individually in the transverse direction until a column within the pier reaches its maximum inelastic curvature capacity. Similarly, the bridge is pushed in the longitudinal direction as well. The displacement demands obtained from the multimode elastic response analysis is used to find the displacement Capacity/Demand. If the C/D ratio is less than 1.0 then there is a possibility of a structural deficiency and a seismic retrofit is required. Although a column displacement Capacity/Demand ratio larger than 1.0 may be interpreted as an indication of the column's satisfactory performance under seismic loading, some column damage can be expected at these inelastic displacements, particularly when the inelastic displacement exceeds two times the yield displacement [3]

III. Bridge Description

Fig. 2 and Fig. 3 show a photograph of the bridge and how the precast girders are resting on the piers, respectively. The bridge consists of four simply supported spans totaling 322'-6" (101.3 m) in length. The original roadway width was 26' (7.92 m) with 3' (0.91 m) wide sidewalks on each side of the roadway. The original superstructure consisted of precast concrete girders with span lengths of 55'-6" (16.92 m), 100' (30.48 m), 100' (30.48 m), and 67' (20.42 m). The intermediate piers consist of three 3' (0.91 m) diameter columns founded on spread footings. The bridge abutments are seat type abutments, each supported by two tapered columns with a depth of 2' (0.61 m) by a minimum width of 2'-9" (0.84 m). At Pier 1 and Pier 5, each column is founded on a spread footing.

Fig. 2: Photograph of the Bridge

Fig. 3: Photograph of the Precast Girders at Pier 3

In 1979, the roadway was widened by 28' (8.53 m), to an overall width of 60' (18.29 m). The sidewalk on the north side of the bridge was removed as a part of this widening. The widening included the addition of six precast concrete girders. At each intermediate pier three 3' (0.91 m) diameter columns support the new girders. At Pier 2 to Pier 4, each new column is founded on a spread footing. The bridge's seat type abutments were widened while maintaining the same cross-section dimensions as the existing abutments. The widened portion of the abutment at Pier 1 is founded on two spread footings. The widened portion of the abutment at Pier 5 is founded on two footings and supported by creosoted timber piles.

In 2001, the bridge was widened again. The roadway was widened by 20'-8" (6.30 m), to an overall width of 80'-8" (24.59 m). Two 6'-7" (2.00 m) wide sidewalks on each side of the bridge were also added. The bridge superstructure was widened with four W50MG precast concrete girders added to the north side of the bridge over Spans 2 and 3 and three W50MG precast concrete girders added to the north side of the bridge over Spans 1 and 4. In addition, two W58MG precast concrete girders were added to the south side of the bridge over all of the spans. At each intermediate pier, three 3' (0.91 m) diameter columns, each founded on a 5' (1.52 m) diameter shaft, support the new girders. The bridge's seat type abutments were widened with dimensions that varied from a minimum width and depth equal to the existing abutments to a width of 4' (1.22 m) and a depth of 3'-9" (1.14 m). The widened portions of each abutment are founded on two, 3'-6" (1.07 m) diameter shafts. Two longitudinal seismic restrainers were added at each intermediate pier during this bridge widening.

IV. Mathematical Modeling

The idealized mathematical model of the bridge was created using SAP2000 Version 10.1.3 [4], as shown in Fig. 4. The superstructure is represented by a single line of multiple three-dimensional frame elements (i.e., a spine-type configuration), which passes through the centroid of the superstructure. Each of the columns and the tie beams are represented by three-dimensional frame elements, which pass through the geometric center of the section.

