Seismic Analysis of the Brent-Spence Bridge

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SEISMIC ANALYSIS  
OF THE  
BRENT - SPENCE BRIDGE

by

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in cooperation with

Transportation Cabinet  
Commonwealth of Kentucky

and

Federal Highway Administration  
U.S. Department of Transportation

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**SEISMIC ANALYSIS OF THE BRENT-SPENCE BRIDGE**

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**Abstract**

This report presents the findings of a seismic evaluation of the Brent-Spence bridge. The bridge is a double-deck cantilever through truss which was constructed in 1961; it is a critical highway link conveying I-75 over the Ohio river. While not yet subjected to a moderate or major earthquake, the bridge is within the influence of the New Madrid, Wabash Valley, and Anna seismic zones.

The seismic evaluation program consisted of field testing and seismic analysis. The vibrational properties of the main bridge were determined through field testing and used to calibrate a 3-D finite element model. The model was then subjected to time histories of projected earthquakes in the aforementioned seismic zones, including a maximum credible earthquake of magnitude 8.5 on the Richter scale at New Madrid. The Covington approach spans were analyzed using simple models and response spectrum analyses.

Analytic results indicate that the main bridge will survive the projected maximum credible earthquake in the elastic range without significant damage and no loss of span; however, the Covington approach spans are vulnerable to loss of span failure in this event. Hence retrofit and a thorough seismic analysis of the approach spans is required.

**Key Words**
Earthquake, Seismic Analysis, Time History, Response Spectra, Span-Loss Collapse, Retrofitting Priority

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EXECUTIVE SUMMARY

The October 17, 1989 Loma Prieta earthquake brought attention to the seismic risk of double-deck bridges and elevated double-deck freeway structures. Following the earthquake, the Federal Highway Administration commissioned the seismic evaluation of all double-deck bridges located in seismically active regions.

In Kentucky, there are two double-deck bridges. These are the Brent-Spence bridge on Interstate 75 connecting Covington (Kentucky) to Cincinnati (Ohio) and the Sherman-Minton Bridge on Interstate 64 connecting Louisville (Kentucky) to New Albany (Indiana). Both bridges cross the Ohio River, and are located in regions that are influenced by the seismically active New Madrid zone and other local zones. This report concentrates on the seismic evaluation of the main Brent-Spence Bridge and the Covington approach (Figures 1 and 2).

Given the state of knowledge of seismic design and seismology when the bridge was designed and the low seismicity of the region, the designers presumed that wind loads governed lateral design and traffic loads governed longitudinal design. As a result, the structure's response to dynamic earthquake loading has not been previously evaluated.

SCOPE OF STUDY

The primary aim of this study was to assess the structural integrity of the Brent-Spence bridge when subjected to a seismic event from the New Madrid, Wabash Valley, or Anna seismic zone. To achieve this, the scope of work was divided into several tasks as follows: experimental analysis of the main bridge (or field testing), analytical modeling, site specific ground motion, and seismic response analysis.

Experimental Analysis

The ambient vibration properties of the main bridge were determined through field testing using traffic to excite the structure. The purpose of measuring the ambient vibration properties was to determine the mode shapes and the associated natural frequencies. These vibration properties
were subsequently used as the basis for calibrating the analytical computer model for seismic response analysis.

Analytical Modeling

Applied dynamic loads and responses were regarded as disturbances of the current dead load configuration of the bridge.

A three dimensional finite element model of the main bridge was used for free vibration and seismic response analyses. The model was calibrated by comparing the free vibration analysis results with ambient vibration properties from field testing. Once calibrated, the model was then used for seismic response analysis.

The approach spans were modeled using simple one or two degree of freedom systems. The mass was concentrated over the piers at points of longitudinal fixity (i.e. lollipop model). Seismic response was analyzed in the longitudinal direction only.

Site Specific Ground Motion

The site specific ground motion scenarios were developed to represent probable earthquakes that may occur in the New Madrid, Wabash Valley, and Anna seismic zones. A local seismic event, defined as being 12.5 miles (20 km) from the site of the bridge, was also considered. Time histories and response spectra were generated for a number of events. The time histories were then used in the seismic analysis of the main bridge while the response spectra were used for analyzing the approach spans.

Seismic Response Analysis

The three dimensional model of the main bridge was subjected to the time histories of the aforementioned earthquakes to determine maximum displacements, forces and stresses. Maximum relative displacements were determined at a number of locations including the top and bottom of the piers, and between the upper and lower deck levels. The maximum forces and stresses were determined at a number of locations including the piers, towers, and anchor bolts at fixed bearings.
For the approach spans, the seismic analysis dealt only with the potential for loss of span due to excessive longitudinal displacement along the highway main line. The seismic analysis of the models for the approach spans was conducted using the response spectra. The maximum relative longitudinal displacements at expansion joints were determined and compared with the maximum support length to check for susceptibility to span loss.

RECOMMENDATIONS

Main Bridge

The seismic analysis indicates that the main bridge will resist the maximum credible earthquake in the elastic range without any damage or loss of span. Consequently, retrofitting is not required.

Approach Spans

The primary concern in this study for the approach spans is the potential for loss of span due to excessive longitudinal displacement. Based on the findings of the seismic analysis, the following actions are recommended:

1- Retrofit of the upper deck expansion joints at pier 1 thru 9 (Figure 3) is strongly recommended since the capacity/demand ratio C/D do not meet the current standard of AASHTO Specifications.

It is recommended that properly designed cable restrain system be installed at these expansion bearing seats. The components are easily available and installation is not outside the scope of local contractor capabilities.

2- Only the main-line approach spans were considered in this study. The access ramps were not evaluated and may be vulnerable to loss of span. It is recommended that the access ramps on both approaches be evaluated for possible loss of span.
3- In view of the results for the approach spans, it is recommended that a detailed three dimensional seismic analysis, including strength check of critical components, be conducted on the approach spans and the ramps.

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1. **Introduction**

The October 17, 1989 Loma Prieta earthquake brought to the attention of the public the seismic risk to double-deck bridges and elevated double-deck freeway structures. The partial collapse of the San Francisco-Oakland Bay Bridge (Fig. 1.1) and the Cypress Viaduct portion of Interstate 880 (Fig. 1.2) not only caused loss of life, but also caused considerable problems to the transportation infrastructure. The Bay bridge was unusable for one month and transbay commuters were forced to commute on ferries or the crowded Bay Area Rapid Transit system (Reference). The Cypress viaduct has not yet been replaced. Following the Loma Prieta earthquake, the Federal Highway Administration commissioned the seismic evaluation of all double-deck bridges located in seismically active regions.

In Kentucky, there are two double-deck bridges that cross the Ohio River to adjoining states. These are the Brent-Spence Bridge (Fig. 1.3) on Interstate 75 connecting Covington (Kentucky) to Cincinnati (Ohio) and the Sherman-Minton Bridge on Interstate 64 connecting Louisville (Kentucky) to New Albany (Indiana). Both bridges are located in regions that are influenced by the seismically active New Madrid zone (Fig. 1.4) and other local zones. This report will concentrate on the seismic evaluation of the main Brent-Spence Bridge and the Covington approach.

The Brent-Spence Bridge is located well outside of any generally accepted seismic zone. The nearest seismic zones to the site are the New Madrid, Wabash Valley, and Anna seismic zones as shown in Figure 1.4. Of these three zones, the New Madrid has historically produced the largest events. In the three-month period from December, 1811 through February, 1812, three of the most severe earthquakes in American history occurred in the New Madrid seismic zone. Each earthquake is estimated to have measured over 8.0 on the Richter scale (Nuttli 1983). The zone is named after the small river town of New Madrid on the Mississippi River, as a result of the close proximity of the epicenter of these earthquakes to the town. The New Madrid seismic zone is regarded by seismologists and disaster response planners as the most hazardous earthquake zone east of the Rocky Mountains.
1.1 The Brent-Spence Bridge

The Brent-Spence bridge is a double deck cantilever through truss. It is externally statically determinate with no redundant supports, which is typical of this type of construction. The upper deck carries three lanes of southbound traffic. The lower deck currently conveys four lanes of northbound traffic; it was designed for three and subsequently modified.

The Covington approach, consists of two ramps and the I-75/I-71 mainlines merging into a double deck structure to meet the Brent-Spence bridge at pier IV.

The structural analysis techniques which were used to design this bridge provided reasonable estimates of stresses and deflections due to static dead loads and traffic loads. Wind load stresses and deflections were conservatively calculated using lateral equivalent loads, and seismic response was neglected. Given the state of knowledge of seismology and seismic design, and the low seismicity of the region, it was presumed that wind loads governed lateral design and traffic loads governed longitudinal design. As a result, the structure's response to dynamic earthquake loading has not been previously evaluated.

1.2 Scope of Study

The primary aim of this study is to assess the structural integrity of the Brent-Spence bridge when subjected to a seismic event from the New Madrid, Wabash Valley, or Anna seismic zone. To achieve this, the scope of work was divided into several tasks as follows: Experimental analysis of the main bridge (or field testing), analytical modeling, site specific ground motion, and seismic response analysis.

1.2.1 Experimental Analysis

The ambient vibration properties of the main bridge have been determined through field testing using traffic to excite the structure. The purpose of measuring the ambient vibration properties is to determine the mode shapes and the associated natural frequencies. These vibration properties are subsequently used as the basis for calibrating the analytical computer model for seismic response analysis.
1.2.2 Analytical Modeling

Applied dynamic loads and responses are regarded as disturbances of the current dead load configuration of the bridge.

A three dimensional finite element model of the main bridge was used for free vibration and seismic response analyses. The model was calibrated by comparing the free vibration analysis results with ambient vibration properties from field testing. Once calibrated, the model was then used for seismic response analysis.

The approach spans were modeled using simple one or two degree of freedom systems. The mass was concentrated over the piers at points of longitudinal fixity (i.e. lollipop model). Seismic response was analyzed in the longitudinal direction only.

1.2.3 Site Specific Ground Motion

The site specific ground motion scenarios were developed to represent probable earthquakes that may occur in the New Madrid, Wabash Valley, and Anna seismic zones. A local seismic event, defined as being 12.5 miles (20 km) from the site of the bridge, was also considered. Time histories and response spectra were generated for a number of events. The time histories were then used in the seismic analysis of the main bridge while the response spectra was used for analyzing the approach spans.

Based on the analysis in chapter 5, the Brent Spence bridge is subjected to a 6.5 m<sub>b.Lg</sub> (7.4 M<sub>s</sub>) magnitude earthquake at New Madrid, a 6.5 m<sub>b.Lg</sub> (7.4 M<sub>s</sub>) at the Wabash Valley, and 5.25 m<sub>b.Lg</sub> (4.6 M<sub>s</sub>) Local earthquake. m<sub>b.Lg</sub> = body wave magnitude, and refers to to the 1-sec-priod Lg wave magnitude scale proposed by Nuttli (1973), and M<sub>s</sub> = surface wave magnitude. A New Madrid 7.3 m<sub>b.Lg</sub> (8.5 M<sub>s</sub>) earthquake, which has a very low probability of occurring, is recommended for consideration as the maximum credible earthquake.

1.2.4 Seismic Response Analysis

The three dimensional model of the main bridge was subjected to the time histories of the aforementioned earthquakes to determine maximum displacements, forces and stresses. Maximum relative displacements were
determined at a number of locations including the top and bottom of the piers, and between the upper and lower deck levels. The maximum forces and stresses were determined at a number of locations including the piers, towers, anchor bolts at fixed bearings, and etc.

For the approach spans, the seismic analysis dealt only with the potential for loss of span due to excessive longitudinal displacement along the main line of the bridge. The seismic analysis of the models for the approach spans was conducted using the response spectra. The maximum relative longitudinal displacements at expansion joints were determined and compared with the maximum support length to check for possible span loss.
Figure 1.1  Aerial Photo Looking North to the Collapsed Upper and Lower 50-ft (15.25 m) Deck Spans of the Bay Bridge
(Source: Lew, 1990)
Figure 1.2  Aerial Photo Looking West to the Collapsed Portion of Cypress Structure Between Bents 94-100 (Source: Lew, 1990)
Figure 1.3 The Double-Deck Brent-Spence Bridge
Figure L.4 Seismic Zones (modified after Nuttli and Herrmann, 1978)
2. Description of the Bridge

The design of the Brent-Spence Bridge was completed by Modjeski and Masters in March 1961; it was constructed and opened to traffic the same year.

The bridge is a double deck cantilever through truss, a bridge type commonly employed for spans of 600' (183 m) to 1500' (457 m) through the mid 1970's. The total length of the bridge is 1736'6" (529.3 m) with a center span of 830'6" (253.1 m) and symmetric side spans of 453'0" (138.1 m). See Fig. 2.1 for the plan and elevation of the bridge and Fig. 2.2 for pier elevations.

The Covington approach, shown in Fig. 2.3, consists of two ramps and the I-75/I-71 mainlines merging into a double deck structure to meet the Brent-Spence bridge at pier IV.

