

PUSHOVER ANALYSIS OF THE I-155 BRIDGE NEAR CARUTHERSVILLE, MISSOURI

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Abstract

This paper presents a pushover based seismic evaluation of a 7,100 foot (2,164 m) long bridge located near the New Madrid Seismic Zone in southeastern Missouri. The evaluation includes the existing structure, and the substructure retrofitted with column jackets, cap-beam modifications, and seismic isolation bearings. The evaluation shows that the existing structure has 30% to 40% of the displacement capacity required for the 500 year design level, and significantly less than required for the 1,000 and 2,500 year levels; that retrofits can improve performance to the 500 year level; and that isolation bearings can improve performance of the main spans.

Introduction

This paper presents a seismic evaluation of the I-155 Bridge over the Mississippi River in southeastern Missouri. The objectives of this evaluation are to extend previous studies, and to develop a range of seismic retrofit alternatives for three earthquake design levels. The focus of this paper is the evaluation of the bridge substructure based on Method D2: Structure Capacity/Demand (Pushover) Method of (FHWA 2006). The following paragraphs present a description of the bridge, previous studies, geotechnical investigations and evaluations, structural evaluation, and conclusion.

Description Of The Bridge

The subject bridge is a 59-span structure carrying northbound and southbound Route I-155 across the Mississippi River between Pemiscot County, Missouri and Dyer County, Tennessee. The structure is about six miles southeast of Caruthersville, Missouri and is within the New Madrid Seismic Zone, and is about six miles from an arm of the zone. This bridge was originally designed and constructed in the late 1960's through early 1970's. The substructure units are numbered sequentially from Abutment 1 on the Missouri side to Abutment 60 located on the Tennessee side. The total length of the structure from abutment to abutment is about 7,100 feet (2,164 m), which is comprised of the following 3 sections: Missouri approaches, main spans, and Tennessee approaches.

The Missouri approach spans consist of a seven-span unit with spans of 70 feet (21.3 m) and a six-span unit with spans of 90 feet (27.4 m), for a total length of 1,030 feet (313.9 m). The superstructure is made up of multiple precast, prestressed concrete girders made continuous for live load. The substructure consists of Abutment 1 and three-column framed

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(i.e. multiple columns with continuous cap-beams) Bents 2 to 14. Clear column heights vary from about 30 feet (9.1 m) at the Missouri abutment to 60 feet (18.3 m) nearest the river channel. The approach spans are supported on steel friction piles, driven into the near surface silty sands and clayey materials to a depth of approximately 50 feet (15.2 m).

The main spans comprise about 3,590 feet (1,094 m) of structure consisting of five units. The channel crossing is via a two-span asymmetric cantilever steel truss with a primary span of 920 feet (280.4 m) and an anchor span of 520 feet (158.5 m). The substructures for this unit consist of two-column framed Piers 19, 20, and 21 founded on deep caissons which bear on dense to very dense sand at depths of 50 to 90 feet (15.2 to 27.4 m) below the river bottom. Bedrock is located beneath about 2,700 feet (823 m) of sands, gravels, and hard clay strata. Approaching both sides of the truss spans are five steel girder spans consisting of a four-span continuous two-girder system with spans of about 240 feet (73.2 m) and a simple span multiple girder unit spanning approximately 130 feet (39.6 m). The Missouri side superstructure is supported two-column framed Bents 15 to 18 on steel friction pile foundations. The substructure for the Tennessee side consists of two-column framed Bents 22 to 25 on steel friction piles driven to a depth of about 50 feet (15.2 m).

The Tennessee approach spans consist of four multiple precast, prestressed concrete girder units made continuous for live load, with a total length of about 2,480 feet (756 m): a five span unit of 90 foot (27.4 m) spans, two ten span units of 70 foot (21.3 m) spans, and a nine span unit of 70 foot (21.3 m) spans. The configuration of the Tennessee approach spans is the same as the Missouri approach spans, except the Tennessee approaches are low level structures with typical clear column heights of approximately 20 feet (6.1 m). Steel friction piles support the three-column framed Bents 26 to 59 and Abutment 60, and are terminated in the dense to very dense sandy soils at a depth of about 50 feet (15.2 m).

Previous Studies

In 1993-1994, Jacobs (formerly Sverdrup Civil, Inc.) performed a seismic evaluation of the I-155 Bridge. Capacity/demand (C/D) ratios for the key structural components of the existing bridge were evaluated in accordance with (FHWA 1983). This evaluation demonstrated that many critical structural components are deficient when subjected to the specified earthquake loadings. Typical C/D ratios ranged from 0.25 to 0.60 for bearing displacements, and 0.10 to 0.60 for bearing forces. C/D ratios were typically found to be about 0.30 for columns, 0.20 for reinforcing details, 0.30 for footings, and 0.50 for footing rotations. Evaluation of truss members and details found similar results for the lateral load system and the hinge and wind transfer device. These results indicate that the bridge has significantly less capacity than that required for the 500 year return period design spectra (AASHTO 1992) and a 2,500 year site specific response spectra developed for the project. A retrofit feasibility study developed a series of retrofit components, including: liquefaction mitigation and foundation strengthening, seismic isolation bearing retrofit, seismic restrainers, pier ductility retrofits (column jackets), pier strengthening and stiffening retrofits, foundation modifications, and truss modifications. The study also included

development of a matrix of seismic retrofit strategy alternatives and preliminary cost estimates. At the completion of these tasks, the Missouri Department of Transportation (MoDOT) elected to put further work on the project on hold while focusing attention on other major seismic retrofit projects in the state.

Geotechnical Investigations And Evaluations

This project includes significant geotechnical investigations and evaluations, including site characterization, field exploration and laboratory testing, probabilistic seismic hazard study, development of site specific ground motions and acceleration response spectra (ARS) for earthquakes with 500, 1,000, and 2,500 year return periods (see Figure 1), and development of linear soil/foundation stiffness for use in the structural evaluation. Liquefaction potential was evaluated, and widespread liquefaction to depths of 40 to 65 feet (12.2 to 19.8 m) along the entire bridge alignment is expected for all three seismic design levels. Liquefaction mitigation alternatives were evaluated and the recommended mitigation approach includes a zone of compaction grouting and vertical drains along the entire length of the bridge, as well as edge containment consisting of stone/gravel columns to arrest liquefaction induced lateral displacements of the sloping embankments. Although these investigations and mitigation measures are significant components of the overall evaluation, further discussion of these efforts is beyond the scope of this paper.