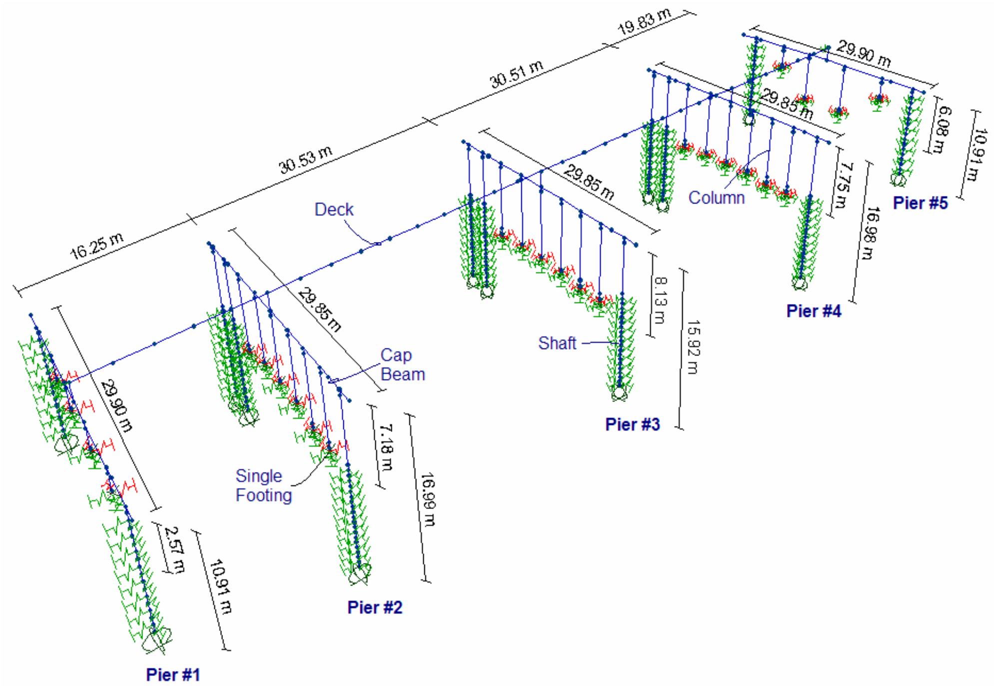

Fig. 4: Bridge Mathematical Model

The mass was specified per unit length of the members with half the member mass being subsequently assigned to each node. The columns of Pier 1 through Pier 7 are circular with a diameter of 3.0 ft (0.91 m). Since the bridge columns are expected to respond inelastically under the input ground motion, effective column properties were used to reflect concrete cracking and reinforcement yielding. The effective flexural stiffness (Ieff) of the bridge columns depends on the axial load ratio (P/f'cAg) and the longitudinal reinforcement ratio (Ast/Ag) where P is the axial load, f'c. is the concrete compressive strength, Ag is the gross area of the section, and Ast is the area of the longitudinal steel [5].

The column spread footings were restrained against translation and rotation with springs provided in each orthogonal direction to account for soil flexibility. The 2008 WSDOT Bridge Design Manual (BDM) [6] recommends the use of spring-only foundations (i.e., no mass or dashpot elements) for spread footings constructed on intermediate and stiff soils. The stiffness of the translational and rotational springs for the column spread footings was determined based on the provisions of article 7.2.6 in the BDM [6]. The stiffness of the translational springs for the column shafts was determined by the software DFSAP Version 1.0 [7].

The support provided by the abutment is assumed to be fixed against translation vertically, and has translational spring in the longitudinal direction. The stiffness of the translational springs was determined in accordance with the BDM [6].

V. Analysis Results

The Bridge under investigation was evaluated under design level seismic event with Method C and Method D2 analyses. The following are the findings of both analysis methods

A. Method C Evaluation

A summary of Method C Capacity/Demand ratios for the Bridge is provided in Table 1. The Capacity/Demand ratios of the original columns and footing for Piers 3 and 4 were found to correspond to Case III as previously described in the Method C analysis approach section. Under Case III, pier columns are more likely to yield before the footings; therefore, the column details in potential plastic hinge regions are checked for their ability to exhibit a ductile response. The column longitudinal reinforcement lap splice detail at the bases of the columns of Piers 3 and 4 were found to be inadequate. Also, the columns transverse confinement reinforcement for Pier 4 was found inadequate. Therefore, based on Method C Evaluation, seismic retrofit to improve the ductility and lap splice detail of Piers 3 and 4 original columns is required.