The bridge was rehabilitated in 1984/1985 at which time the barriers, drainage, and lighting were refurbished, deck joints were repaired, an additional lane was added to the lower deck, and diagonals U3-L4 and U43-L42 were strengthened (for the additional live load). The approach structures were rehabilitated concurrently; deck slab, barriers, drainage, and lighting were refurbished and the Covington approach mainlines were widened.

2.1 The Bridge Superstructure

The superstructure can be described in terms of the vertical load system, the lateral load system, and the floor system.

The vertical load system (see Fig. 2.1 Elevation) consists of a central "Suspended Span" of 453'0" (138.1 m) supported via eyebar hangers from two continuous side spans which are cantilevered over the river piers (Piers II & III). The continuous side spans consist of an "Anchor Arm" of 453'0" (138.1 m) which spans from the end pier (Piers I & IV) to the river pier and a "Cantilever Span" which is cantilevered 188'9" (57.5 m) from the inner pier. The "Anchor Arm" counterbalances the "Cantilever Span" which in turn supports the "Suspended Span". The vertical trusses are spaced 53'0"
(16.2 m) apart.

The lateral load system consists of horizontal trusses in the top and bottom chord planes combined with portals and sway bracing between the two vertical trusses. The top chord lateral load system is discontinuous at the hanger between the "Cantilever Span" and the "Suspended Span" (panel point 17). The bottom chord lateral load system transfers lateral load from the "Suspended Span" to the "Cantilever Span" through wind links as shown in Fig. 2.4.

2.1.1 Hinge at Panel Points 17 and 28

The joint between the "Suspended Span" and the "Cantilever Span" at panel point 17 was designed for static vertical and lateral loads only. As a result, it is kinematically indeterminate due to lack of longitudinal restraint.

Eyebar hangers provide vertical support. Wind links deliver lateral restraint. Longitudinal sliding joints (Figure 2.5) in both the top and bottom chords are designed for free thermal expansion, they limit longitudinal displacement to approximately 5" (12.7 cm). Rotation due to vertical and transverse bending are free to take place. Torsional rotation is prevented primarily by the mass of the structure (safety factor against overturning); it is further restrained by the sliding joints in the top and bottom chords of the truss.

2.1.2 Floor System

The floor system consists of a 7" concrete slab supported by longitudinal WF stringers carried by transverse built-up box floor beams. The floor beams span 53'0" (16.2 m) between the main trusses and are rigidly attached to the truss verticals. The stringers span 37'9" (11.5 m) between floor beams and are continuous over 3 or 4 spans on the lower deck and 5 or 6 spans on the upper deck. There are expansion joints at the discontinuities in the floor beams and both the upper and lower decks have expansion joints at panel point 17. The slab is not composite with the stringers.
2.2 The Bridge Bearings

The superstructure is supported by expansion bearings (which permit vertical bending rotation and longitudinal translation) at piers I and IV and fixed bearings (which only allow vertical bending rotation) at piers II and III.

The fixed bearings are a standard pinned bearing design consisting of a cast steel upper shoe supported on a 1'2" (35.6 cm) diameter forged steel pin which bears on a cast steel bottom shoe. The upper shoe is bolted to the bottom chord of the truss and the bottom shoe is rigidly attached to the pier via anchor bolts.

The expansion bearings consist of a pinned shoe assembly supported by a roller assembly. The pinned shoe assembly consists of a cast steel upper shoe supported by an 8" (20 cm) diameter forged steel pin which bears on a cast steel bottom shoe. The bottom shoe has a pintled stainless steel roller plate attached to its base which bears on five stainless steel rollers. The rollers rest on another pintled stainless steel roller plate which is fastened to a base plate. The upper shoe is bolted to the bottom chord of the truss and the base plate is attached to the pier via anchor bolts.

2.3 The Bridge Substructure

The end piers (Piers I and IV) are three column portal bents on pile footings (see Fig. 2.2). Pier I is 26'0" (7.9 m) tall and pier IV is 33'0" (10.0 m) tall. The outer columns of the bent support both the Brent-Spence bridge and the approach structure while the inner column supports only the approach structure. The river piers (Piers II and III) are wall type piers, consisting of two large tapered shafts (approx. 15 ft. or 4.6 m in diameter) connected by an 8'0" (2.4 m) thick wall supported by a caisson foundation (see Fig. 2.2). Pier II is 30'0" (9.1 m) tall while pier III is 28'0" (8.5 m) tall; the caissons are 56'6" (17.2 m) and 57'6" (17.5 m) deep, respectively.
2.4 The Approach Structure

The Covington Approach consists of four structures, the lower deck approach (northbound mainline), the upper deck approach (SB mainline), ramp B, and ramp H; plan and elevation shown in Fig. 2.3. The approach superstructure is composite slab and beam construction. The piers are reinforced concrete portal bents on pile foundations. Typical pier configurations are illustrated in Fig. 2.6.

The lower deck approach consists of 18 spans with an overall length of 1185.1' (361.2 m). Ramp H merges with the lower deck at Pier 8. Ramp H is a two span structure 104.0' (31.7 m) long.

The upper deck approach consists of 18 spans with a total length of 1181.4' (360.1 m). Ramp B splits from the upper deck at Pier 9. Ramp B has 4 spans for a total length of 239.5' (73.0 m).
Figure 2.2 Pier Elevations and sections
Figure 2.5  Superstructure - Longitudinal Joints at Panel Point 17
Figure 2.6 Covington Approach - Typical Piers
3. **Experimental Analysis**

3.1 **Introduction**

All data were collected during a 4-hour time window on Sunday morning when traffic was light. Ambient vibrations from traffic were used to excite the bridge. The two outer northbound lanes (bottom deck) and the outer lane on the western side of the bridge in the southbound direction (top deck) were closed for traffic during testing (Figure 3.1). All testing was conducted on the south end of the bridge between test locations 1 and 13 (Figure 3.2). Measurements were performed at each of the locations beginning at the southern most pier of the main span (panel 1 in Figure 3.2) and continuing to the centerline of the bridge (location 13 in Figure 3.2). Due to symmetry, only 13 locations were tested. All measurements were taken by placing the instruments on the pavement just above the floor stringers as shown in Figure 3.1. Instruments were placed on the deck, due to the limited access to the actual floor beams and the time constraints involved. Each instrument was placed with its longitudinal axis aligned parallel to the longitudinal axis of the bridge. At each location, the instruments were held in place with cloth bags filled with lead shot. Each bag weighed approximately 25 pounds (11.3 kg).

3.2 **Equipment**

The equipment used to measure the acceleration time histories consisted of a triaxial accelerometer in conjunction with its own data acquisition system. The system which was used consisted of Kinematics SSA-2 digital recording strong motion accelerograph. Two of the units contained internal accelerometers, the remaining two were connected to Kinematics FBA-23 force balance accelerometers. Each of the accelerometers is capable of measuring accelerations of +/- 2g's with a frequency response of DC-50 Hz. All data were sampled using a 200-Hz sampling rate. All data were stored internally on the SSA-2, then downloaded to a personal computer in the laboratory. Each of these units were triggered simultaneously using laptop personal computers connected to each SSA-2. A nominal 40-second record was obtained at each location.
3.3 Testing Sequence

A reference location (hereinafter referred to as "base station") was selected at location 9 on the bottom deck on the east side of the bridge (Figure 3.1). One of the accelerometers remained at the base station throughout the testing sequence. The remaining three instruments (hereinafter referred to as "movable stations") were moved to each of the locations from number 13 to number 1 (Figure 5.2). At each location, data were recorded simultaneously on all instruments. Data collection began at the center of the bridge (location 13) and continued through location 1.

One set of measurements consisted of recording an acceleration time history on all stations simultaneously. Once the data were collected, the movable stations were moved to the next location while the base station remained stationary. This sequence was repeated at each of the 13 locations.

3.4 Data Analysis

Vertical accelerations from test location 6 are given in Figure 3.3. Similar results were obtained for the longitudinal and transverse directions, and for each location. For tests conducted at locations 1 through 8, the base station receives the increased vibration from the traffic stream later in the time window, since traffic is moving from location 1 to location 13.

Once the data have been downloaded, a Fast Fourier Transform (FFT) is performed on each time record (Harris, 1988). The results of the FFT are represented as both magnitude and phase. For the vertical direction, and when the moveable stations were placed at location 6, the results of the FFT of the base station data are given in Figures 3.4 and 3.5.

A three-point moving average was applied to the FFT magnitude data shown in Figure 3.5, and the results are shown in Figure 3.6. It is clear that the three-point moving average provides a cleaner signal to analyze.

Three point moving averages were calculated for all magnitude data prior to the mode shape analysis. To determine the mode shape of the structure at a specific frequency, which is generally associated with a peak in the magnitudes of the FFT, the magnitude at each location is divided by the corresponding magnitude of the reference station. If the result of this
division is small, this indicates very little movement between the station being analyzed and the base station. These peaks do not always occur at exactly the same frequency at all locations. A typical distribution of frequency is given in Figure 3.7. It may be seen from this figure that the peak in the magnitude varied from 0.63 Hz to 0.68 Hz, with a majority of the peaks occurring, at 0.66 Hz. In this case, the 0.66 Hz would be compared to the results derived from the theoretical analysis.

The phase portion of the FFT is utilized to determine the sign of the ratio which was obtained by dividing the results for each station by the results from the base station. Due to the sensitivity of the phase to frequency, a three-point moving average of the phase was not utilized in determining the sign of ratio. In some cases, it was necessary to determine the sign based on the evaluation of the phase at adjacent locations and at other stations (east, west, top). This procedure was necessary since there were large phase shifts observed across just one increment in frequency (0.025 Hz). By evaluating each of the stations in this manner, the deflected shape of the bridge structure at a given frequency was be determined.

The mode shape was obtained by plotting the ratio at each station with the appropriate sign versus station location. An example of this type of plot is given in Figure 3.8 for the fundamental vertical mode shape. Mode shapes were determined for various frequencies for all three components (vertical, transverse, and longitudinal). For comparisons with the theoretical analysis, the mode shapes were determined along the eastern side of the bridge on the lower deck.
Figure 3.1 Bridge Cross Section Showing Placement of Instruments or Stations (the Top, West and East Stations are Moveable Stations, While the Base Station is Stationary at Test Location 9 Shown in Figure 3.2)
Figure 3.3 Vertical Accelerations When Moveable Stations are Placed at Location 6 in Figure 3.2 (Top, West, East and Base Stations are Identified in Figure 3.1)
Figure 3.4 Fast Fourier Transform for the Phase in the Vertical Direction at Base Station While Moveable Stations are Placed at Location 6 in Figure 3.2.
Figure 3.5 Fast Fourier Transform for the Magnitude in the Vertical Direction at Base Station While Moveable Stations are Placed at Location 6 in Figure 3.2.
Figure 3.6 Three-Point Moving Average for the FFT of the Magnitude Shown in Figure 3.5.
Figure 3.7 Frequency Distribution in the Vertical Direction When the Moveable Stations Are Placed at Location 6 in Figure 3.2.
Figure 3.8 Experimental Fundamental Vertical Mode Shape.
4. **Analytical Modeling and Verification**

Applied dynamic loads and responses may be regarded as disturbances to the dead load configuration; therefore, the analysis of the main bridge and the approach spans is based on the current dead load state of the structures.

### 4.1 The Main Bridge

Given the general dynamic characteristics of cantilever truss bridges and the proximity and activity of the study seismic zones, the main bridge model was expected to remain elastic and displacements were anticipated to be small enough to neglect second order effects. Hence, linear elastic small displacement analysis was deemed appropriate. The results of the earthquake response analysis validates these assumptions.

A 3-D linear elastic finite element model (Figure 4.1) of the main bridge was generated using the computer program ANSYS (Swanson 1991). Developed for both the free vibration analysis and the earthquake response analysis, the model represents the structure in its current as-built configuration. It has a total of 1536 elements (1078 beam elements plus 458 truss elements) and 756 nodes. The total number of active degrees of freedom (DOF) was reduced by establishing internodal master/slave DOF relationships where physically appropriate.

Free vibration analysis is a key process in the dynamic analysis of a structure; the resulting natural period and normal modes succinctly describe the dynamic characteristics of a complex structure. The analytical model was calibrated by comparing free vibration analysis results with ambient vibration measurements (see section 4.1.3).

Direct time integration (i.e. Time History) analysis is the most rigorous earthquake response analysis technique in current use; it eliminates the modal combination error associated with the more commonly used multi-mode spectral analysis. Due to the importance of the bridge and
the lack of seismic considerations in its design, direct time integration was used to perform the earthquake response analysis (see Chapters 5 and 6 for details).

4.1.1 Structural Modeling

Three element types were used to model the main bridge: the three-dimensional elastic beam element (12 DOF), the three-dimensional spar element (6 DOF), and the spring-damper (6 DOF) element.

The eyebars and the diagonals of the lateral load system were modeled using 6 DOF spar elements. Spring-damper elements were used to model foundations/soil-structure interaction. All other members were modeled as 12 DOF beam elements.

Tension-only eyebars can be modeled as spar elements provided the member force induced by the dead load and the multiple support excitations remains tensile; the seismic analysis results satisfy this condition.