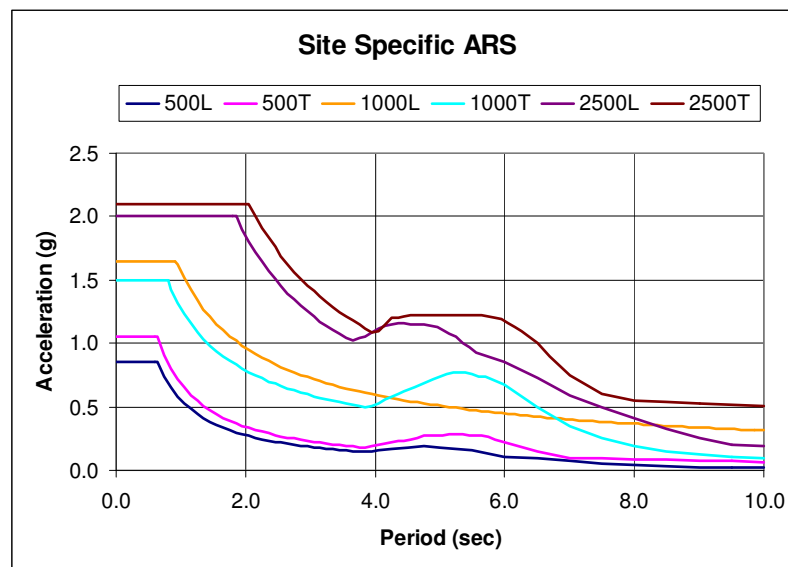


Figure 1. Acceleration Response Spectra

Structural Evaluation - Analytical Model

The analytical models used in this study for dynamic analysis were based on the models developed for the previous 1993-1994 studies. Due to limitations of MSTRUDL, the structure was previously divided into six models, with overlapping spans used to account

for the portions of the structure that were not included in each analysis. For the current study, the six models were combined into a single model of the entire structure and an in-house translator was used to convert the input files from MSTRUDL into SAP2000 format. The combined model contains the entire bridge and includes 3,008 joints, 3,754 frame elements, and 14,472 degrees of freedom.

(FHWA 2006) focuses detailed seismic evaluation of bridges on quantifying the seismic behavior of the substructure, as acted on by ground motion and the dynamic effects of the superstructure. Consequently, the analytical model is designed to include direct representation of the following significant substructure elements: bearings, cap-beams, columns, footings, monolithic walls and caissons, seal courses, and pile/soil stiffness. Three-dimensional Frame elements, located along the center of gravity, are used to model each of the substructure components. All concrete components are assumed to be rigidly connected, while member releases are used to account for the longitudinal and transverse restraint provided by each type of bearing. Geometric properties of the various concrete components are based on gross sections taken from the dimensions on record drawings, except for the columns and cap-beams that are expected to be subject to cracking and plastic hinging, and are consequently assigned moments of inertia equal to 50% of the gross section values, Section 7.3.2.1 of (FHWA 2006). The eccentricity between the center of the cap-beams and the bottom of the bearings is modeled with “rigid” Frame elements. The soil-foundation system is modeled with linear springs, based on site specific geotechnical data and the assumption that liquefaction mitigation measures are implemented.

The analytical model of the concrete girder and steel girder superstructure spans is based on three-dimensional Frame elements located along the center of gravity of the superstructure. Superstructure properties are based on gross transformed sections taken from the dimensions shown on record drawings. The superstructure elements are connected to the tops of the bearings at each pier with rigid body constraints.

The analytical model of the truss spans is designed to include representation of each of the individual truss members and the effects of the internal hinge and wind transfer device at panel L20-U20. Three-dimensional Frame elements are used to represent the geometric and elastic effects of the various components, and all connections were assumed to be rigid, with the exception of the hinges noted above. The slab/roadway is also modeled with three-dimensional Frame elements and member releases were used to account for the longitudinal restraint conditions at each of the floor beams. Frame elements are modeled at the center of gravity of the various components, and rigid beam elements with no mass were used to connect the center of gravity of the slab/roadway to the floor beams. Truss member properties are based on the gross sections shown on the record drawings.

Dynamic Analysis of the Existing and Retrofitted Structure

Dynamic analysis was performed on the analytical model using SAP2000 Advanced Version 11.0.0 and the multi-mode response spectra analysis method. Method D2, Section

5.6 of (FHWA 2006) uses this linear-elastic analysis method to define the seismic displacement demands for complex structures, such as the subject bridge. We extracted the first 400 modes from the model based on eigenvalues to achieve mass participation in the analysis of at least 90%. Periods of these modes range from 2.61 to 0.12 seconds and the total mass participation is 94% in the X (bridge longitudinal), 91% in the Y (bridge transverse), and 76% in the Z (bridge vertical) directions. Displaced shapes of the modes were reviewed and it was seen that the first 400 modes include longitudinal and transverse vibration of all of the major structural units along the bridge alignment.

Multi-mode response spectra analysis was performed in the longitudinal and transverse directions using the ARS shown in Figure 1 for the 500, 1,000 and 2,500 year design levels. Results from the first 400 modes were combined by the CQC method and the longitudinal and transverse analyses were combined using 40% combinations, Section 7.4.2 of (FHWA 2006). Displacement and force demands from these analyses were used for evaluation of the structure in the existing configuration, as well as for the structure when retrofitted with ductility enhancing modifications such as column jackets or cap-beam modifications, because the change in stiffness caused by these retrofits is expected to be on the order of 10% to 15%, Section 9.2.1.3 of (FHWA 2006), which is judged to be within the uncertainty of other parameters used in the study.

Dynamic Analysis of the Structure with High Capacity Isolation Bearings

The 1993-1994 seismic evaluation considered the alternative of replacing the existing fixed and expansion bearings that support the steel approach and truss spans with seismic isolation bearings. These bearings are designed to decrease the demands on the substructure elements by lengthening the structural period, and thereby shifting the response to a lower value on the ARS curve, and by increasing damping, which also reduces structural response. Preliminary designs for lead-rubber isolation bearings were considered in the previous study; however, lead-rubber bearings of the size necessary to support the truss were not available at the time.

Since the previous evaluation, high capacity isolation bearings have been developed, tested, and installed on large scale bridge projects in California and Tennessee. As part of the current study, we contacted Earthquake Protection Systems Inc. (EPS), who recommended Triple Friction-Pendulum bearings for this structure. These bearings are 12 to 13 feet (3.7 to 4.0 m) in diameter, 32 inches (81.3 cm) tall, and have a displacement capacity of +/- 100 inches (2.54 m). Preliminary analyses by EPS indicate bearing displacements of 14, 40, and 98 inches (35.6, 101.6, and 248.9 cm) for the 500, 1,000, and 2,500 year return period earthquakes, respectively.

Seismic isolation bearings were not considered for the concrete girder approach spans because the height of the existing fixed bearings is 1 inch (2.5 cm) and the expansion bearing height is 5 inches (12.7 cm), which are not sufficient for isolation bearing retrofit

without raising the existing roadway profile. In addition, the diaphragms at the fixed bearings are restrained by cast-in-place concrete shear keys.

We performed preliminary analysis of isolation bearings in the steel approach and truss spans for this study using the multi-mode response spectra analysis method with modifications to the bearing stiffness and the ARS curves. We used effective isolation bearing stiffness of 20 and 60 kips per inch (35.0 and 105.1 kN/cm) in the steel approach and truss spans, respectively, based on manufacturer information. We also modified the ARS curves to account for 20% damping for periods greater than 2.2 seconds using $B=1.5$ and the procedures of Section 7.3 of (AASHTO 1999). The damping value of 20% was selected as a value that should be attainable with isolation bearings from several manufacturers, rather than a specific design. The results from these analyses were used to define the substructure displacement demands on the isolation bearing retrofitted structure for Method D2, Section 5.6 of (FHWA 2006).

Pushover Analysis of the Existing Structure

In Method D2, Section 5.6 of (FHWA 2006), the displacement capacity of the substructure units is determined by pushover analysis of the individual bents and piers. This procedure differs from the Capacity/Demand (C/D) ratio method (FHWA 1983) used in the 1993-1994 evaluation because the C/D ratio method involves comparing component plastic hinging capacity to component moment demand determined from multi-mode response spectra analysis, while the pushover procedures involve comparing non-linear substructure displacement capacity to displacement demands determined from multi-mode response spectra analysis. The displacement based procedure is therefore consistent with the “equal displacement” assumption on which seismic design using linear analysis is based.