TABLE I
METHOD C ANALYSIS RESULTS SUMMARY (CAPACITY/DEMAND)*

	Pier 1		Pier 2		Pier 3		Pier 4		Pier 5	
	Top	Bot.	Top	Bot.	Top	Bot.	Top	Bot.	Top	Bot.
Col. Anchorage	NA	NA	1.0 (1.0) (1.0)	0.42 (1.0) (NA)	1.0 (1.0) (1.0)	1.0 (1.0) (NA)	1.0 (1.0) (1.0)	1.0 (1.0) (NA)	NA	NA
Col. Lap Splice	NA	NA	NA (NA) (NA)	0.26 (NA) (NA)	NA (NA) (NA)	0.36 (NA) (NA)	NA (NA) (NA)	0.25 (NA) (NA)	NA	NA
Col. Confinement	NA	NA	1.41 (NA) (4.1)	1.0 (NA) (1.9)	1.24 (NA) (2.1)	1.39 (NA) (1.7)	0.88 (NA) (2.0)	0.94 (NA) (1.5)	NA	NA
Col. Shear	NA		1.27 (7.64) (3.24)		2.11 (6.14) (2.59)		1.65 (4.06) (2.41)		NA	
Footing Rotation	NA		0.88 (NA) (NA)		NA (NA) (NA)		NA (NA) (NA)		NA	
Abut. Deflection	1.27L&0.95T		NA		NA		NA		1.27L&0.95T	
Seat Length	1.48		0.68		0.68		0.68		1.48	
Shear Key	<1		<1		<1		<1		<1	

* C/D's in parentheses are for the widened portion columns and foundations.

The Capacity/Demand ratios of the original columns and foundations for Pier 2 were found to correspond to Case IV. Under Case IV, the columns and foundations of a pier are equally likely to yield; therefore, an investigation of the column ductility and foundation rotational ability is required. The original columns foundation rotational capacity was determined to be inadequate. Also, the columns longitudinal reinforcement lap splice and anchorage details at the bases of the columns of Pier 2 were found to be inadequate. Therefore, based on Method C Evaluation, seismic retrofit of the original columns of Pier 2 is required.

The Capacity/Demand ratios of the columns and foundations in the first widened portions of Piers 2, 3 and 4 were found to correspond to Case I. Under Case I, the columns and foundations of a pier are not likely to yield; therefore, an investigation of the column confinement and foundation rotational ability is not required. The lap splice and anchorage details of the columns longitudinal reinforcement were investigated and found to be adequate. Therefore, seismic retrofit of the columns in the first widened portions of Piers 2, 3 and 4 will not be required.

The Capacity/Demand ratios of the columns and foundations in the second widened portions of Piers 2, 3 and 4 were found to correspond to Case III, which means that the columns are more likely to yield before the footings; therefore, the column details in potential plastic hinge regions are checked for their ability to exhibit a ductile response. The columns longitudinal reinforcement lap splice and anchorage details were found to be adequate. Also, the columns transverse confinement reinforcement was determined to be adequate. Based on Method C Evaluation, seismic retrofit of the columns in the second widened portions of Piers 2, 3 and 4 is not required

B. METHOD D2 EVALUATION

The results of Method D2 or pushover analysis show that the existing columns of the Bridge intermediate piers have adequate displacement capacities to accommodate the anticipated seismic displacement demands. A summary of the Method D2 analysis results is shown in Table 2 and Table 3.

In the transverse direction, the pushover analysis resulted in an inelastic displacement capacity of 3.66 inches (93.0 mm), 3.97 inches (100.8 mm) and 4.03 inches (102.4 mm) for Piers 2, 3, and 4, respectively. All piers reached their ultimate displacement capacity when the top and bottom hinges of one of the original columns reached its maximum plastic rotational capacity. The controlling limit state for the plastic hinge degradation in Pier 2 columns was the compression failure of the unconfined concrete. The controlling limit state for the plastic hinge degradation in Piers 3 and 4 columns was buckling of longitudinal rebar. The transverse displacement demand for Piers 2, 3, and 4 was calculated to be 1.51 inches (38.4 mm), 2.30 inches (58.4 mm) and 3.09 inches (78.5 mm), respectively. This resulted in displacement Capacity/Demand ratios of 2.42, 1.72 and 1.30 for Piers 2, 3, and 4, respectively. Piers 2, 3, and 4 pushover curves in the transverse direction are shown in Fig. 5.

TABLE II
METHOD D-2 ANALYSIS RESULTS SUMMARY –ORIGINAL STRUCTURE (CAPACITY/DEMAND)*

Item	Pier 2	Pier 4	Pier 5
Foundation			
Footing Overturning	0.54	0.80	0.63
Footing Moment	0.32	0.85	0.95
Footing Shear	0.88	0.78	0.84
Column			
Δ Longitudinal	1.79	1.90	1.62
Δ Transverse	2.42	1.72	1.30
Shear	2.73	3.04	2.95
Crossbeam			
Moment	0.51	0.49	0.45
Shear	2.16	1.76	1.83
Girder Stop			
Shear	0.75	0.76	0.43
Diaphragm			
Shear	19.47	19.66	11.06

* C/D based on element forces associated with column plastic hinging.