Modeling of the Deck

The floor system consists of a nominal 7-inch (18 cm) concrete deck supported by eight longitudinal wide flange stringers carried in turn by transverse floor beams which frame into the vertical truss system at each panel point. To reduce the model to a more tractable size, the deck and stringers were modeled as a series of beam elements connected to a node at the midspan of each floor beam.

The beam elements have section properties and lumped masses equivalent to those of the deck section they model; the kinematic integrity of the transverse floor beam elements is preserved by establishing a master/slave transformation between the midspan node and the end nodes.

Modeling of the Hinge

The as-built structure is kinematically indeterminate due to the lack of longitudinal restraint in the hinges which join the cantilever arm to the Seismic Analysis
suspended span. Kinematic indeterminacy was eliminated from the structural model by introducing a massless longitudinal spar member with a very small cross-sectional area at the hinge.

Modeling of the Bearings

The bearings at the river piers are "fixed" and those at the end piers are "expansion" (see Section 2.2 for a more complete description). The "fixed" bearings at the river piers were modeled by simply releasing the rotational DOF in the bending plane of the vertical truss system (the $\theta_x$ DOF).

The "expansion" bearings at the end piers were modeled by establishing nodes in the bottom chord of the truss and the top of the pier at the bearing centers and coupling all DOF except the longitudinal translation and the vertical bending rotation (the $\theta_x$ and $u_z$ DOF). The coupled nodes provide direct output of the relative displacement between the top and bottom shoes of the bearings and thus indicate if the translation has exceeded the bearing seat expansion capacity.

Modeling of the Foundations and Piers

The river piers (piers II and III in Figures 2.1 and 2.2) are wall type piers supported by caisson foundations and the end piers (piers I and IV in Figures 2.1 and 2.2) are three column portal bents supported by pile foundations. The bridge is founded on a loose cohesionless soil consisting of a mixture of sand, silt and clay.

The river piers were treated as single cantilever beam-columns with section properties equivalent to those of the actual bridge piers. Rigid beam elements with a master/slave transformation between the midspan node and the end nodes attach the piers to the bridge truss at the bearing points. The end piers were modeled as a multi-column frame.

While the piers can be modeled relatively accurately, the foundations pose a problem; there are too many uncertainties due to the heterogeneous nature of soil.

Often, foundations are modeled as being infinitely rigid resulting in
overstated foundation stiffness, underestimated deflections, and overestimated stresses. Alternatively, they can be modeled using nonlinear springs which represent the soil characteristics, the foundation stiffness, and the interaction between the soil and the foundation. The difficulty in this approach is in determining the spring stiffness coefficients. Accurate evaluation of these coefficients depends on the following:

1. The stiffness of the foundation including the pile cap or caisson.

2. The soil properties including the constitutive relationship and soil stability.

3. The boundary conditions including ultimate skin friction, and end bearing load-displacement characteristics.

4. For pile foundations, the group geometry including the number, direction, and spacing of the piles and the influence of piles on the soil properties.

Three models are presently in common use in engineering practice (Lam, Martin, & Imbsen 1990): 1) the equivalent cantilever model, 2) the uncoupled base springs model, and 3) the coupled foundation stiffness matrix model.

To provide conservatively bounded stress and displacement results the foundations were modeled using both the linear uncoupled base springs foundation model and the rigid foundation condition.

The stiffness coefficients for the linear uncoupled base springs foundation model were derived using the method presented by Lam, Martin, and Imbsen (Lam, Martin, & Imbsen 1990) for the coupled foundation stiffness matrix model and dropping the off-diagonal terms. For the pile foundations, the coupling terms may be dropped with minimal error because the pile group is very large (Lam & Martin 1986). The caisson stiffness may be assumed to be infinite; hence, passive soil pressure is the only component considered in estimating the foundation stiffness and the resulting foundation stiffness matrix is diagonal (Bowies 1988). Linear springs may be used when small foundation displacements (less than 0.5 inch or 1.3 cm) are expected (Lam & Martin 1986).
4.1.2 Free Vibration Analysis

In modal analysis, undamped free vibration is considered, and the bridge vibration is governed by the equilibrium equation

\[ Ku + M\ddot{u} = 0 \]  \hspace{1cm} (4.1)

in which \( K \) is the structure (or global) stiffness matrix, \( M \) is the structure mass matrix, \( u \) is the nodal displacement vector, and \( \ddot{u} \) is the nodal acceleration vector.

For a linear system, free vibrations will be harmonic and the displacement vector takes the following form

\[ u = \phi \cos \omega t \]  \hspace{1cm} (4.2)

in which \( \phi \) is the mode shape, \( \omega \) is the natural frequency and \( t \) is time. For any mode of vibration \( r \), the natural frequency \( \omega_r \) and mode shape \( \phi_r \), are the values of \( \omega \) and \( \phi \) that satisfy the eigenvalue equation derived from equation (4.1)

\[ (K - \omega^2 M) \phi = 0 \]  \hspace{1cm} (4.3)

Figures 4.2-4.4 show the first mode shape in the transverse, vertical and longitudinal directions, respectively. The first four mode shapes in each of the three directions are presented in Appendix I. Figure 4.5 shows the first torsional mode. A plan view is given to show displacements in the transverse and longitudinal directions, an elevation view is given to show vertical displacements, and an isometric view is given to show 3-D displacements.

Table 4.1 presents the characteristics of the modes up to a frequency of 3.0 Hz. Some of the longitudinal modes are incidental to the vertical modes of vibration (Modes 7 and 10 in Table 4.1). Some of the longitudinal modes are close to other modes, and it is difficult to decide whether they are pure longitudinal motion or are associated with other vibration. For illustrative purpose, in Appendices I and II global mode 7 (\( f = 1.16 \) Hz) is used for both mode 2 vertical (Figures I.6 and II.6) and mode 2 longitudinal (Figures I.10 and II.10). Also, global mode 10 (\( f = 1.89 \) Hz) is used for both mode 4 vertical (Figures I.8 and II.8) and mode 3 longitudinal (Figures I.11 and II.11).
### TABLE 4.1: Theoretical Natural Frequency

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz) (2)</th>
<th>Period (seconds) (3)</th>
<th>Type and Symmetry</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>.48</td>
<td>2.08</td>
<td>Transverse #1, symmetric</td>
</tr>
<tr>
<td>2</td>
<td>.65</td>
<td>1.53</td>
<td>Vertical #1, symmetric</td>
</tr>
<tr>
<td>3</td>
<td>.85</td>
<td>1.18</td>
<td>Transverse #2, asymmetric</td>
</tr>
<tr>
<td>4</td>
<td>.93</td>
<td>1.08</td>
<td>Transverse #3, symmetric</td>
</tr>
<tr>
<td>5</td>
<td>.99</td>
<td>1.01</td>
<td>Longitudinal #1, symmetric</td>
</tr>
<tr>
<td>6</td>
<td>1.02</td>
<td>.98</td>
<td>Transverse #4, symmetric</td>
</tr>
<tr>
<td>7¹</td>
<td>1.16</td>
<td>.86</td>
<td>Vertical #2, asymmetric (Longitudinal #2, symmetric)</td>
</tr>
<tr>
<td>8</td>
<td>1.43</td>
<td>.70</td>
<td>Vertical #3, symmetric</td>
</tr>
<tr>
<td>9</td>
<td>1.68</td>
<td>.60</td>
<td>Torsional, symmetric</td>
</tr>
<tr>
<td>10¹</td>
<td>1.89</td>
<td>.53</td>
<td>Vertical #4, asymmetric (Longitudinal #3, symmetric)</td>
</tr>
<tr>
<td>11</td>
<td>2.38</td>
<td>.42</td>
<td>Transverse #5, symmetric</td>
</tr>
<tr>
<td>12</td>
<td>2.40</td>
<td>.42</td>
<td>Transverse #6, asymmetric</td>
</tr>
<tr>
<td>13</td>
<td>2.66</td>
<td>.38</td>
<td>Vertical #5, symmetric</td>
</tr>
<tr>
<td>14</td>
<td>2.80</td>
<td>.36</td>
<td>Torsional #2, asymmetric</td>
</tr>
<tr>
<td>15</td>
<td>3.01</td>
<td>.33</td>
<td>Longitudinal #4, asymmetric</td>
</tr>
</tbody>
</table>

¹Some of the longitudinal modes are incidental to the vertical modes of vibration. For illustrative purpose, global modes 7 and 10 are plotted for the corresponding vertical and longitudinal modes in Appendices I and II.
4.1.3 Model Calibration

The model was calibrated by comparing the results of the free vibration analysis (section 4.1.2) with the ambient vibration data from field testing (Chapter 3). The initial free vibration analysis results for vertical and transverse modes agreed closely with field data but the longitudinal vibrational properties did not. The structural model was calibrated by adjusting the axial stiffness of the beam elements which were used to model the deck and stringers.

4.1.4 Model Verification

For the first mode of vibration in the transverse, vertical and longitudinal directions, the experimental frequency distributions are presented, and the experimental mode shapes are compared with the theoretical ones in Figures 4.6-4.8. Appendix II presents the results for the first four modes of vibration in each of the three directions. The mode shapes in Figures 4.6-4.8 are along the east side on the bottom deck (Figure 3.1).

In Figures 4.6-4.8 and in the figures in Appendix II, some of the experimental data points were not presented. For example, for the first mode shape in the vertical direction (Fig. 4.7), data points at locations 1, 13 and 25 were deleted. Figure 4.9 shows these points. The data points are deleted when it is not physically possible for the bridge to behave in such a manner, or when there is some concern about the accuracy of the test data.

It was expected that some experimental data points will not agree with the theoretical ones. The theoretical results for the mode shapes in Figures 4.6-4.8 are generated at the end nodes in the floor beams. On the other hand, due to the limited access to the actual floor beams, all measurements were taken by placing the instruments on the pavement just above the floor stringers (Figure 3.1). Furthermore, the expansion joints in the top and bottom decks were not accounted for in the 3-D model.
4.2 The Approach Spans

The primary concern with the approach spans is the potential for loss of span failure due to excessive longitudinal displacement. Given the general dynamic characteristics of composite slab and beam bridges and the proximity and activity of the study seismic zones, a response spectrum analysis using simple one or two degree of freedom models and conservative modeling techniques can be expected to yield conservative results.

4.2.1 Structural Modeling

Two types of models were used to analyze the longitudinal response of the approach spans; one or two degree of freedom "lollipop" models were used for single or double deck sections respectively (Figure 4.10).

The bents chosen for analysis have fixed bearings (on both decks for double deck bents). The mass of the deck, beams, barriers, and appurtenances from the fixed bearing to the next structural discontinuity plus the appropriate bent mass were lumped at the nodes.

The bent section properties were modeled using the sum of the column section properties. The foundations were modeled using a simplified equivalent cantilever model; the bent section was simply extended half the length of the piles.
Figure 4.1  Global Model of the Brent-Spence Bridge
Figure 4.2  Transverse Mode #1 (f = 0.48 Hz, T = 2.08 sec)
Figure 4.3  Vertical Mode #1 (f = 0.65 Hz, T = 1.53 sec)
Figure 4.4  Longitudinal Mode #1 (f = 0.99 Hz, T = 1.01 sec)
Figure 4.5  Torsional Mode #1 (\(f = 1.68\) Hz, \(T = 0.60\) sec)
Figure 4.6 Frequency Distribution and Mode Shape for Transverse Mode #1
Figure 4.7 Frequency Distribution and Mode Shape for Vertical Mode #1
Figure 4.8 Frequency Distribution and Mode Shape for Longitudinal Mode #1
Figure 4.9 Frequency Distribution and Mode Shape for Vertical Mode #1 including data points 1, 13 and 25
Figure 4.10. Typical One and Two Degree of Freedom Models for the Approach Spans
5. Site Specific Ground Motion

5.1 Introduction

The Brent-Spence bridge is located well outside of any generally accepted seismic zone. The nearest zones to the site are the New Madrid, Wabash Valley, and Anna (Ohio) seismic zones as shown in Figure 5.1. Of these three zones, the New Madrid has historically produced the largest events, and is the best understood. Based on the work of Stauder (1982) and the recently completed PANDA experiment (Chiu et al., 1992), the most active areas of the seismic zone have been shown to consist of a predominantly right-lateral strike-slip segment trending northeast/southwest, and a predominantly reverse dip-slip segment striking northwest/southeast. The two segments are indicated in Figure 7.1 by the letters A and B, respectively.

The New Madrid earthquakes of 1811-1812 caused minor damage to structures in Cincinnati. Newspaper accounts from the Liberty Hall (Dec. 18, 1811; Feb. 12, 1812) and Western Spy (Jan. 25, 1812) and records kept by Drake (1815) and Mansfield (1812) indicate that the Dec. 16, 1811 and Feb. 7, 1812 earthquakes broke off several chimney tops, stopped clocks, and were believed to have caused cracks in brick walls (Street, 1984). Drake (1815) notes that the earthquakes were felt more strongly "...in the valley of the Ohio, than the adjoining uplands." Based on these accounts, Street (1984) assigned Modified Mercalli intensities of VII to Cincinnati for the Dec. 16, 1811 and Feb. 7, 1812 earthquakes. These are the highest Modified Mercalli intensities assigned to the Cincinnati area for any earthquake in the historical record.