We used an in-house program called PDPUSH to perform the pushover analyses for this evaluation. This pushover analysis is based on the “Linear Incremental” method of non-linear analysis described in (Chajes and Churchill 1987). The element stiffness matrix in this analysis procedure includes both linear stiffness terms and initial stress stiffness terms which account for $P-\Delta$ effects, as required by (FHWA 2006). Because changes in stress and deformation that take place during the application of a load step are neglected in this method, it is necessary to apply the load in relatively small increments. This limitation is well suited to pushover analysis in which intermediate results at relatively small increments of load are of interest. For purposes of this study, initial loads such as dead load were applied in 100 equal steps, and subsequent pushover loads were applied in 1 kip (4.4 kN) steps. PDPUSH was set to report results every 10 steps to limit output file size.

Non-linear material models are used for confined and un-confined concrete based on (Mander et al. 1988). The strain at maximum unconfined concrete strength, ϵ_{co} is taken as 0.002, and the ultimate unconfined concrete spalling strain, ϵ_{sp} is taken as 0.005, Section 3.2.6 (Caltrans 2006). The ultimate confined concrete strain, ϵ_{cu} is based on Equation B-20 of (FHWA 1995), rather than Equation 7-36 of (FHWA 2006). Equation B-20 was selected

because it is consistent with Equation 5.14 of (Priestley et al. 1996), and the justification of changing the term from 0.004 to 0.005 was not apparent from reading (FHWA 2006). Also, the expected concrete strength, f'_{ce} is taken as the expected value of 1.5 times the specified minimum design value, f'_c , Section B.2.1 (FHWA 1995).

Non-linear material was also used for reinforcing steel based on Mander's model. The equations and parameters used in this model were taken from (Esmaeily-Gh. and Xiao 2002). The strain at onset of strain hardening, ϵ_{sh} is taken as 0.02; the strain at peak stress, ϵ_{su} is taken as 0.148; yield stress, f_y is taken as 44.0 ksi (303 MPa); and peak stress, f_{su} is taken as 68.0 ksi (469 MPa) based on Grade 40 reinforcing steel and Figure 5.5 of (Priestley, et al. 1996). The effective peak stress, f_{sr} is taken as 66.0 ksi (455 MPa) and the effective peak strain, ϵ_{sur} is taken as 0.123, Section 3.2.2 (Caltrans 2006).

At the end of each incremental load step, the curvature of each member, Φ , is calculated from $\Phi=M/EI$, where M is the member end moment computed in the analysis load step, E is the material elastic modulus, and I is the moment of inertia used in the analysis load step. A moment-curvature section analysis using the relationship, $\Phi=\epsilon_c/C$, is then performed using the member axial load and moment computed in the analysis load step and the non-linear material models to find the maximum concrete strain, ϵ_c , depth to the neutral axis, C , and the effective moment of inertia, $I_{eff} = MC/E\epsilon_c$. If I_{eff} from the section analysis is not within 10% of I used in the analysis, the stiffness matrix is updated and the analysis load step is repeated. At the end of each load step, the incremental results are added to the cumulative totals to compute the displacement, element internal force, and applied load matrices. The geometry and section properties based on I_{eff} are updated prior to beginning each subsequent load step, (Chajes and Churchill 1987, 1989).

PDPUSH is a two-dimensional analysis, so transverse and longitudinal analyses are conducted with separate analytical models with the various components modeled along the center of gravity of the columns and cap-beams. The models include closely spaced elements in the regions at the top and bottom of the columns, as well as at locations of splices in the longitudinal reinforcing steel within the columns and near the ends of the cap-beams and foundation tie beams to capture the development and progression of inelastic behavior during the pushover analysis. Elastic elements are also included in the model to represent the dimensions of the footings and the column to beam joints. Properties of these elastic elements are based on gross sections and are not revised at each load step. The analytical models also include the linear spring stiffness of the soil/pile system. Incremental pushover lateral loads are applied at the bearing locations along the cap-beams and include a force couple of vertical loads to represent the eccentricity between the center of gravity of the superstructure and the center of the cap-beam.

The following limit states are tracked for each element within PDPUSH to capture important changes in behavior: (0) elastic, (1) first steel yield at ϵ_{yield} , (2) maximum unconfined concrete strength at ϵ_{co} , (3) maximum confined concrete strength at ϵ_{cc} , (4) unconfined concrete spalling at ϵ_{sp} , (5) ultimate concrete compressive strain at ϵ_{cu} , (6) poorly

confined lap splice failure, (7) bar anchorage failure, and (8) member compression or tension failure. The relationships for lap splice failure and bar anchorage failure, Limit States 6 and 7, respectively, are based on Section B2.2 of (FHWA 1995). Limit State 8 represents a condition in which internal forces developed in the section analysis can not balance the externally applied axial load and moment. Limit State 8 typically is observed at large curvatures and strains in which unconfined concrete has failed, possibly some of the reinforcing bars have failed, and/or the strain in significant portions of the confined core has exceeded ϵ_{cu} . Physically this limit state represents member collapse.

Section 5.6.2 of (FHWA 2006) limits displacement capacity to the first component to reach maximum deformation capacity, which is characterized as Limit State 5 or 8 in PDPUSH. To continue the pushover analysis beyond this limit, whenever a member reaches Limit State 8, I_{eff} is set equal to $I_{gross}/1000$ and the member is omitted from section analysis in subsequent load steps. For this study, the pushover analyses are continued to a maximum lateral displacement of 10 inches (25.4 cm), or instability, whichever occurs first, which allows insight into the progression of hinging and failure of the bents.

Transverse pushover analysis of representative three-column Bent 2 with no retrofits produces the following progression of events:

- Limit State 1, yielding in the reinforcing steel begins at the top of the center column at a displacement of 1.36 inches (3.45 cm) and an initial effective stiffness of 257 kips per inch (450.1 kN/cm).
- Limit State 2, cracking of the unconfined concrete at $\epsilon_{co}=0.002$, begins at the top of the center column at a displacement of 2.06 inches (5.23 cm) as yielding of the reinforcing steel progresses down the center column and begins developing at the top of the other columns.
- Limit State 4, spalling of unconfined concrete at $\epsilon_{sp}=0.005$, begins at the top of the center column at a displacement of 2.23 inches (5.66 cm). Because the confining hoops terminate with lap splices in unconfined concrete, the onset of spalling in the existing columns will quickly progress to loss of confinement followed by buckling of the longitudinal bars and failure of the column. At this point, yielding of the reinforcing steel is also beginning at the bottom of the center and left columns. Because the poorly confined lap splices at the base of the columns are of sufficient length, the splices are not expected to slip until compressive strain in the concrete reaches 0.002, Section B.2.2 of (FHWA 1995), or spalling begins in the unconfined cover concrete.
- Limit State 8, which means that the section can no longer balance the externally applied loads, begins at the top of the center column at a displacement of 2.30 inches (5.84 cm). Based on Section 5.6.2 of (FHWA 2006), this represents the ultimate displacement capacity of the existing bent, and the displacement ductility, $\mu\Delta = \Delta u/\Delta y = 2.30/1.36 = 1.69$. For reference purposes, the effective stiffness of the bent in this condition is 230 kips per inch (402.8 kN/cm).
- Limit State 8 is reached at the tops of all three columns and at the bottom of the center column at a displacement of 3.68 inches (9.35 cm). The reinforcing steel in the cap-

beam is also beginning to yield at this point. The effective stiffness of the bent in this condition is 179 kips per inch (313.5 kN/cm).