TABLE III
TABLE 3: METHOD D2 ANALYSIS RESULTS SUMMARY –WIDENED STRUCTURE (CAPACITY/DEMAND)*

Item	Pier 2	Pier 4	Pier 5
Foundation			
Shaft Moment	1.14	1.09	1.06
Shaft Shear	4.02	3.54	4.09
Footing Overturning	0.61	0.77	0.35
Footing Moment	1.69	2.00	1.93
Footing Shear	1.21	1.43	1.37
Column			
Δ Longitudinal	1.79	1.90	1.62
Δ Transverse	2.42	1.72	1.30
Shear	3.49	2.93	2.76
Crossbeam			
Moment	0.29	0.22	0.20
Shear	1.79	1.45	1.53
Girder Stop			
Shear	1.13	1.14	0.64
Diaphragm			
Shear	19.47	19.66	11.06

* C/D based on element forces associated with column plastic hinging.

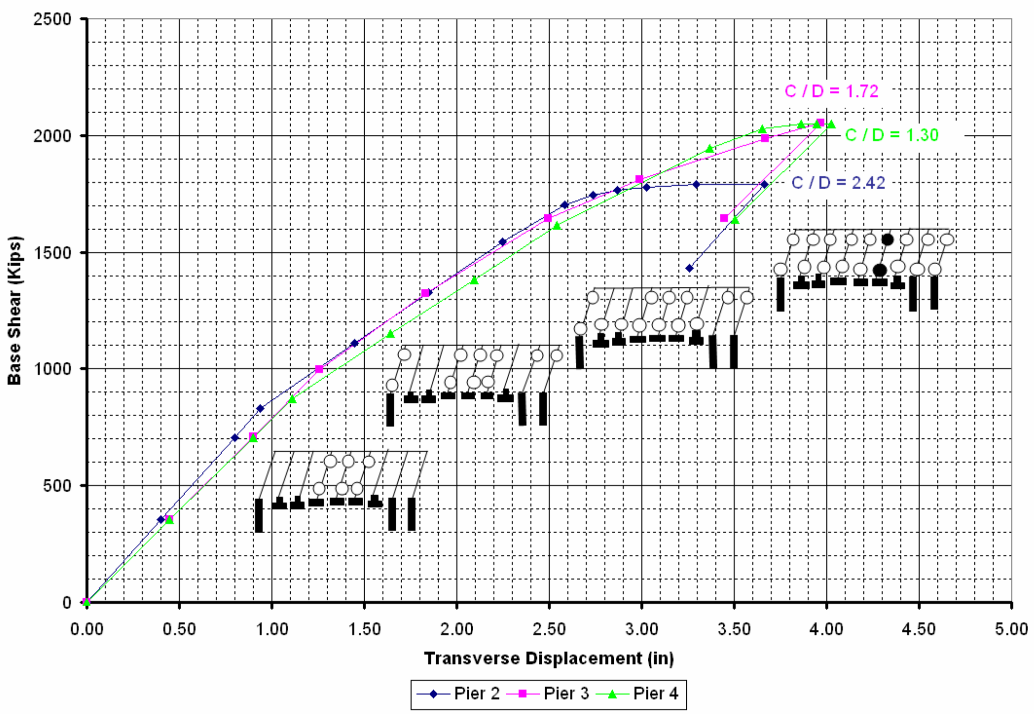

Fig. 5: Pushover Curve from Analysis in the Transverse Direction

In the longitudinal direction, the pushover analysis results show that the inelastic displacement capacities of Piers 2, 3, and 4 are 6.99 inches (177.5 mm), 8.51 inches (216.2 mm) and 8.13 inches (206.5 mm), respectively. The columns behave as cantilever columns in the pier longitudinal direction; therefore, the ultimate displacement capacity of both piers was controlled by the plastic rotational capacity near the base of the columns. The controlling limit state for Piers 2 and 4 was the compression failure of the unconfined concrete at the bases of the columns. The controlling limit state for Pier 3 was buckling of longitudinal rebar. The longitudinal displacement demand for Piers 2, 3, and 4 was calculated to be 3.91 inches (99.3 mm), 4.47 inches (113.5 mm) and 5.03 inches (127.8 mm), respectively. This resulted in displacement Capacity/Demand ratios of 1.79, 1.90, and 1.62 for Piers 2, 3, and 4, respectively. Piers 2, 3, and 4 pushover curves in the longitudinal direction are shown in Fig. 6.