Earthquakes in the other two seismic zones, that is the Wabash Valley and Anna seismic zones, are not as well understood as those in the New Madrid seismic zone. Historically, the largest event in the Wabash Valley seismic zone is the 5.5 $m_{b, L_5}$ to 5.8 $m_{b, L_5}$ magnitude event of September 27, 1891 (Street, 1980), where $m_{b, L_5}$ = body wave magnitude, and refers to the 1-sec-period Lg wave magnitude scale proposed by Nuttli (1973).
\[ m_{b,Lg} = 3.30 + 1.66 \log_{10} D + \log_{10} A/T \quad \text{and} \quad 4 \leq D \leq 30 \quad (5.1) \]

and \( Lg = \) higher mode guided surface wave, \( D = \) epicentral distance expressed in degrees, \( A = \) zero-to-peak amplitude, in microns, as read off the seismogram, and \( T = \) period in seconds. Historically the largest event in the Anna seismic zone is the 5.0 \( m_{b,Lg} \) magnitude event of March 7, 1937. Street (1980) assigned a Modified Mercalli intensity \( V \) to Cincinnati for the 1891 event. Westland and Ross (1940) describe the 1937 event as being only moderately felt in Cincinnati, and not causing any damage. Because of the lack of significant earthquake activity in the Anna seismic zone and the modest effects of the 1937 event in the Cincinnati area, the effects of an earthquake from this zone are not considered any further.

Based on the rate of seismicity of these and other seismic zones, Algermissen et al. (1982) derived contour maps of the United States showing peak particle motions. Figures 5.2 and 5.3, taken from Algermissen et al. (1982), show estimates of the maximum horizontal accelerations for the intervals of 50 and 250 years, respectively. Superimposed on the figures is the location of the Brent-Spence bridge. The maximum horizontal accelerations at the site for the two intervals, as read from the figures, are 0.05g-0.06g and 0.11g, respectively. A difficulty with using results from this type of analysis, of course, is the lack of knowledge as to the frequency content and duration of the ground motions. For these reasons, a more preferable method of specifying ground motions at a site is to generate appropriate time histories.

5.2 Time Histories

For the purpose of deriving recommended time-histories for the Brent-Spence Bridge, the stochastic simulation model proposed by Boore (1983) is used.

Adopting the same mathematical representation as O'Connor and Ellingwood (1992), the spectral density of the strong motions derived using the Boore model, \( S_s(\omega) \), can be described by the equation

\[ S_s(\omega) = C M_o S(\omega, \omega_s) P(\omega, \omega_m)(1/R)e^{-w^2/2Q^2} \quad (5.2) \]

where \( C = \) constant related to the source parameters, \( S(\omega, \omega_s) = \) spectral density of the energy released at the source, \( M_o = \) seismic moment (dyne-
cm), \( P(\omega, \omega_m) \) = high-cut filter with cutoff \( \omega_m \), \( R \) = epicentral distance, \( Q \) = specific quality factor which is inversely related to attenuation, \( \beta \) = shear wave velocity (km/s), \( \omega \) = circular frequency, \( \omega_c \) = corner frequency exhibited by the spectral density of the shear waves, and \( \omega_m \) = maximum circular frequency. The constant \( C \) is related to the source parameters by

\[
C = \frac{(RP \times FS \times RF)}{(4\pi \rho \beta^3)} \tag{7.3}
\]

in which \( RP \) = radiation pattern, \( FS \) = amplification due to the free-surface boundary condition, \( RF \) = reduction factor to account for the partitioning of the seismic energy into two horizontal components, \( \beta \) = shear wave velocity, and \( \rho \) = ground density.

The source parameters seismic moment, \( M_o \), and spectral corner frequency, \( f_c = \omega_c/2\pi \), are related by the equation

\[
f_c = 4.9 \times 10^6 \beta (\Delta \sigma/M_o)^{1/3} \tag{5.4}
\]

where \( f_c \) is in Hertz, \( \beta \) is in km/s, \( \Delta \sigma \) = stress drop in bars, and \( M_o \) is in dyne-cm (Brune 1970, 1971).

Time histories for four hypothetical earthquakes, based on the Boore model, were derived for the Brent-Spence bridge. The four events included simulated time histories for a 7.3 \( m_{b, L4} \) (8.5 \( M_s \)) and 6.5 \( m_{b, L4} \) (6.7 \( M_s \)) magnitude event at New Madrid, Missouri, a 6.5 \( m_{b, L4} \) (6.7 \( M_s \)) magnitude event in the Wabash Valley seismic zone near the site of a recent event of some significance, and a local event having a magnitude of 5.25 \( m_{b, L4} \) (4.6 \( M_s \)). The local event is herein defined as being twenty kilometers from the site of the bridge. \( M_s \) = surface wave magnitude, and the relationship the \( m_{b, L4} \) and \( M_s \) magnitudes is discussed in section 5.2.1. The curve between \( m_{b, L4} \) and seismic moment in Torro et al. (1992) was used to calculate the seismic moments of the four events. Other parameters used in deriving the time histories through the use of the Boore model are given in Table 5.1.

Figures 5.4-5.6 show the simulated time-histories (acceleration, velocity, and displacement) for the hypothetical 7.3 \( m_{b, L4} \) (8.5 \( M_s \)) New Madrid event using the values of the parameters given in Table 5.1. Figures 5.7-5.9 illustrate the 0%, 2% and 5% damped response spectra for the same event. The time-histories and response spectra for all four hypothetical events are presented in Appendix III.
TABLE 5.1: Values for Recommended Earthquakes

<table>
<thead>
<tr>
<th>Seismic Zone Event (1)</th>
<th>Epicenter (°N/°W) (2)</th>
<th>$M_{w, Lg}$ (3)</th>
<th>Epicentral Distance (km) (4)</th>
<th>Seismic Moment$^3$ (dyne-cm) (5)</th>
<th>Seismic Moment$^3$ (dyne-cm) (6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Madrid</td>
<td>36.5/89.5</td>
<td>7.3</td>
<td>555</td>
<td>1000.0x10$^{26}$</td>
<td>1400.0x10$^{26}$</td>
</tr>
<tr>
<td>New Madrid</td>
<td>36.5/89.5</td>
<td>6.5</td>
<td>555</td>
<td>5.0x10$^{26}$</td>
<td>9.0x10$^{26}$</td>
</tr>
<tr>
<td>Wabash Valley</td>
<td>33.71/87.95</td>
<td>6.5</td>
<td>340</td>
<td>5.0x10$^{26}$</td>
<td>9.0x10$^{26}$</td>
</tr>
<tr>
<td>(So. Illinois)</td>
<td>Local</td>
<td>not specified</td>
<td>20</td>
<td>0.02x10$^{26}$</td>
<td>0.028x10$^{26}$</td>
</tr>
</tbody>
</table>

$^1$Values used in modeling that are common to all four hypothetical earthquakes are:

density ($\rho$) = 2.7 gm/cm$^3$;
specific quality factor ($Q_s$) = $Q_s f^0.44$, where $Q_s = 550$ (Torro et al., 1992) and $f$ is frequency (Hz);
$RP = 0.63$;
$RF = 0.71$;
$FM = 25$ Hz;
$Stress$ drop $\Delta \sigma = 100$ bars;
$FS = 2$.

$^3$Seismic moment used for calculating the vertical component of the ground motions.

$^4$Seismic moment used for calculating the horizontal component of the ground motions. Different starting points were used in the random numbers generator during the modeling to insure that the two horizontal components were not in phase with one another.

The two New Madrid events in Table 5.1 are designed to be representative of the largest earthquake to have occurred in the seismic zone, and the magnitude of an event that is thought to have a much greater probability of occurring by the year 2035 A.D. in the seismic zone. The epicentral location and magnitude for the largest of the two events in the New Madrid seismic zone, are the same as what Nuttli (1973b) assumed for the magnitude and location of the Feb. 7, 1812 earthquake. The location of the 6.5 $m_{w, Lg}$ event is arbitrarily taken to be near New Madrid since that
places the event at the northern end of the seismic zone and brings it closer to the bridge site than if it were to occur further south.

Johnston and Nava (1985) estimated that there is less than 4% probability that an event such as the great earthquakes of 1811-1812 occurring again by 2035 A.D. In contrast, they estimated in 1985 that an event of \( m_{b,Lg} \geq 6.0 \) has an 86-97% probability of occurring in the New Madrid seismic zone by the year 2035 A.D. Consequently, the ground motions simulated for the 6.5 \( m_{b,Lg} \) are considered to be far more likely to occur than those for the 7.3 \( m_{b,Lg} \) event.

The 6.5 \( m_{b,Lg} \) magnitude Southern Illinois event, given in Table 5.1 for the Wabash Valley seismic zone, is approximately what Nuttli and Herrmann (1978) suggested as being the maximum-magnitude event for the seismic zone. The epicentral location of the event is chosen to be the same as the epicentral location of the 4.9 \( m_{b,Lg} \) magnitude southeastern Illinois event that occurred on June 10, 1987 (Taylor et al., 1989).

Table 5.2 gives the peak particle motions for each component for the four events described by the information in Table 5.1.

### 5.2.1 \( m_{b,Lg} \) and \( M_s \) Magnitudes

The conversion between magnitude scales, typically requires knowledge as to the source parameters of the earthquakes being considered. Nearly all of the earthquakes in the central United States are given in terms of their \( m_{b,Lg} \) magnitudes. The only exceptions are some of the more recent and larger events for which seismic moments \( (M_s) \) have also been determined. \( M_s = \mu S<d> \) where \( M_s \) is the seismic moment, \( \mu \) is the shear strength of the faulted rock, \( S \) is the area of the fault, and \( <d> \) is the average displacement on the fault. The 7.3 \( m_{b,Lg} \) magnitude used in this study for the hypothetical New Madrid earthquake is based on the work of Nuttli (1973b) and Street (1982). These estimates were derived using empirical formulas relating felt areas and the falloff of Modified Mercalli (MM) intensities to the \( m_{b,Lg} \) magnitude scale. To derive a \( M_s \), surface magnitude for the same event requires information or assumptions about the stress drop and seismic moment of the earthquake. Since there were no seismographs in operation in 1812 to measure the February 7th earthquake, any \( M_s \) magnitude estimated for the event is of necessity based on assumptions concerning its source parameters.
<table>
<thead>
<tr>
<th>Seismic Zone Event</th>
<th>Particle Motion(^1)</th>
<th>Component</th>
<th>Longitudinal (3)</th>
<th>Vertical (4)</th>
<th>Horizontal (5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Madrid</td>
<td>A (cm/s/a)</td>
<td></td>
<td>240</td>
<td>223</td>
<td>269</td>
</tr>
<tr>
<td></td>
<td>V (cm/s)</td>
<td></td>
<td>6.54</td>
<td>7.11</td>
<td>7.38</td>
</tr>
<tr>
<td></td>
<td>D (cm)</td>
<td></td>
<td>1.16</td>
<td>1.20</td>
<td>1.75</td>
</tr>
<tr>
<td>New Madrid</td>
<td>A (cm/s/a)</td>
<td></td>
<td>58</td>
<td>44</td>
<td>59</td>
</tr>
<tr>
<td></td>
<td>V (cm/s)</td>
<td></td>
<td>2.21</td>
<td>1.72</td>
<td>3.09</td>
</tr>
<tr>
<td></td>
<td>D (cm)</td>
<td></td>
<td>0.50</td>
<td>0.23</td>
<td>0.75</td>
</tr>
<tr>
<td>Wabaah Valley</td>
<td>A (cm/s/a)</td>
<td></td>
<td>34</td>
<td>24</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>V (cm/s)</td>
<td></td>
<td>0.99</td>
<td>0.82</td>
<td>1.07</td>
</tr>
<tr>
<td></td>
<td>D (cm)</td>
<td></td>
<td>0.19</td>
<td>0.10</td>
<td>0.23</td>
</tr>
<tr>
<td>Locai</td>
<td>A (cm/s/a)</td>
<td></td>
<td>124</td>
<td>123</td>
<td>127</td>
</tr>
<tr>
<td></td>
<td>V (cm/s)</td>
<td></td>
<td>3.13</td>
<td>2.79</td>
<td>2.59</td>
</tr>
<tr>
<td></td>
<td>D (cm)</td>
<td></td>
<td>0.31</td>
<td>0.17</td>
<td>0.22</td>
</tr>
</tbody>
</table>

\(^1\)A = Acceleration; V = Velocity; D = Displacement.