- Limit State 8 is reached at the tops and bottoms of all three columns at a displacement of 4.38 inches (11.13 cm). The effective stiffness of the bent in this condition is 153 kips per inch (267.9 kN/cm). If the damage areas are contained sufficiently to prevent collapse, this condition represents fully developed plastic hinges at the top and bottom of all three columns, which is the design condition assumed for new structures, Section 7.7.2(B) of (AASHTO 1992).
- The pushover analysis stops at a displacement of 10.50 inches (26.67 cm) with plastic hinges fully developed at the top and bottom of all columns and at the ends of the foundation tie beams. The effective stiffness of the bent in this condition is 65 kips per inch (113.8 kN/cm).

PDPUSH was run for Bents 2 through 59 to establish the displacement capacity and the displacement ductility of the existing substructure units in both the transverse and longitudinal directions. These analyses show that the transverse yield displacement of the existing structure ranges from 0.63 to 3.49 inches (1.60 to 8.86 cm); the displacement capacity ranges from 1.15 to 6.08 inches (2.92 to 15.44 cm), and the displacement ductility ranges from 1.48 to 2.74. In the longitudinal direction, yield displacement of the existing structure ranges from 1.03 to 7.03 inches (2.62 to 17.86 cm); the displacement capacity ranges from 2.02 to 15.86 inches (5.13 to 40.28 cm), and the displacement ductility ranges from 1.63 to 5.42.

In addition to the limit states tracked by PDPUSH, Table 7-3 of (FHWA 2006) also identifies 8 limit states to be evaluated during a pushover analysis. Some of these limit states, such as compression failure of unconfined and confined concrete are tracked directly in PDPUSH, while others are not. The material models in PDPUSH directly monitor lap splice and anchorage failure and fracture of longitudinal reinforcement, and appropriate adjustments in stiffness are accounted for in the section analysis, but these occurrences are not directly reported. The decision as to which of these limit states are tracked in PDPUSH and which require additional processing was based on selecting those that represented changes in material behavior and thus stiffness, as opposed to those that reflect member or joint behavior that likely indicate collapse or otherwise unacceptable conditions and therefore stop the analysis. An in-house post-processor called LSTATE was developed to evaluate each of the limit states listed in Table 7-3. LSTATE reads the detailed output from PDPUSH and a supplemental data file, and computes the status of each limit state for each member for each load step. For organizational purposes, Limit State 0 is defined as elastic behavior and Limit States 1 through 8 are assigned in the order listed in Table 7-3.

While Section 7.7.1.1 of (FHWA 2006) allows calculation of curvature, strains, and neutral axis location by detailed section analysis, the manual presents the various limit states and simplified relationships for approximating these values under the assumption that more detailed results are not available. Because the necessary detailed results are captured in the PDPUSH output files, the limit states evaluated in LSTATE are expressed in terms of the

relevant strain or curvature parameters, rather than in terms of curvatures based on the simplified relationships presented in (FHWA 2006).

In evaluating the limit states of Table 7-3 of (FHWA 2006), it was observed that the low-cycle fatigue limit state would sometimes indicate failure at strains just into the strain-hardening portion of the curve with only a few effective equal amplitude cycles. This result was discussed with the author of these provisions, and it was found that the limiting plastic strain amplitude, ϵ_{ap} , computed in Equation (7-41) of (FHWA 2006) should be compared to the average plastic strain range in the longitudinal steel, not the maximum tensile strain, in recognition of the fact that in these members that compressive strain is usually much less than the tensile strain. The desired average plastic strain value is obtained from the PDPUSH total strain results by the expression $\epsilon_{apavg} = (\epsilon_s + \epsilon_s' - 2\epsilon_y)/2$, where ϵ_s is the maximum steel tension strain, ϵ_s' is the maximum steel compression strain, $\epsilon_y = f_y/E$ is the yield strain, f_y is the expected yield stress, and E is the reinforcing steel elastic modulus.

LSTATE was run for Bents 2 through 59 to establish the displacement capacity, based on Table 7-3 of (FHWA 2006) limit states. These results show that the minimum transverse displacement capacity of the existing bents ranges from 0.69 to 5.05 inches (1.75 to 12.83 cm) and the minimum longitudinal displacement capacity ranges from 1.93 to 15.86 inches (4.90 to 40.28 cm). Review of the limit states shows that bent displacement capacities were controlled by Limit State 1 – compression failure of unconfined concrete, 5 – low cycle fatigue, 6 – lap-splice failure, and 7 – shear failure.

Pushover Analysis of the Structure with Retrofits

Section 9.2.1.3 of (FHWA 2006) presents the information necessary to modify the expressions for ϵ_{cu} and f'_{cc} in the non-linear concrete models to account for steel jackets, fiberglass/epoxy jackets, and prestressed wire wraps. Because these retrofits are installed on the outside of existing columns, the section analysis is also modified to account for elimination of unconfined concrete in the jacketed sections. While certainly viable, fiberglass/epoxy jackets are not evaluated in this study because they are not commonly used in this region. PDPUSH includes the capability to analyze sections with steel jackets and prestressed wire wraps. In evaluating the effectiveness of these retrofits, it is observed that the maximum confining pressure that can be applied with prestressed wire wraps is limited by the physical spacing of the wires and the sizes of couplers that are commercially available. The thickness of the steel jackets is also limited by fabrication capacity to 1 inch (25.4 mm), based Section 9.2.1.3(a) of (FHWA 2006). Given these limitations, higher confining pressures can be developed with steel jackets than prestressed wire wraps, so the study effort is focused on steel jacket retrofits.

In addition to enhancing column ductility, it is also sometimes necessary to consider modifications to the cap-beams to force inelastic behavior into the columns, as preferred in (FHWA 2006, 1995). Two types of cap-beam modifications are incorporated in PDPUSH: the first is external post-tensioning, and the second is additional conventional reinforcement.

The effect of external post tensioning is accomplished by applying compressive forces to the joints at the ends of the cap-beam model. These loads are stepped on to the beam after the initial dead loads, and before the pushover loads. In the case of additional conventional reinforcing steel, the reinforcing steel strain is captured at the end of the initial dead load steps and subtracted from the strain in the additional bars during pushover analysis. Both of these retrofits give the cap-beam additional moment capacity, which in some cases is sufficient to force inelastic behavior into the columns.

Pushover analysis was performed on the bents and piers of the structure with three thicknesses of steel jackets applied to the top, bottom, and intermediate lap-splice regions of the columns. The first jacket thickness represents a minimum jacket with thickness of the greater of either 0.375 inches (9.5 mm) or the minimum thickness required to prevent lap-splice failure based on the equations in Section 9.2.1.3 of (FHWA 2006). The second jacket thickness represents a medium value of 0.125 inch (3.2 mm) thicker than the minimum, and the third jacket thickness represents the maximum thickness of 1 inch (25.4 mm).