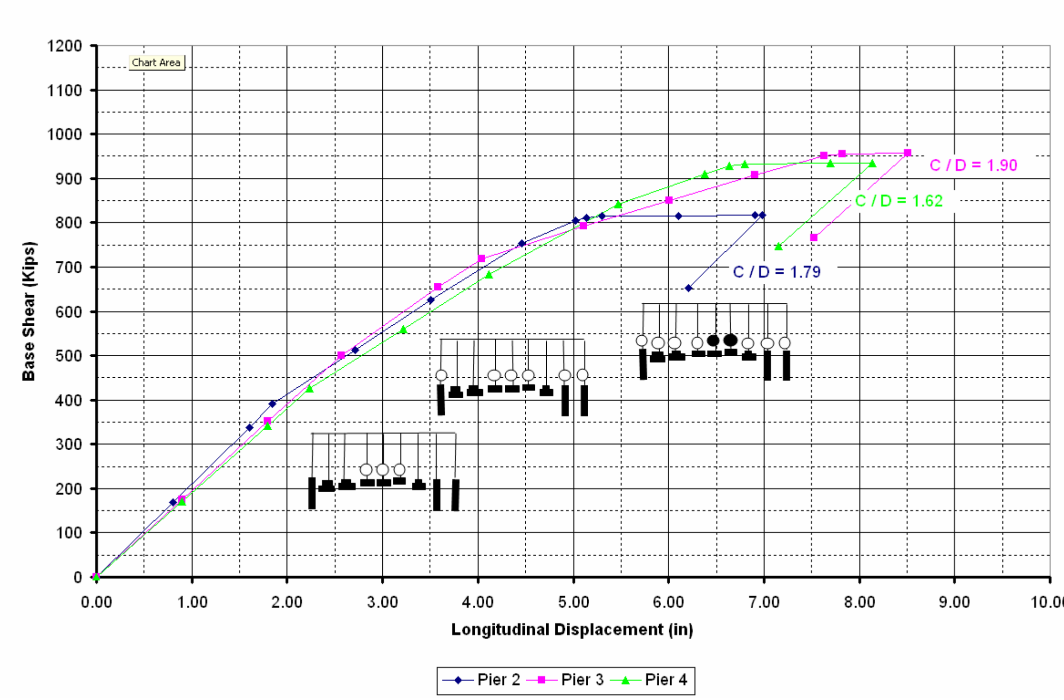

Fig. 6: Pushover Curve from Analysis in the Longitudinal Direction

The crossbeams of Piers 2, 3, and 4 were assumed to remain elastic for the purposes of the pushover analysis. The nominal bending capacity of the original and widened portions crossbeams was found to be inadequate in resisting the anticipated column plastic hinging forces. The bottom layer of reinforcement at the original and first widening overhangs is found inadequate in resisting the anticipated seismic demands when connected to the widened portion. Also, the crossbeam connection to the first and second widening is determined to be inadequate. Therefore, seismic retrofit of the existing crossbeams of the intermediate piers is required.

C. Summary of Analysis Results

Based on Method C, the original columns typically have a deficient longitudinal reinforcement lap splice detail at the base of the columns. Also, some columns have inadequate transverse confinement reinforcement in the plastic hinge regions near the top and bottom of columns. The widened portion columns are found to be satisfactory. Therefore, under Method C, seismic retrofit will only be required for the original columns.

Based on Method D2 analysis, both the original and widened piers are found to have adequate inelastic displacement capacities to accommodate the anticipated seismic displacement demands, and the displacement based Capacity/Demand ratios are determined to be greater than 1.0. Therefore, both the original and widened portion columns will not require retrofitting.

VI. Conclusions

Analysis results showed that the crossbeams at Piers 2, 3 and 4 need to be retrofitted to prevent plastic hinging from occurring within the crossbeams. Also, girder stops need to be provided between all girders at the intermediate piers and abutments. This will ensure the adequate transfer of the seismic forces from the superstructure to the substructure. Furthermore, results showed that Method C and Method D2 analyses lead to different conclusions regarding the adequacy of the columns in the original and widened portions of the bridges.

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