As pointed out by Johnston and Shedlock (1992), the \(M_s\) magnitude of the February 7, 1812 New Madrid event has been estimated by various authors ranging from a low of 5.5-6.9 to a high 8.4-8.8. In this study, we have assumed a \(M_s\) value of 8.5 for the hypothetical 7.3 \(m_{a,\text{le}}\) magnitude earthquake at New Madrid. The 8.5 \(M_s\) magnitude is consistent with our assumed stress drop of 100 bars used in the modeling and the 1x10\(^{20}\) dyne-cm seismic moment derived from the \(m_{b,\text{le}}\) - \(M_s\) scaling suggested by Torro et al. (1992) for the New Madrid seismic zone. Based on the stress drop, the scaling suggested by Torro et al. (1992), and the Brune (1970, 1971) source model to relate these parameters, the following approximate \(m_{b,\text{le}}\) - \(M_s\) conversions are used in this study:

Seismic Analysis

53
<table>
<thead>
<tr>
<th>$m_{b, L_g}$</th>
<th>$M_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.3</td>
<td>8.5</td>
</tr>
<tr>
<td>6.5</td>
<td>7.4</td>
</tr>
<tr>
<td>6.0</td>
<td>5.7</td>
</tr>
<tr>
<td>5.25</td>
<td>4.6</td>
</tr>
</tbody>
</table>

5.3 Recommended Ground Motions

Based on the above analysis, it is recommended that Brent-Spence bridge be evaluated for the 6.5 $m_{b, L_g}$ (7.4 $M_s$) magnitude earthquake at New Madrid, Missouri, and the So. Illinois Local earthquakes in Table 5.1. The larger event at New Madrid, Missouri, the 7.3 $m_{b, L_g}$ (8.5 $M_s$) earthquake, has a very low probability of occurring and is, therefore, recommended only for consideration as the maximum credible earthquake. The time-histories, damped response spectra, and peak particle motions given in the above analysis are intended to be representative of the free-field ground motions at the top of bedrock at the site.
Figure 5.1. Seismic Zones (modified after Nuttli and Herrmann, 1978)
Estimates of Maximum Horizontal Accelerations for 250 Years Interval From 1982 - 2232 (Source: Algermissen et al., 1982)

Seismic Analysis
Figure 5.4. Acceleration Time History for the 7.3 m/s (24 ft/s) New Madrid Event (Note: 1 cm = 0.39 in)

Brent-Spence Bridge
Figure 5.5. Velocity Time History for the 7.3 m$_{b,ls}$ (8.5 M$_{s}$) New Madrid Event (Note: 1 cm = 0.394 in)
Figure 5.6. Displacement Time History for the 7.3 $m_{b,lg}$ (8.5 $M_s$) New Madrid Event
(Note: 1 cm = 0.394 in)
Figure 5.7. Transverse Earthquake Spectrum for the 7.3 $m_{w,s}$ (8.8 $M_s$) New Madrid Event (Note: 1 in = 2.54 cm)
Figure 5.8. Vertical Earthquake Spectrum for the 7.3 m_a (8.8 M_s) New Madrid Event (Note: 1 in = 2.54 cm)
Figure 5.9. Longitudinal Earthquake Spectrum for the 7.3 $m_{wo}$ (8.8 $M_s$) New Madrid Event (Note: 1 in = 2.54 cm)
6. Seismic Response Analysis

The seismic response of the Brent-Spence bridge was evaluated for the 5.25 $m_{b,s}$ (4.6 $M_s$) local event, the 6.5 $m_{b,s}$ (7.4 $M_s$) Wabash Valley event, the 6.5 $m_{b,s}$ (7.4 $M_s$) New Madrid event, and the 7.3 $m_{b,s}$ (8.5 $M_s$) New Madrid event (see Table 5.2 and section 5.3). The New Madrid 7.3 $m_{b,s}$ event is considered to be the maximum credible earthquake.

6.1 Forced Vibration

For forced vibration, the equations of motion are

$$ Ku + C\ddot{u} + M\ddot{u} = f $$  \hspace{1cm} (6.1)

where $C$ is the global damping matrix, $K$ is the global stiffness matrix, $M$ is the global mass matrix, $f$ is the forcing function, $\ddot{u}$ is the nodal displacement vector, $\dot{u}$ is the nodal velocity vector, and $\ddot{u}$ is the nodal acceleration vector.

6.2 Damping

For the forced vibration analysis of the main bridge, Rayleigh damping was applied. The damping matrix is of the form

$$ C = \alpha M + \beta K $$  \hspace{1cm} (6.2)

where $\alpha$ and $\beta$ are arbitrary proportionality factors.

The damping coefficient exhibited by a structure during a seismic event is difficult to predict. A range of $2% \leq \xi \leq 5\%$ is adopted in this study, and the results are presented for the limiting values of $\xi = 2\%$ and $\xi = 5\%$. In Eq. 6.2, $\alpha$ and $\beta$ are determined using $\xi = 2\%$ and $\xi = 5\%$ at the two specific frequencies of $f = 0.5$ Hz ($T = 2$ sec.) and $f = 4$ Hz ($T = 0.25$ sec.), which is the range of primary interest.
6.3 The Main Bridge

Direct time integration (i.e. Time History) was used to perform the seismic response analysis of the main bridge due to the importance of the bridge and the lack of seismic considerations in its design.

The bridge was subjected simultaneously to all three components of each event (Longitudinal, Transverse and Vertical). To provide conservative stress and displacement results, the bridge was subjected twice to each event. In the first application, the longitudinal component of the event was placed along the transverse direction of the bridge (x-direction) and the transverse component of the event was placed along the longitudinal direction of the bridge (z-direction). In the second application, the longitudinal component of the event was placed along the longitudinal direction of the bridge while the transverse one was placed along the transverse direction of the bridge. The maximum displacements, forces, and stresses from the two are presented in the following sections.

The foundations were modeled using both the linear uncoupled base springs foundation model (Lam, Martin, and Imbsen 1990), and the rigid foundation condition. The two models provide conservatively bounded stress and displacement results. Section 4.1.1 provides a more detailed description of the springs-foundation model.

Of the four seismic events, the 7.3 \( m_{b,Le} \) (8.5 \( M_{w} \)) New Madrid event is critical. The results for the main bridge are presented for this event only.

6.3.1 The Superstructure

The critical members in the truss are in the tower over the main piers and at the hinges (Figure 6.1).

Figure 6.2 presents the maximum stresses resulting from dead load (DL) and earthquake (or seismic) loads (EQ) for members over Pier II and for the vertical hanger at the hinge next to Pier II. These maximum stresses occurred at 2% damping when soil structure interaction was neglected and the maximum relative transverse displacement between the upper and lower decks was considered.

The maximum stresses from the combined loading in the eyebars [32.0 ksi (221 MPa)], at the base of the tower [33.3 ksi 9229 MPa]), at the
base of the portal [34.7 ksi (239 MPa)] (Figure 6.2a), and in the hanger [29.6 ksi (204 MPa)] (Figure 6.2b), are all well below the yield stress of 50 ksi (345 MPa).

Tables 6.1 and 6.2 present the seismic stresses at the bottom of the tower over Pier II for rigid and flexible foundations and for the two damping coefficients. In Table 6.1, the maximum relative transverse displacement between the upper and lower decks was considered. In Table 6.2, the maximum relative vertical displacement between the upper and lower decks was considered. The critical stresses occurred at 2% damping when soil-structure interaction was neglected in Table 6.1.

Maximum stresses resulting from the combined loading DL (dead load) + EQ (earthquake) were generated for a large number of truss members. In all cases, the combined stresses were below the yield stress. Consequently, the bridge components remain elastic under the maximum credible seismic event and linear elastic analysis is valid.

6.3.2 The Substructure

The time history for the relative longitudinal displacement between the top and bottom of Pier II (Figure 6.1) is presented in Figure 6.3. The maximum relative displacement of 0.9 in (2.2 cm) occurs 17 seconds into the seismic event when soil-structure interaction is neglected and $\xi = 0.02$ (Figure 6.3a). For Pier I, the time history is presented in Figure 6.4, and the maximum relative longitudinal displacement of 0.34 in (0.85 cm) occurs 11 seconds into the event when soil-structure interaction is neglected and $\xi = 0.02$ (Figure 6.4a). These displacements are small, and second order effects (i.e. P-Δ) in the piers are minimal.

The time history for the relative longitudinal displacement between the top of Pier II and the top of Pier III (Figure 6.1) is presented in Figure 6.5. The maximum displacement of 0.1 in (0.3 cm) occurs 21 seconds into the event when soil-structure interaction is considered and $\xi = 0.02$ (Figure 6.5b). For the relative longitudinal displacement between the top of Pier I and the top of Pier II (Figure 6.1), the time history is presented in Figure 6.6, and the maximum relative displacement of 1.3 in (3.2 cm) occurs 21 seconds into the event when soil-structure interaction is neglected and $\xi = 0.02$ (Figure 6.6b). These relative displacements are small and span-loss type failure in the main bridge is not expected.

The stresses in the piers resulting from seismic loads only are very
shapes and the associated natural frequencies. These vibration properties were subsequently used as the basis for calibrating the analytical computer model for seismic response analysis.

Analytical Modeling

Applied dynamic loads and responses were regarded as disturbances of the current dead load configuration of the bridge.

A three dimensional finite element model of the main bridge was used for free vibration and seismic response analyses. The model was calibrated by comparing the free vibration analysis results with ambient vibration properties from field testing. Once calibrated, the model was then used for seismic response analysis.

The approach spans were modeled using simple one or two degree of freedom systems. The mass was concentrated over the piers at points of longitudinal fixity (i.e. lollipop model). Seismic response was analyzed in the longitudinal direction only.

Site Specific Ground Motion

The site specific ground motion scenarios were developed to represent probable earthquakes that may occur in the New Madrid, Wabash Valley, and Anna seismic zones. A local seismic event, defined as being 12.5 miles (20 km) from the site of the bridge, was also considered. Time histories and response spectra were generated for a number of events. The time histories were then used in the seismic analysis of the main bridge while the response spectra were used for analyzing the approach spans.

Seismic Response Analysis

The three dimensional model of the main bridge was subjected to the time histories of the aforementioned earthquakes to determine maximum displacements, forces and stresses. Maximum relative displacements were determined at a number of locations including the top and bottom of the piers, and between the upper and lower deck levels. The maximum forces and stresses were determined at a number of locations including the piers, towers, and anchor bolts at fixed bearings.
small in all piers. Table 6.3 presents the maximum bending and shear stresses at the bottom of Pier II. The maximum bending stress of 0.7 ksi (5.0 MPa) occurs when \( \xi = 0.02 \) and soil-structure interaction is neglected. The maximum shear stress is below 0.1 ksi (0.2 MPa).

### 6.3.3 The Bearings

The shear stresses in the anchor bolts at the fixed bearings over Piers II and III (Figure 6.1), are presented in Table 6.4. The maximum shear stress \( \tau \) of 21.8 ksi (150.3 MPa) occurs when \( \xi = 0.02 \) and soil-structure interaction is considered, and is well below the bolts' yield stress of 50 ksi (345 MPa). The ratio of the maximum seismic pullout force to dead load force, \( \rho_{SG} \), over the fixed bearings is less than 0.4 which can be expressed as a factor of safety of 2.5 against overturning. Consequently, pullout of the anchor bolts is not expected.

For the expansion bearings over Piers I and IV (Figure 6.1), the maximum relative movement between the top and bottom of the bearings over Pier I for example is equal to the maximum relative longitudinal movement between the top of Pier I and the top of Pier II. This maximum relative movement is 1.3 in (3.2 cm) [refer to section 6.3.2 and Figure 6.6]. When a maximum thermal movement of 3.2 in (8.1 cm) [for a 90 °F (32 °C) maximum differential temperature], and an allowable displacement of 24 in (61 cm) are considered, the Capacity/Demand (C/D; refer to section 6.4) displacement ratio over pier I or IV is 16.0. Consequently, loss of span over Pier I or IV is not expected.

### 6.4 The Approach Spans

For the approach spans (Figure 6.7), the primary concern is the potential for loss of span due to excessive longitudinal displacement at the expansion joints. Typical expansion joints on the Covington approach are shown in Figures 6.8 and 6.9.

The FHWA Seismic Design and Retrofit Manual for Highway Bridges (Buckle et.al. 1987) clarifies the use of the minimum support length equations in the seismic evaluation process. Capacity/Demand (C/D) ratios are suggested as a means of easily identifying vulnerable components. A C/D ratio of less than 1.0 indicates that failure may occur under the design
C/D ratio of less than 1.0 indicates that failure may occur under the design earthquake and retrofit may be appropriate.

Displacement C/D ratios should be calculated by two methods (Buckle et al. 1987): 1) using the AASHTO minimum bearing support length equations and 2) using maximum relative displacements from seismic analysis. The most conservative of these is used to evaluate the support. If displacement limiting devices, such as seismic restrainers, have been installed only, seismic analysis results need be considered (Method 2).

6.4.1 AASHTO Minimum Support Length

In major earthquakes, the loss of support at bearings due to excessive relative displacement has been responsible for numerous loss of span bridge failures. Division I-A Section 4.9 of the AASHTO Bridge Specifications sets out minimum bearing support length requirements for seismic design.

While the AASHTO Division I-A provisions are not directly applicable to the main bridge (due to the span length and bridge type), they do apply to the approach spans (AASHTO 15th ed., 1992). According to the AASHTO seismic provisions, the approach spans are classified as Seismic Performance Category A (SPC A); however, if the peak acceleration from the site specific ground motion is considered, they may be classified as SPC B. In either case, the appropriate minimum bearing support length for expansion bearings at abutments, piers, and hinges is specified in AASHTO Equations 4-3A and 4-3B.