Review of the data from the pushover analyses with jackets shows in some cases, improving column ductility results in inelastic behavior in the beams before the columns have reached limiting behavior. To limit inelastic behavior in the cap-beams, pushover analysis was also performed on the bents with jackets and cap-beam post-tensioning.

Review of the displacement capacity data from these analyses shows that the limit states from Table 7-3 of (FHWA 2006) govern over the material based limits implemented in PDPUSH. This is particularly evident as retrofits are implemented that increase confinement and the controlling failure mode shifts from concrete failure to low cycle fatigue of the reinforcing steel. For example, with minimum jackets, Bent 2 has a transverse displacement capacity of 3.97 inches (10.08 cm) from PDPUSH but is limited to 2.59 inches (6.58 cm) by LSTATE Limit State 5 – low-cycle fatigue of the longitudinal reinforcing steel. It can also be seen from this data that increasing jacket thickness beyond some optimum level can actually decrease displacement capacity. For example, Bent 2 with 1 inch (25.4 mm) thick jackets which has a transverse displacement capacity of 3.24 inches (8.23 cm) in PDPUSH and 2.11 inches (5.36 cm) from LSTATE Limit State 5, which is less than the values discussed above. This behavior has also been observed in laboratory tests and is discussed in Section 7.8.2.5 of (FHWA 2006).

Substructure Displacement Capacity versus Demand

Figure 2 shows the PDPUSH transverse displacement capacity of the existing (Original) and retrofitted (Jacket, Jacket2, Jacket3, and Jacket-PT) substructure units (Piers 2 to 59) and the displacement demands for the three earthquake design levels. This graph shows that the existing bents and piers typically have about 2 inches (5 cm) of displacement capacity and the demands from the 500, 1000, and 2,500 year design levels are typically greater than about 4, 7, and 9 inches (10, 18, and 23 cm) respectively. This results in displacement C/D ratios for the existing substructure units that are typically less than 50%,

29%, and 22% for the three design levels, respectively. The chart also shows that with retrofits of the columns and cap-beams, the displacement capacity can typically be increased to about 3 inches (8 cm). Similar comparison with the displacement capacities from LSTATE shows that the limit states from Table 7-3 of (FHWA 2006) often produce less displacement capacity than the PDPUSH limit states, particularly with retrofits, and consequently produce somewhat lower C/D ratios. Comparison of the displacement capacity versus demand in the longitudinal direction produces similar results.

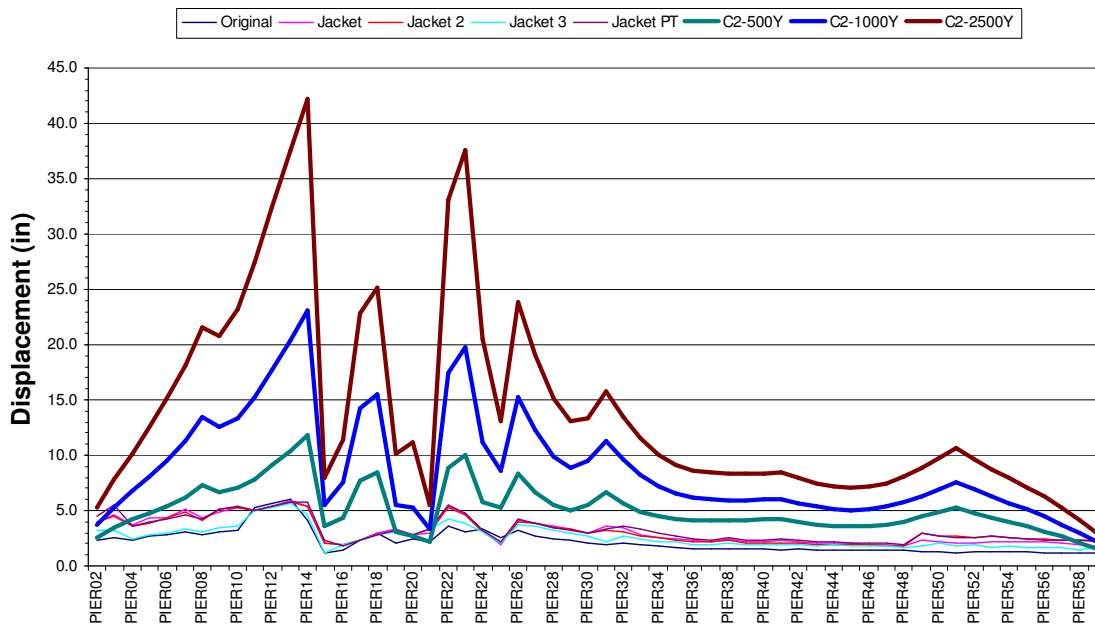


Figure 2. Transverse Capacity and Demand

The capacities used in this comparison are limited by the first component reaching maximum capacity, based on Section 5.6.2 of (FHWA 2006). It is observed that this definition of displacement capacity is inherently different from the plastic hinging analysis of the C/D Ratio method of (FHWA 1983, 1995), which is based on formation of plastic hinges at the top and bottom of all columns in the plane of a multi-column bent and at the bottom of all of the columns in the direction perpendicular to a multi-column bent, using the procedures of Section 7.7.2(B) of (AASHTO 1992). In light of the poorly confined columns and vulnerable details in the existing structure, it is appropriate to limit displacement capacity to the first component reaching capacity because the ability to support vertical loads will likely degrade rapidly beyond this limit; however, with confining retrofits, it is argued that consideration of multiple well contained damage points should be allowed, as provided by Section 3.3.3 of (FHWA 1995) and (AASHTO 1992). In making the decision to consider displacements beyond the first component to reach maximum capacity, it must be recognized that damage to the structure in the form of crushed concrete and fractured reinforcing steel will occur and is acceptable, provided that the hinging sections are sufficiently contained so that the crushed concrete can still carry gravity loads, Section 9.1.2.5 of (FHWA 2006), and that the structure retains sufficient shear capacity to prevent

collapse, Section 3.3.3 of (FHWA 1995). It was found that the substructure units of this bridge with column jackets and cap-beam retrofits can likely resist the 500 year design level with displacement ductility's of about 4 if multiple components are allowed to reach maximum capacity.

Capacity versus Demand of Other Components

The seismic evaluation also included comparison of the footing/foundation system capacity to the demands of column hinging, and the force and displacement capacity of the bearings and the force capacity of the truss members and connections to the demands from the dynamic analysis. These comparisons showed that many of the foundations, bearings, and members and connections in the truss lateral load system lack sufficient capacity to resist the demands from the three design earthquake levels. Comparison of the capacities of these components to the demands from analysis with seismic isolation bearings on Piers 15 to 25 showed that isolation bearing retrofit can significantly reduce demands on the truss members and connections and the piers with isolation bearings, but that additional modifications to the substructure and foundations will still be required. Further discussion of these important results is beyond the scope of this paper.

Conclusion

This paper presents a summary of the seismic evaluation of the I-155 Bridge near Caruthersville, Missouri. Based on the findings of this evaluation, the existing bridge has significantly less capacity than required to resist the 500, 1,000, or 2,500 year seismic design levels. In spite of advances in understanding of the site specific ground motions and application of the Method D2 (FHWA 2006) structure capacity / demand method, this is the same conclusion reached by the 1993-1994 study based on the C/D ratio method of (FHWA 1983), which is the same as Method C component capacity / demand evaluation of (FHWA 2006). In addition to displacement capacity deficiencies related to the columns and cap-beams discussed in this paper, the seismic deficiencies include soil liquefaction and associated ground failures, footings and piles, bearings, seat lengths, load path in the two girder systems, and the lateral load system of truss members and connections.

Based on pushover analysis of the retrofitted bents, column jackets and cap-beam retrofits do not provide sufficient displacement ductility to resist the 500, 1,000, or 2,500 year seismic design levels, as limited by the first component to reach capacity. In addition, foundation modifications are required in the pile supported bents and piers to resist full column hinging demands. The bearing pressures beneath the caissons also exceed acceptable levels at full column hinging demands.

Based on the pushover analysis, the bents with column jackets and cap-beam modifications can likely provide sufficient displacement capacity to resist the 500 year design level with displacement ductility's of about 4 if multiple components are allowed to reach maximum capacity; however, the formation of multiple plastic hinges will require

containment of severe crushing of the concrete and failure of the reinforcing bars. In addition, foundation modifications and cap-beam modifications are required to prevent unacceptable failures in these components at large column demands.

Based on preliminary evaluation of a seismic isolation system for Piers 15 through 25, this portion of the structure is likely to resist the 500 year and possibly the 1,000 year design level, with isolation retrofit in addition to structural and foundation modifications.

Acknowledgement

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References

- AASHTO 1992, "Standard Specifications for Highway Bridges", 15th Edition, American Association of State Highway and Transportation Officials, Division 1A
- AASHTO 1999, "Guide Specifications for Seismic Isolation Design", American Association of State Highway and Transportation Officials, Section 7.3, pp. 32
- Caltrans, 2006, "Seismic Design Criteria Version 1.4, June 2006, pp. 3-5 to 3-9
- Chajes, A. and Churchill, J. E. 1987, "Nonlinear Frame Analysis by Finite Element Methods", *Journal of Structural Engineering*, ASCE, Vol. 113, No. 6, June 1987, pp. 1221-1235
- Chajes, A. and Churchill, J. E. 1989, "Closure", *Journal of Structural Engineering*, ASCE, Vol. 115, No. 2, February 1989, pp. 499-500
- Esmaily-Gh., A. and Xiao, Y. 2002, "Seismic Behavior of Bridge Columns Subjected to Various Loading Patterns, PEER 2002/15, December 2002, pp. 80-81
- FHWA 1983, "Seismic Retrofitting Guidelines for Highway Bridges, Report No. FHWA/RD-83/007, December 1983
- FHWA 1995, "Seismic Retrofitting Manual for Highway Bridges, Report No. FHWA-RD-95-052., May 1995, pp. 48-57, 201, 214
- FHWA 2006, "Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges", Report No. FHWA-HRT-06-032, January 2006, pp. 175-177, 226-227, 230, 249, 356-357, 379-380
- Mander, J. B., Priestley, M. J. N., and Park, R. 1988, "Theoretical Stress-Strain Model for Confined Concrete", *Journal of Structural Engineering*, ASCE, Vol. 114, No. 8, August 1988, pp. 1804-1826
- Priestley, M. J. N., Seible, F., and Calvi, G. M. 1996 "Seismic Design and Retrofit of Bridges", John Wiley & Sons Inc., 1996, pp. 272-273

SEIMIC REINFORCEMENT OF 1ST IBI VIADUCT

Sunao Kawabata¹

Abstract

The 1st Ibi viaduct is a three-span continuous bridge with two abutments and two piers. The piers of the bridge are designed flexible to follow the superstructure's displacement caused by the temperature change and by others.

The bridge was designed according to the old seismic design specifications. After the evaluation by the latest specifications, it was found that the seismic performance of the bridge was not enough. Dissipation measures using viscous dampers were applied to reducing the seismic force at large-scale earthquakes. This reinforcement is expected not to make unfavorable influence on the bridge except when large-scale earthquakes occur.

1. Introduction

The 1st Ibi viaduct is in the Kobe-Awaji-Naruto Expressway, the most eastern route of three Honshu-Shikoku Expressways connecting Shikoku Island with Honshu Island, as shown in Fig. 1. The Akashi Kaikyo Bridge and the Ohnaruto Bridge are in the route. The 1st Ibi Viaduct began to service in 1985.



Fig. 1 LOCATION OF 1ST IBI VIADUCT

The 1st Ibi viaduct was designed according to the 1980's seismic design specifications for highway bridges. The design specifications, however, was revised in 1996 to require significantly higher performance against large-scale earthquakes due to 1995 Hyogo-ken Nanbu Earthquake. Also, according to the latest studies, it was found that major earthquakes will occur in the near future nearby the bridge. Therefore, we started to review its seismic performance, and then found that the bridge did not have enough performance to satisfy the latest specifications almost same as that of 1996. In addition, the expressway which includes the bridge has been decided as the emergency transportation route at the time of disaster. Under such a situation, we decided to reinforce the 1st Ibi viaduct in order to get enough performance against large-scale earthquakes.

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2. Outline of the 1st Ibi Viaduct

As shown in Fig. 2, the 1st Ibi viaduct is a three-span continuous bridge, and its length is 146m. Its superstructure is composed of six I-section steel plate girders and a reinforced concrete slab. There are two abutments (1A and 4A) at both ends of the bridge, and there are two flexible reinforced concrete piers (2P and 3P) between both abutments. The superstructure is supported at 1A, 2P and 3P against the longitudinal direction movement, and supported at all piers and abutments against the transverse direction movement. Since the fixed bearings are used at 2P and 3P, these piers have to follow the superstructure's displacement caused by temperature change, by live load and by others.

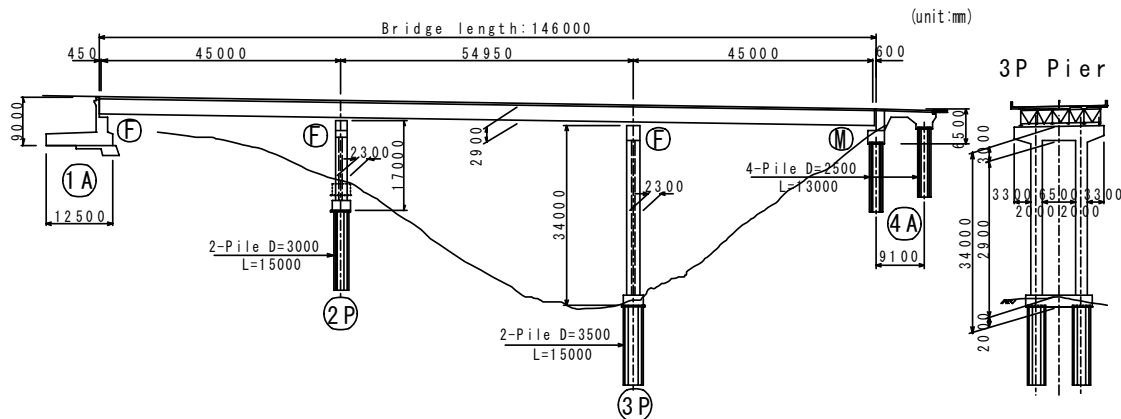


Fig. 2 ELEVATION OF 1ST IBI VIADUCT

3. Seismic vulnerability evaluation of existing bridge

3.1. Seismic performance and design earthquake motion

In the latest seismic design specifications for highway bridges, two levels of design earthquake ground motions have to be considered. The first level corresponds to a middle-scale earthquake (Level 1 earthquake ground motion). And second level corresponds to large-scale earthquakes (Level 2 earthquake ground motion). There are two types earthquake ground motion in the Level 2, one is earthquake ground motion generated by a Plate boundary type earthquake with a large magnitude (Type 1), and the other is earthquake ground motion generated by an Inland active fault type earthquake (Type 2). All the design earthquake ground motions are shown in Fig. 3. Any highway bridges have to have the