The AASHTO minimum bearing support lengths for the approach spans and the ensuring displacement C/D ratios are listed in Table 6.5 (Table 6.5a is in metric units while Table 6.5b is in English units). A displacement ratio C/D < 1.0 (column 6 in Table 6.5) indicates that the AASHTO minimum bearing support length is not satisfied and the bearing support is susceptible to loss of span failure. Two (2) out of the twelve (12) evaluated expansion bearings are identified as vulnerable on the upper deck on the Covington approach (Table 6.5 and Figure 6.7). Locations having a C/D < 0.5 are particularly vulnerable and should be retrofitted (Buckle et. al. 1987).
6.4.2 Response Spectra Analysis

The response spectra for the four seismic events are presented in Appendix III. Trilinear envelopes of the spectra in Appendix III were generated for the "design" spectra. The influence of the vertical seismic component on longitudinal displacements is negligible, and is not considered in the analysis.

In determining the maximum displacement for the single degree of freedom model (refer to section 4.2.1 and Figure 4.10), the pier was first subjected to the transverse component of the event placed in the longitudinal direction of the bridge, and the maximum longitudinal displacement $D_T$ was recorded. Then the longitudinal component of the event was placed in the longitudinal direction of the bridge, and the maximum longitudinal displacement, $D_L$, was recorded. The resultant maximum longitudinal displacement, $D_{\text{max}}$, at pier $i$, is estimated as

$$D_{i,\text{max}} = \sqrt{D^2_{i,T} + D^2_{i,L}}.$$  

At an expansion joint between piers $i$ and $j$, the maximum relative displacement $\Delta_{eq}(d)$ is taken as the sum of the absolute values of the maximum longitudinal displacements at piers $i$ and $j$, $\Delta_{eq}(d) = |D_{i,\text{max}}| + |D_{j,\text{max}}|$. 

In determining the maximum displacements for the two degree of freedom model (refer to section 4.2.1 and Figure 4.10), the pier was first subjected to the transverse component of the event placed in the longitudinal direction of the bridge, and the maximum longitudinal displacement at the upper deck level, $D_T^u$, and the lower deck level, $D_T^l$, were recorded. Then, the longitudinal component of the event was placed in the longitudinal direction of the bridge, and the maximum displacements, $D_L^u$ and $D_L^l$, were recorded. The resultant maximum longitudinal displacement, $D_{i,\text{max}}^u$, at pier $i$ and at the upper deck, is taken as

$$D_{i,\text{max}}^u = \sqrt{(|D_T^u| + |D_L^u|)^2 + (|D_T^l| + |D_L^l|)^2}.$$ 

The subscripts 1 and 2 denote the mode of vibration. At an expansion joint on the upper deck level, between piers $i$ and $j$, the maximum relative displacement, $\Delta_{eq}(d)$, is taken as the sum of the absolute values of the maximum longitudinal displacements at piers $i$ and $j$, $\Delta_{eq}(d) = |D_{i,\text{max}}^u| + |D_{j,\text{max}}^u|$. The process for determining the maximum relative displacement at an expansion joint on the lower deck level is similar to the one for the upper deck level.
Table 6.6 gives the relative displacements at the expansion joints for the four seismic events. The most critical one is the $7.3 \text{ m}_{b, L_s}$ (8.5 M$_s$) New Madrid event, and the relative displacements range from 1.7 in (4.3 cm) to 7.4 in (18.8 cm) for 2% damping, and from 1.6 in (4.1 cm) to 6.2 in (15.7 cm) for 5% damping.

The Capacity/Demand ($C/D = (\Delta_c(c) - \Delta_c(d))/\Delta_{eq}(d)$) is determined in Table 6.7 (Table 6.7a is in metric while Table 6.7b is in English units) following the method based on the maximum relative displacements due to earthquake loading (Buckle et. al. 1987). $\Delta_c(c)$ = allowable movement of the expansion joint or bearing; $\Delta_c(d)$ = maximum possible movement resulting from temperature, shrinkage and creep shortening; and $\Delta_{eq}(d)$ = maximum relative displacement due to earthquake loading. In determining $\Delta_c(c)$ for the rocker bearing, the unreinforced cover concrete is excluded. For bearing plates, $\Delta_c(c)$ was determined following the conservative assumption that the top plate is allowed to fall off the bottom plate right at failure. For $\Delta_c(d)$, the maximum differential temperature was taken to be 90°F (32°C), and shrinkage and creep were not considered since the girders are steel girders.

Comparison between Tables 6.5 and 6.7 indicates that the AASHTO method leads to lesser C/D displacement ratios for both the 2% and 5% damping coefficients. Consequently, The AASHTO method will govern at all locations.
Figure 6.1  Plan and Elevation of Brent-Spence Bridge
Figure 6.2 Maximum Stresses Resulting From Dead Load (DL) and Earthquake load (EQ) for Members Over Pier II and for the Vertical Link at the Hinge Next to Pier II ($\xi = 0.02$)
Figure 6.3 Time History of Relative Longitudinal Displacement Between Top and Bottom of Pier II (Figure 6.1): (a) Excluding Soil-Structure Interaction; and (b) Including Soil-Structure Interaction

Seismic Analysis
Figure 6.4 Time History of Relative Longitudinal Displacement Between Top and Bottom of Pier I (Figure 6.1): (a) Excluding Soil-Structure Interaction; and (b) Including Soil-Structure Interaction
Figure 6.5 Time History of Relative Longitudinal Displacement Between Top of Pier II and Top of Pier III (Figure 6.1): (a) Excluding Soil-Structure Interaction; and (b) Including Soil-Structure Interaction
Figure 6.6 Time History of Relative Longitudinal Displacement Between Top of Pier I and Top of Pier II (Figure 6.1): (a) Excluding Soil-Structure Interaction; and (b) Including Soil-Structure Interaction
Figure 6.8  Upper Deck Expansion Joints at Pier 1 thru Pier 9
Figure 6.9  Typical Rocker Bearing at Expansion Joints
Table 6.1: Maximum Seismic Forces and Stresses at Bottom of Steel Tower Over Pier II Resulting From the 7.3 m$_{114}$ (8.5 M$_{w}$) New Madrid Event and the Maximum Relative Transverse Displacement Between the Upper and Lower Decks (Excluding Forces and Stresses Resulting From Dead Loads)

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Units</th>
<th>Excluding Soil-Structure Interaction</th>
<th>Including Soil-Structure Interaction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\xi = 0.05$</td>
<td>$\xi = 0.02$</td>
</tr>
<tr>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
</tr>
<tr>
<td>$F_x$</td>
<td>kN (kip)</td>
<td>259 (193)</td>
<td>1,335 (300)</td>
</tr>
<tr>
<td>$F_y$</td>
<td>kN (kip)</td>
<td>4,121 (926)</td>
<td>4,779 (1,074)</td>
</tr>
<tr>
<td>$F_z$</td>
<td>kN (kip)</td>
<td>40 (9)</td>
<td>53 (12)</td>
</tr>
<tr>
<td>$M_x$</td>
<td>kN.m (kip.ft)</td>
<td>148 (109)</td>
<td>199 (147)</td>
</tr>
<tr>
<td>$M_y$</td>
<td>kN.m (kip.ft)</td>
<td>36 (27)</td>
<td>194 (143)</td>
</tr>
<tr>
<td>$M_z$</td>
<td>kN.m (kip.ft)</td>
<td>2,688 (1,982)</td>
<td>4,300 (3,171)</td>
</tr>
<tr>
<td>$\sigma^1$</td>
<td>MPa (ksi)</td>
<td>77.2 (11.2)</td>
<td>112.4 (16.3)</td>
</tr>
<tr>
<td>$\tau^2$</td>
<td>MPa (ksi)</td>
<td>4.8 (0.7)</td>
<td>7.4 (1.1)</td>
</tr>
</tbody>
</table>

$\sigma^1 = \frac{F_x}{A} + \frac{M_x}{S_x} + \frac{M_z}{S_z}$

$\tau^2 = \sqrt{\frac{F_y^2 + F_z^2}{A} + \frac{M_y}{J_y}}$
Table 6.2: Maximum Seismic Forces and Stresses at Bottom of Steel Tower Over Pier II Resulting From the 7.3 m_{EL} (8.5 M.) New Madrid Event and the Maximum Relative Longitudinal Displacement Between the Upper and Lower Decks (Excluding Forces and Stresses Resulting From Dead Loads)

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Unit</th>
<th>Excluding Soil-Structure Interaction</th>
<th>Including Soil-Structure Interaction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( \xi = 0.05 )</td>
<td>( \xi = 0.02 )</td>
</tr>
<tr>
<td>(1)</td>
<td>(2)</td>
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<td>(4)</td>
</tr>
<tr>
<td>( F_x )</td>
<td>kN</td>
<td>436</td>
<td>418</td>
</tr>
<tr>
<td></td>
<td>(kip)</td>
<td>(98)</td>
<td>(94)</td>
</tr>
<tr>
<td>( F_y )</td>
<td>kN</td>
<td>2,109</td>
<td>2,973</td>
</tr>
<tr>
<td></td>
<td>(kip)</td>
<td>(474)</td>
<td>(668)</td>
</tr>
<tr>
<td>( F_z )</td>
<td>kN</td>
<td>276</td>
<td>271</td>
</tr>
<tr>
<td></td>
<td>(kip)</td>
<td>(62)</td>
<td>(61)</td>
</tr>
<tr>
<td>( M_x )</td>
<td>kN.m</td>
<td>822</td>
<td>1,012</td>
</tr>
<tr>
<td></td>
<td>(kip.ft)</td>
<td>(606)</td>
<td>(746)</td>
</tr>
<tr>
<td>( M_y )</td>
<td>kN.m</td>
<td>199</td>
<td>260</td>
</tr>
<tr>
<td></td>
<td>(kip.ft)</td>
<td>(147)</td>
<td>(192)</td>
</tr>
<tr>
<td>( M_z )</td>
<td>kN.m</td>
<td>1,335</td>
<td>1,252</td>
</tr>
<tr>
<td></td>
<td>(kip.ft)</td>
<td>(999)</td>
<td>(923)</td>
</tr>
<tr>
<td>( \sigma_1 )</td>
<td>MPa</td>
<td>64.8</td>
<td>74.5</td>
</tr>
<tr>
<td></td>
<td>(ksi)</td>
<td>(9.4)</td>
<td>(10.8)</td>
</tr>
<tr>
<td>( \tau_2 )</td>
<td>MPa</td>
<td>2.9</td>
<td>2.8</td>
</tr>
<tr>
<td></td>
<td>(ksi)</td>
<td>(0.4)</td>
<td>(0.4)</td>
</tr>
</tbody>
</table>

\[ \sigma_1 = \frac{F_y}{A} + \frac{M_x}{S_x} + \frac{M_z}{S_z} \]

\[ \tau_2 = \frac{\sqrt{F_x^2 + F_y^2}}{A} + \frac{M_y c}{J_y} \]
Table 6.3: Maximum Seismic Forces and Stresses at Bottom of Pier II Resulting From the 7.3 m\(_{\text{eq}}\) (8.5 M\(_{\text{w}}\)) New Madrid Event (Excluding Forces and Stresses Resulting From Dead Loads)

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Units</th>
<th>Excluding Soil-Structure Interaction</th>
<th>Including Soil-Structure Interaction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(\xi = 0.05)</td>
<td>(\xi = 0.02)</td>
</tr>
<tr>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
</tr>
<tr>
<td>(F_x)</td>
<td>kN (kip)</td>
<td>1,344 (302)</td>
<td>240 (54)</td>
</tr>
<tr>
<td>(F_y)</td>
<td>kN (kip)</td>
<td>2,812 (632)</td>
<td>4,254 (956)</td>
</tr>
<tr>
<td>(F_z)</td>
<td>kN (kip)</td>
<td>9,256 (2,080)</td>
<td>1,0782 (2,422)</td>
</tr>
<tr>
<td>(M_x)</td>
<td>kN.m (kip.ft)</td>
<td>180,571 (133,129)</td>
<td>209,552 (154,615)</td>
</tr>
<tr>
<td>(M_y)</td>
<td>kN.m (kip.ft)</td>
<td>14,493 (10,688)</td>
<td>17,459 (12,875)</td>
</tr>
<tr>
<td>(M_z)</td>
<td>kN.m (kip.ft)</td>
<td>35,156 (25,933)</td>
<td>26,814 (19,774)</td>
</tr>
<tr>
<td>(\sigma^1)</td>
<td>MPa (ksi)</td>
<td>4.0 (0.6)</td>
<td>5.0 (0.7)</td>
</tr>
<tr>
<td>(\tau^2)</td>
<td>MPa (ksi)</td>
<td>0.2 (0.1)</td>
<td>0.2 (0.1)</td>
</tr>
</tbody>
</table>

\[\begin{align*}
\sigma^1 &= \frac{F_y}{A} + \frac{M_x}{S_x} + \frac{M_z}{S_z} \\
\tau^2 &= \frac{\sqrt{F_x^2 + F_z^2}}{A} + \frac{M_y}{J_y}
\end{align*}\]