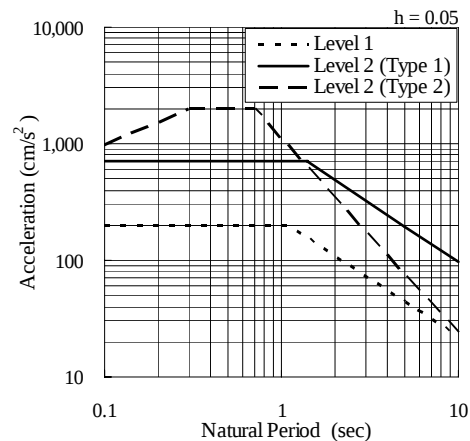


Fig. 3 DESIGN EARTHQUAKE GROUND MOTIONS

seismic performance shown in Table 1.

The 1st Ibi viaduct was designed not to exceed its elastic limit state during only the Level 1 earthquake ground motion in its original seismic design. It was not clear that the bridge had the seismic performance for the Level 2 earthquake ground motion. Therefore, we had to verify whether the bridge had enough seismic performance or not.

Table 1 SEISMIC PERFORMANCE

Earthquake Ground Motions		Importance of Bridges	
		Common	High
Level 1 (highly probable during the bridge life)		Keeping sound functions of bridges (Seismic Performance Level 1)	
Level 2	Type 1 (a Plate boundary type earthquake with a large magnitude)	No critical damages (Seismic Performance Level 3)	Limited seismic damages and capable of recovering bridge functions within a short period (Seismic Performance Level 2)
	Type 2 (an Inland active fault type earthquake)		

3.2. Seismic analysis for large-scale earthquakes

The behaviors of the bridge at large-scale earthquakes were analyzed by non linear time history response analyses with the analytical model of the entire bridge system.

The results of analyses are shown in Table 2. In the longitudinal direction, reaction forces of bearings at 1A significantly exceeded the allowable values. So these bearings will be broken at large-scale earthquakes. On the other hand, in the transverse direction, reaction forces of bearings at 2P and 4A exceeded the allowable values, and shear force of 3P pier also exceeded the allowable values. So these components will suffer damage at large-scale earthquakes.

As the bearings at 2P and 3P are fixed type, it is necessary to know the bridge behavior in the longitudinal direction after broken of 1A fixed bearings. The fixed bearings at 1A were transposed to movable bearings in the analytical model, and seismic response analysis was performed again. This result is shown in Table 3. It shows that 2P pier and 3P pier may be broken by the bending moment when 1A fixed bearings were broken. From these results, it was found that the bridge did not have enough seismic performance for large-scale earthquakes, and needed to be reinforced.

Table 2 SEISMIC VULNERABILITY EVALUATION OF EXISTING BRIDGE

Direction of Input Ground Motion	Evaluatin Items	Type of Level 2 E.Q.	1A		2P		3P		4A	
			Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value
Longitudinal	Curvature ($10^{-6}/m$)	Type 1	---	---	64	2,812	685	2,457	---	---
		Type 2	---	---	255	7,213	3,155	5,305	---	---
	Shear Force (kN)	Type 1	---	---	1,519	4,316	3,102	4,897	---	---
		Type 2	---	---	3,792	4,796	5,006	5,442	---	---
	Reaction force at bearing (kN)	Type 1	23,510	9,408	1,439	6,402	3,207	6,402	---	---
		Type 2	63,276	9,408	2,908	6,402	4,323	6,402	---	---
Transverse	Curvature ($10^{-6}/m$)	Type 1	---	---	634	1,260	83	892	---	---
		Type 2	---	---	2,216	3,268	472	2,233	---	---
	Shear Force (kN)	Type 1	---	---	9,868	13,295	7,841	10,225	---	---
		Type 2	---	---	12,865	13,534	12,016	10,530	---	---
	Reaction force at bearing (kN)	Type 1	2,721	9,408	7,843	6,402	2,482	6,402	5,602	3,942
		Type 2	7,385	9,408	9,450	6,402	4,397	6,402	9,258	3,942

*) value at the most critical section is described at each

Table 3 SEISMIC VULNERABILITY EVALUATION WITH BEARING FAILURE AT 1A

Direction of Input Ground Motion	Evaluatin Items	Type of Level 2 E.Q.	1A		2P		3P		4A	
			Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value
Longitudinal	Curvature ($10^{-6}/m$)	Type 1	---	---	12,659	3,018	2,615	2,551	---	---
		Type 2	---	---	7867	7,213	1,052	5,628	---	---
	Shear Force (kN)	Type 1	---	---	3,000	4,024	2,477	4,897	---	---
		Type 2	---	---	2,917	4,796	3,605	5,442	---	---
	Reaction force at bearing (kN)	Type 1	---	---	2,708	6,402	3,207	6,402	---	---
		Type 2	---	---	2,757	6,402	4,323	6,402	---	---

*) value at the most critical section is described at each

4. Seismic reinforcement

4.1. The measures in the longitudinal direction

It is necessary for the measure of reinforcement to avoid giving the original structural system any unfavorable influence except when large-scale earthquakes occur, and to have ease of execution and economical efficiency. One solution for improving of the seismic performance is to strengthen the function to fix the superstructure horizontally at the bearings of 1A. However, since the horizontal reaction force of bearings at 1A was very large, and there was no place in which such huge devices could be installed in 1A abutment. Therefore, other solutions were studied considering an entire bridge system, on assumption that 1A is free in the longitudinal direction.

Since the broken of the fixed bearings at 1A causes the destructions of 2P and 3P piers, the measure to install the viscous dampers connecting the superstructure and abutments was selected. The damper is mainly composed of a steel cylinder, a steel piston, and silicone resin filling, as shown in Fig. 4. The relationship between the resisting force and the displacement is as shown in Fig. 5, and its resisting force is proportional to the 0.1st power of its velocity ($F = c \times V^{0.1}$, F : resisting force, V : velocity, c : constant value).

Because the resisting force of this damper is small enough in low velocity to make the dampers give no influence to the structural system except when large-scale earthquakes occur.

In the study of the damper specification, the seismic response analyses with varying resisting force of dampers were performed. It was found that the required resisting force, in order to reduce the responses of both piers lower than the allowable values, and in order to avoid the collision between the superstructure and the abutments, was 16,000kN/bridge or more. Then, considering types of the existing damper products and their installing space, we decided that 6 dampers, each 1,500kN/damper, install at both abutments (the whole resisting force 1,500kN/damper x 6 dampers x 2 abutments = 18,000kN/bridge). The result of the seismic response analysis in consideration of resisting force of dampers is shown in Table 4. It shows that the responses of both piers and the displacement of the superstructure are within their allowable values.

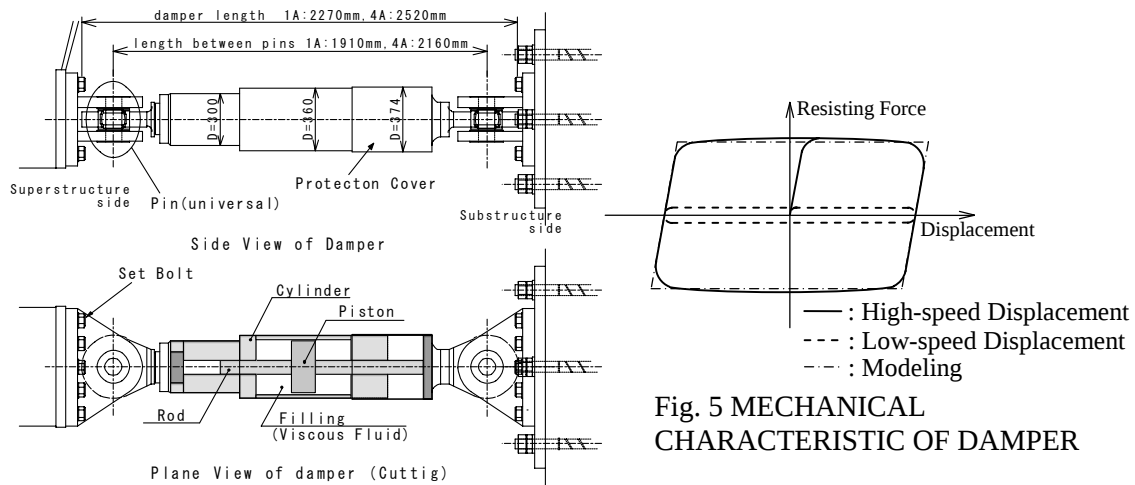


Fig. 4 SCHEMA OF VISCOUS DAMPER

Fig. 5 MECHANICAL CHARACTERISTIC OF DAMPER

Table 4 SEISMIC VULNERABILITY EVALUATION WITH DAMPERS

Direction of Input Ground Motion	Evaluatin Items	Type of Level 2 E.Q.	1A		2P		3P		4A	
			Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value
Longitudinal	Curvature ($10^{-6}/m$)	Type 1	---	---	53	2,701	590	2,551	---	---
		Type 2	---	---	1427	7,213	4,006	5,638	---	---
	Shear Force (kN)	Type 1	---	---	1,591	6,016	4,863	6,826	---	---
		Type 2	---	---	2,714	4,796	5,357	5,674	---	---
	Reaction force at bearing (kN)	Type 1	---	---	2,288	6,402	3,674	6,402	---	---
		Type 2	---	---	3,677	6,402	5,435	6,402	---	---
	Displacement of girder (mm)	Type 1	8.3	100	---	---	---	---	7.9	250
		Type 2	69.9	100	---	---	---	---	70.3	250

*) value at the most critical section is described at each , Considering influence of reinforcement of the transverse direction

4.2. The measures in the transverse direction

In the transverse direction, the measures of the bearings at 2P and 4A, and the measures against the shear force at 3P pier are required.

The horizontal forces at 2P and 4A cause destruction of these bearings, but they are small enough not to make these substructures any damage. Therefore the equipments, which prevent the superstructure movement in the transverse direction, at 2P and 4A were selected to install.

The 3P pier needs reinforcement against the shear force. It is important to minimize the increase in its mass and rigidity of 3P pier as little as possible to avoid the influence of seismic behaviors in another direction. We decided that the wall thickened 500mm with reinforced concrete in order to increase the shear capacity of 3P pier, as shown in Fig. 6.

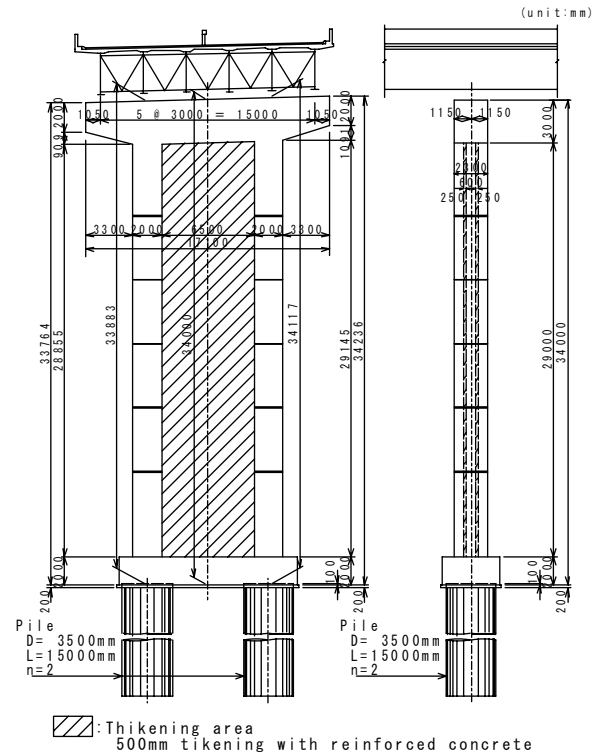


Fig. 6 THICKENING OF WALL AT 3P PIER

4.3. Dampers

Photo 1 shows the dampers which were installed in the 1st Ibi viaduct (at 4A). Since the safety of the bridge at large-scale earthquakes is dependent on these dampers, they are required to have their functions for a long time, and to work certainly in case of any earthquakes. It is necessary to get their condition correctly, and to maintain them in good condition.

It is thought that the dampers would have high durability, since it has been checked that the silicone resin, which is used as the resistance object of the damper, has high chemical stability and durability by the accelerated weathering test. However, the durability of the dampers has not yet checked



Photo 1 DAMPERS INSTALL (4A)

directly. And the dampers had begun to apply to bridges recently, so there are not so many dampers installed for long period. The information about their durability is not enough. Therefore, in order to get reliable information more, we and the maker of the dampers have started the damper's exposure test.

In this test, the small damper of same type installed in the bridge is used. This test damper is designed to have the resistance of 100kN. After its resistance is checked, it will be placed on the open air test site, in order to be exposed to the actual atmosphere such as sunlight and rain. This exposure test is planned to be continued for ten years, and the performance of the test damper is planned to be examined two or three times within this period. This test will show useful data about the durability of the damper. We are going to determine the maintenance plan of dampers, from this data.

5. Conclusion

For seismic reinforcement of the existing bridge, it is very important to know the bridge's condition. The measures of reinforcement have to be selected according to the situation of the bridge, and from the viewpoint of economical efficiency.

In the 1st Ibi viaduct, as shown in this paper, the viscous dampers, which were expected to reduce the seismic force, were used. It is thought that dampers are effective for some types of bridges, in case the seismic reinforcement of their substructures is difficult like the 1st Ibi viaduct. In the Kobe-Awaji-Naruto Expressway, dampers were applied to no less than four bridges except this bridge.

Maintenance of the dampers is also very important after installation in view of the permanency of the reinforcement effect. However, there is not enough information about the durability of the dampers, and its maintenance plan is not established clearly. Therefore, it is necessary to accumulate data about durability, and to establish the maintenance plan.

References

Japan Road Association; Specifications for Highway Bridges Part 5 Seismic design 1980
Japan Road Association; Specifications for Highway Bridges Part 5 Seismic design 2002
K.Endo, S. Kawabata, S. Ogo; A Study on Seismic Retrofit of the Honshu-Shikoku
Bridges Using Isolation and Dissipation Devices, 2007IABSE, Sep. 2007