Brent-Spence Bridge 82
Table 6.4: Maximum Seismic Forces at Fixed Bearings Over Pier II and Corresponding Shear Stresses in Anchor Bolts Resulting From the 7.3 m_s (8.5 M) New Madrid Event (Excluding Forces and Stresses Resulting From Dead Loads)

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Units</th>
<th>Excluding Soil-Structure Interaction</th>
<th>Including Soil-Structure Interaction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
</tr>
<tr>
<td>F_x</td>
<td>kN</td>
<td></td>
<td>3,711</td>
</tr>
<tr>
<td></td>
<td>(kip)</td>
<td></td>
<td>(834)</td>
</tr>
<tr>
<td>F_y</td>
<td>kN</td>
<td></td>
<td>5,576</td>
</tr>
<tr>
<td></td>
<td>(kip)</td>
<td></td>
<td>(1,253)</td>
</tr>
<tr>
<td>F_z</td>
<td>kN</td>
<td></td>
<td>2,448</td>
</tr>
<tr>
<td></td>
<td>(kip)</td>
<td></td>
<td>(550)</td>
</tr>
<tr>
<td>M_x</td>
<td>kN.m</td>
<td></td>
<td>6,785</td>
</tr>
<tr>
<td></td>
<td>(kip.ft)</td>
<td></td>
<td>(5,004)</td>
</tr>
<tr>
<td>M_y</td>
<td>kN.m</td>
<td></td>
<td>57,992</td>
</tr>
<tr>
<td></td>
<td>(kip.ft)</td>
<td></td>
<td>(42,767)</td>
</tr>
<tr>
<td>(\tau^1)</td>
<td>MPa</td>
<td></td>
<td>122.0</td>
</tr>
<tr>
<td></td>
<td>(ksi)</td>
<td></td>
<td>(17.7)</td>
</tr>
<tr>
<td>(\rho_{sg}^2)</td>
<td></td>
<td></td>
<td>0.26</td>
</tr>
</tbody>
</table>

\[
\tau = \sqrt{\frac{(M_y^2 + F_z^2)}{D_b}} + F_x^2
\]

where \(\tau\) = shear stress in anchor bolts, and \(D_b\) = distance between the two fixed bearings over pier II.

\(\rho_{sg}^2\) = ratio of maximum pullout seismic force to dead load force over fixed bearing.
### TABLE 6.5a: AASHTO Design Displacements\(^1\) (Metric Units)

<table>
<thead>
<tr>
<th>Location(^3)</th>
<th>Location (1)</th>
<th>L (m)</th>
<th>H (m)</th>
<th>N(d) (mm)</th>
<th>N(c) (mm)</th>
<th>C/D(^1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Deck</td>
<td>Pier 4 Span 5</td>
<td>17</td>
<td>17</td>
<td>345</td>
<td>152</td>
<td>0.44</td>
</tr>
<tr>
<td></td>
<td>Pier 8 Span 9</td>
<td>14</td>
<td>13</td>
<td>310</td>
<td>152</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 12</td>
<td>69</td>
<td>11</td>
<td>388</td>
<td>381</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 13</td>
<td>59</td>
<td>11</td>
<td>372</td>
<td>381</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 15</td>
<td>59</td>
<td>9</td>
<td>361</td>
<td>381</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 16</td>
<td>59</td>
<td>9</td>
<td>361</td>
<td>381</td>
<td>1.06</td>
</tr>
<tr>
<td>Covington Approach</td>
<td>Pier 4 Span 5</td>
<td>66</td>
<td>11</td>
<td>387</td>
<td>381</td>
<td>0.98</td>
</tr>
<tr>
<td>Lower Deck</td>
<td>Pier 8 Span 9</td>
<td>91</td>
<td>8</td>
<td>405</td>
<td>381</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 12</td>
<td>92</td>
<td>5</td>
<td>390</td>
<td>381</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 13</td>
<td>60</td>
<td>5</td>
<td>333</td>
<td>381</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 15</td>
<td>60</td>
<td>5</td>
<td>339</td>
<td>381</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 16</td>
<td>59</td>
<td>5</td>
<td>339</td>
<td>381</td>
<td>1.12</td>
</tr>
</tbody>
</table>

\(^1\)Design Displacements, AASHTO Div. I-A Sec. 4.9.1

Seismic Performance Category A or B
Minimum Support Length N(d) = 200 + 1.67L + 6.67H
Capacity/Demand Ratio, C/D = N(c)/N(d)

\(^3\)Expansion Bearing Seat Location

Note: See Table 6.5b for English units
# TABLE 6.5b: AASHTO Design Displacements

*(English Units)*

<table>
<thead>
<tr>
<th>Location²</th>
<th>Location</th>
<th>L (ft)</th>
<th>H (ft)</th>
<th>N(d) (in)</th>
<th>N(c) (in)</th>
<th>C/D¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td></td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
<td>(5)</td>
<td>(6)</td>
</tr>
<tr>
<td>Upper Deck</td>
<td>Pier 4 Span 5</td>
<td>57</td>
<td>56</td>
<td>14</td>
<td>6</td>
<td>0.44</td>
</tr>
<tr>
<td></td>
<td>Pier 8 Span 9</td>
<td>45</td>
<td>44</td>
<td>12</td>
<td>6</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 12</td>
<td>225</td>
<td>35</td>
<td>15</td>
<td>15</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 13</td>
<td>195</td>
<td>35</td>
<td>15</td>
<td>15</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 15</td>
<td>195</td>
<td>29</td>
<td>14</td>
<td>15</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 16</td>
<td>194</td>
<td>29</td>
<td>14</td>
<td>15</td>
<td>1.06</td>
</tr>
<tr>
<td>Covington Approach</td>
<td>Pier 4 Span 5</td>
<td>216</td>
<td>37</td>
<td>15</td>
<td>15</td>
<td>0.98</td>
</tr>
<tr>
<td>Lower Deck</td>
<td>Pier 8 Span 9</td>
<td>299</td>
<td>25</td>
<td>16</td>
<td>15</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 12</td>
<td>301</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 13</td>
<td>196</td>
<td>15</td>
<td>13</td>
<td>15</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 15</td>
<td>196</td>
<td>18</td>
<td>13</td>
<td>15</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 16</td>
<td>195</td>
<td>18</td>
<td>13</td>
<td>15</td>
<td>1.12</td>
</tr>
</tbody>
</table>

¹Design Displacements, AASHTO Div. I-A Sec. 4.9.1

Seismic Performance Category A or B

Minimum Support Length $N(d) = 8 + 0.02L + 0.08H$

Capacity/Demand Ratio, $C/D = N(c)/N(d)$

²Expansion Bearing Seat Location

Note: See Table 6.5a for metric units
Table 6.6a: Elastic Response Spectra Displacements (Metric Units)

<table>
<thead>
<tr>
<th>Location</th>
<th>Damping Coefficient</th>
<th>Response Spectra Relative Displacement (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ξ</td>
<td>New Madrid 7.3 m_{max} (8.5 M_s)</td>
</tr>
<tr>
<td>Pier 4 Span 5</td>
<td>0.02</td>
<td>9.3</td>
</tr>
<tr>
<td></td>
<td>0.05</td>
<td>6.0</td>
</tr>
<tr>
<td>Pier 8 Span 9</td>
<td>0.02</td>
<td>9.0</td>
</tr>
<tr>
<td></td>
<td>0.05</td>
<td>6.0</td>
</tr>
</tbody>
</table>

1 Expansion Bearing Seat Location

Note: See Table 6.6b for English units
Table 6.6b: Elastic Response Spectra Displacements (English Units)

<table>
<thead>
<tr>
<th>Location¹</th>
<th>Damping Coefficient</th>
<th>Response Spectra Relative Displacement (in)</th>
</tr>
</thead>
</table>
|           | ε                    | New Madrid 6.5 m
|           |                      | 7.3 m
de (6.5 M)          |                            |
|           |                      | New Madrid 6.5 m
|           |                      | 7.3 M               |                            |
|           |                      | Local 5.26 m
|           |                      | 6.5 M               |                            |
|           |                      | Wabash Valley 6.5 m
|           |                      | 7.3 M               |                            |
| Pier 4 Span 5 | 0.02 | 3.7 | 2.5 | 4.0 | 2.7 |
|              | 0.05 | 2.4 | 2.3 | 3.3 | 2.4 |
| Pier 8 Span 9 | 0.02 | 3.5 | 2.5 | 4.1 | 2.4 |
|              | 0.05 | 2.4 | 2.4 | 3.3 | 2.1 |

¹ Expansion Bearing Seat Location

Note: See Table 6.6a for metric units
**TABLE 6.7a:** Displacement Capacity/Demand Ratio Using Displacements due to the 7.3 m\(_{\text{M}}\) (8.5 M\(_{\text{s}}\)) New Madrid Earthquake Loading\(^1\) (Metric Units)

<table>
<thead>
<tr>
<th>Location(^2)</th>
<th>(\Delta_{\text{c}}) (cm)</th>
<th>(\Delta_{\text{d}}) (cm)</th>
<th>(\Delta_{\text{eq}}(d)) (cm)</th>
<th>C/D(^1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(2)</td>
<td>(3)</td>
<td>(4) (\xi=2)%</td>
<td>(5) (\xi=5)%</td>
</tr>
<tr>
<td>Upper Deck</td>
<td>Pier 4 Span 5</td>
<td>15.2</td>
<td>1.0</td>
<td>9.3</td>
</tr>
<tr>
<td></td>
<td>Pier 8 Span 9</td>
<td>15.2</td>
<td>0.8</td>
<td>9.0</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 12</td>
<td>22.9</td>
<td>1.3</td>
<td>3.8</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 13</td>
<td>22.9</td>
<td>2.3</td>
<td>3.3</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 15</td>
<td>22.9</td>
<td>1.0</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 16</td>
<td>22.9</td>
<td>2.3</td>
<td>3.3</td>
</tr>
<tr>
<td>Lower Deck</td>
<td>Pier 4 Span 5</td>
<td>15.2</td>
<td>2.4</td>
<td>6.7</td>
</tr>
<tr>
<td></td>
<td>Pier 8 Span 9</td>
<td>15.2</td>
<td>3.4</td>
<td>6.1</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 12</td>
<td>38.1</td>
<td>3.9</td>
<td>3.1</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 13</td>
<td>38.1</td>
<td>2.3</td>
<td>3.3</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 15</td>
<td>38.1</td>
<td>1.0</td>
<td>2.7</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 16</td>
<td>38.1</td>
<td>2.3</td>
<td>2.2</td>
</tr>
</tbody>
</table>

\(^1\)Capacity/Demand ratio \(C/D = \frac{\Delta_{\text{c}}(\text{c}) - \Delta_{\text{d}}(\text{d})}{\Delta_{\text{eq}}(d)}\)

\(\Delta_{\text{c}}(\text{c})\) = Allowable movement of the expansion joint or bearing.

\(\Delta_{\text{d}}(\text{d})\) = Maximum possible movement resulting from temperature, creep and shrinkage.

\(\Delta_{\text{eq}}(d)\) = Maximum relative displacement due to earthquake loading.

\(^2\)Expansion Bearing Seat Location

Note: See Table 6.7b for English units
## TABLE 6.7b: Displacement Capacity/Demand Ratio Using Displacements due to the $7.3 \text{ M}_{L_{\text{eq}}}$ (8.5 M$_{L}$) New Madrid Earthquake Loading¹ (English Units)

<table>
<thead>
<tr>
<th>Location²</th>
<th>Location</th>
<th>$\Delta_c$ (in)</th>
<th>$\Delta_d$ (in)</th>
<th>$\Delta_{\xi=2%}$ (in)</th>
<th>$\Delta_{\xi=5%}$ (in)</th>
<th>C/D¹</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
<td>(5)</td>
<td>(6)</td>
</tr>
<tr>
<td>Upper Deck</td>
<td>Pier 4 Span 5</td>
<td>6.0</td>
<td>0.4</td>
<td>3.7</td>
<td>2.4</td>
<td>1.53</td>
</tr>
<tr>
<td></td>
<td>Pier 8 Span 9</td>
<td>6.0</td>
<td>1.0</td>
<td>3.5</td>
<td>2.4</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 12</td>
<td>9.0</td>
<td>0.5</td>
<td>1.5</td>
<td>1.3</td>
<td>5.98</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 13</td>
<td>9.0</td>
<td>0.9</td>
<td>1.3</td>
<td>1.1</td>
<td>6.29</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 15</td>
<td>9.0</td>
<td>0.4</td>
<td>1.4</td>
<td>1.3</td>
<td>6.28</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 16</td>
<td>9.0</td>
<td>0.9</td>
<td>1.3</td>
<td>1.1</td>
<td>6.24</td>
</tr>
<tr>
<td>Lower Deck</td>
<td>Covington Approach</td>
<td>Pier 4 Span 5</td>
<td>6.0</td>
<td>0.9</td>
<td>2.6</td>
<td>2.2</td>
</tr>
<tr>
<td></td>
<td>Pier 8 Span 9</td>
<td>6.0</td>
<td>1.3</td>
<td>2.4</td>
<td>1.9</td>
<td>1.93</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 12</td>
<td>15.0</td>
<td>1.5</td>
<td>1.2</td>
<td>1.1</td>
<td>11.40</td>
</tr>
<tr>
<td></td>
<td>Pier 12 Span 13</td>
<td>15.0</td>
<td>0.9</td>
<td>1.3</td>
<td>1.2</td>
<td>10.85</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 15</td>
<td>15.0</td>
<td>0.4</td>
<td>1.1</td>
<td>1.1</td>
<td>13.74</td>
</tr>
<tr>
<td></td>
<td>Pier 15 Span 16</td>
<td>15.0</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>16.27</td>
</tr>
</tbody>
</table>

¹Capacity/Demand ratio $C/D = \frac{\Delta_c - \Delta_d}{\Delta_{\xi}}$

$\Delta_c$ = Allowable movement of the expansion joint or bearing.
$\Delta_d$ = Maximum possible movement resulting from temperature, creep and shrinkage.
$\Delta_{\xi}$ = Maximum relative displacement due to earthquake loading.

²Expansion Bearing Seat Location

Note: See Table 6.7a for Metric units

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7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

The October 17, 1989 Loma Prieta earthquake brought attention to the seismic risk of double-deck bridges and elevated double-deck freeway structures (Figures 1.1-1.2). Following the earthquake, the Federal Highway Administration commissioned the seismic evaluation of all double-deck bridges located in seismically active regions. The Brent-Spence bridge is a double-deck cantilever through truss bridge on Interstate 75 connecting Covington (Kentucky) to Cincinnati (Ohio) over the Ohio River (Figure 1.3).

The seismic zones nearest the Brent-Spence site are the New Madrid, Wabash Valley, and Anna seismic zones (Figure 1.4). Given the state of knowledge of seismic design and seismology when the bridge was designed and the low seismicity of the region, the designers presumed that wind loads governed lateral design and traffic loads governed longitudinal design. As a result, the structure’s response to dynamic earthquake loading has not been previously evaluated.

The primary aim of this study was to assess the structural integrity of the Brent-Spence bridge when subjected to a seismic event from the New Madrid, Wabash Valley, or Anna seismic zone. To achieve this, the scope of work was divided into several tasks as follows: experimental analysis of the main bridge (or field testing), analytical modeling, site specific ground motion, and seismic response analysis. A brief description of these tasks follows in sections 7.2.1-7.2.4 (sections 1.2.1-1.2.4 provide a detailed description).

7.1.1 Experimental Analysis

The ambient vibration properties of the main bridge were determined through field testing using traffic to excite the structure. The purpose of measuring the ambient vibration properties was to determine the mode shapes and the associated natural frequencies. These vibration properties were subsequently used as the basis for calibrating the analytical computer model for seismic response analysis.
7.1.2 Analytical Modeling

Applied dynamic loads and responses were regarded as disturbances of the current dead load configuration of the bridge.

A three dimensional finite element model of the main bridge was used for free vibration and seismic response analyses. The model was calibrated by comparing the free vibration analysis results with ambient vibration properties from field testing. Once calibrated, the model was then used for seismic response analysis.

The approach spans were modeled using simple one or two degree of freedom systems. The mass was concentrated over the piers at points of longitudinal fixity (i.e. lollipop model). Seismic response was analyzed in the longitudinal direction only.

7.1.3 Site Specific Ground Motion

The site specific ground motion scenarios were developed to represent probable earthquakes that may occur in the New Madrid, Wabash Valley, and Anna seismic zones. A local seismic event, defined as being 12.5 miles (20 km) from the site of the bridge, was also considered. Time histories and response spectra were generated for a number of events. The time histories were then used in the seismic analysis of the main bridge while the response spectra were used for analyzing the approach spans.

7.1.4 Seismic Response Analysis

The three dimensional model of the main bridge was subjected to the time histories of the aforementioned earthquakes to determine maximum displacements, forces and stresses. Maximum relative displacements were determined at a number of locations including the top and bottom of the piers, and between the upper and lower deck levels. The maximum forces and stresses were determined at a number of locations including the piers, towers, and anchor bolts at fixed bearings.

For the approach spans, the seismic analysis dealt only with the potential for loss of span due to excessive longitudinal displacement along...
the highway main line. The seismic analysis of the models for the approach spans was conducted using the response spectra. The maximum relative longitudinal displacements at expansion joints were determined and compared with the maximum support length to check for susceptibility to span loss.

7.2 Recommendations

7.2.1 Main Bridge

The seismic analysis indicates that the main bridge will resist the maximum credible earthquake in the elastic range without any damage or loss of span. Consequently, retrofitting is not required.

7.2.2 Approach Spans

The primary concern in this study for the approach spans is the potential for loss of span due to excessive longitudinal displacement. Based on the findings of the seismic analysis, the following actions are recommended:

1- Retrofit of the upper deck expansion joints at pier 1 thru pier 9 (Figure 3) is strongly recommended since the capacity/demand (C/D) ratios do not meet the current standard of AASHTO Specifications.

It is recommended that properly designed cable restrain system be installed at these expansion bearing seats. The components are easily available and installation is not outside the scope of local contractor capabilities.

2- Only the main-line approach spans were considered in this study. The access ramps were not evaluated and may be vulnerable to loss of span. It is recommended that the access ramps on both approaches be evaluated for possible loss of span.
3- In view of the results for the approach spans, it is recommended that a detailed three dimensional seismic analysis, including strength check of critical components, be conducted on the approach spans and the ramps.
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APPENDIX I: MODES OF VIBRATION

This appendix presents the first four modes of free vibration for each of the three principal directions. Figures I.1-I.4 present the results for the transverse direction, Figures I.5-I.8 for the vertical direction, and Figures I.9-I.12 for the longitudinal direction.
Figure 1.1  Transverse Mode #1  \( (f = 0.48 \text{ Hz}, T = 2.08 \text{ sec}) \)
Plan View

Elevation

Figure I.2  Transverse Mode #2  \( (f = 0.85 \text{ Hz}, T = 1.18 \text{ sec}) \)
Figure 1.3  Transverse Mode #3  \( (f = 0.83 \text{ Hz, } T = 1.08 \text{ sec}) \)
Figure I.4  Transverse Mode #4  \((f = 1.02 \text{ Hz}, T = 0.98 \text{ sec})\)
Figure I.5  Vertical Mode #1  \( f = 0.65 \text{ Hz, } T = 1.53 \text{ sec} \)
Figure I.6  Vertical Mode #2  \((f = 1.16 \text{ Hz}, \ T = 0.86 \text{ sec})\)
Figure I.7  Vertical Mode #3  (f = 1.43 Hz, T = 0.70 sec)
Figure I.8  Vertical Mode #4  \( f = 1.89 \text{ Hz}, T = 0.53 \text{ sec} \)
Figure I.9  Longitudinal Mode #1  \( f = 0.99 \text{ Hz}, \ T = 1.01 \text{ sec} \)
Figure I.10  Longitudinal Mode #2  \((f = 1.16 \text{ Hz}, T = 0.36 \text{ sec})\)
Figure I.11  Longitudinal Mode #3  \( (f = 1.89 \text{ Hz}, T = 0.53 \text{ sec}) \)
Figure I.12  Longitudinal Mode #4  \( (f = 3.01 \text{ Hz}, T = 0.33 \text{ sec}) \)
APPENDIX II: MODEL VERIFICATION

This appendix presents the bar charts showing the range of experimental frequencies and compares the experimental and theoretical mode shapes for the first three modes of vibration and for the three principal directions. Figures II.1-II.4 present the results for the transverse direction, Figures II.5-II.8 for the vertical direction, and Figures II.9-II.12 for the longitudinal direction.
Figure II.1 Frequency Distribution and Mode Shape for Transverse Mode #1
Figure II.2  Frequency Distribution and Mode Shape for Transverse Mode #2
Figure II.3  Frequency Distribution and Mode Shape for Transverse Mode #3
Figure II.4  Frequency Distribution and Mode Shape for Transverse Mode #4
Figure II.5  Frequency Distribution and Mode Shape for Vertical Mode #1
Figure II.6  Frequency Distribution and Mode Shape for Vertical Mode #2
Figure II.7  Frequency Distribution and Mode Shape for Vertical Mode #3
Figure II.8  Frequency Distribution and Mode Shape for Vertical Mode #4
Figure II.9  Frequency Distribution and Mode Shape for Longitudinal Mode #1
Figure II.10  Frequency Distribution and Mode Shape for Longitudinal Mode #2
Figure II.11  Frequency Distribution and Mode Shape for Longitudinal Mode #3
Figure II.12 Frequency Distribution and Mode Shape for Longitudinal Mode #4
APPENDIX III: GROUND MOTION

This Appendix presents the time-histories and response spectra for four hypothetical seismic events, and they are presented in sections III.1 to III.4 as follows:

III.1 New Madrid 7.3 $m_{b,lg}$ (8.5 $M_s$)
III.2 New Madrid 6.5 $m_{b,lg}$ (7.4 $M_s$)
III.3 Wabash Valley 6.5 $m_{b,lg}$ (7.4 $M_s$)
III.4 Local 5.25 $m_{b,lg}$ (4.6 $M_s$)

For each event, the following are provided: (1) time histories for the acceleration, velocity and displacement; and (2) response spectra for the transverse, vertical and longitudinal directions.
III.1 New Madrid 7.3 m_{b,Lg} (8.5 M_s)
Figure III.1.1. Acceleration Time History for the 7.3 m$_{b_{1,0}}$ (8.5 M$_{s}$) New Madrid Event (Note: 1 cm = 0.394 in)
Figure III.1.2. Velocity Time History for the 7.3 $m_{b,LF}$ (8.5 $M_s$) New Madrid Event (Note: 1 cm = 0.394 in)
Figure III.1.3. Displacement Time History for the 7.3 $m_{b,L}$ (8.5 $M_s$) New Madrid Event
(Note: 1 cm = 0.394 in)
Figure III.14: Transverse Earthquake Spectrum for the 7.3 m_w. (8.5 M_s) New Madrid Event (Note: 1 in = 2.54 cm)
Figure III.1.5. Vertical Earthquake Spectrum for the 7.3 m.b. (8.5 M,) New Madrid Event  (Note: 1 in = 2.54 cm)
Figure III.1.6. Longitudinal Earthquake Spectrum for the 7.3 $m_{b.LS}$ (8.5 $M_s$) New Madrid Event (Note: 1 in = 2.54 cm)
III.2 New Madrid $6.5 \, m_{b,Lg} \, (7.4 \, M_s)$
Figure III.2.2.  Velocity Time History for the 6.5 $m_{b, L_g}$ (7.4 $M_s$) New Madrid Event (Note: 1 cm = 0.394 in)
Figure III.2.3. Displacement Time History for the 6.5 \( m_{b,lg} \) (7.4 \( M_s \)) New Madrid Event
(Note: 1 cm = 0.394 in)
Figure III.2.4. Transverse Earthquake Spectrum for the 6.5 $m_{b,L}$ (7.4 $M_s$) New Madrid Event (Note: 1 in = 2.54 cm)
Figure III.2.5. Vertical Earthquake Spectrum for the 6.5 $m_{_{\text{w}}}(7.4 \text{ M}_s)$ New Madrid Event (Note: 1 in = 2.54 cm)
Figure III.2.6. Longitudinal Earthquake Spectrum for the 6.5 $m_{b}L_s$ (7.4 $M_s$) New Madrid Event (Note: 1 in = 2.54 cm)
III.3 Wabash Valley  $6.5 \text{ m}_{b,\text{Lg}} \ (7.4 \text{ M}_\text{S})$
Figure III.3.1. Acceleration Time History for the 6.5 \( m_{b, L_{g}} \) (7.4 M,) Wabash Valley Event
(Note: 1 cm = 0.394 in)
Figure III.3.2. Velocity Time History for the 6.5 m₁₀₀ (7.4 M₄) Wabash Valley Event (Note: 1 cm = 0.394 in)
Figure III.3.8. Displacement Time History for the 6.5 mb, 7.4 M, Wabash Valley Event (Note: 1 cm = 0.394 in)
Figure III.3.4. Transverse Earthquake Spectrum for the 6.5 $m_{b, L_4}$ (7.4 $M_L$) Wabash Valley Event (Note: 1 in = 2.54 cm)
Figure III.3.5. Vertical Earthquake Spectrum for the $6.5 \text{ m}_{hL4}$ ($7.4 \text{ M}_L$) Wabash Valley Event (Note: 1 in = 2.54 cm)
Figure III.3.6. Longitudinal Earthquake Spectrum for the 6.5 m_b, Lg (7.4 M_s) Wabash Valley Event (Note: 1 in = 2.54 cm)
III.4 Local 5.25 $m_{b,Lg}$
Figure III.4.1. Acceleration Time History for the 5.25 m (20.6 ft) Local Event (Note: 1 cm = 0.0394 in)
Figure III.4.2. Velocity Time History for the 5.25 m$_{b,L}$ (4.6 M$_{s}$) Local Event (Note: 1 cm = 0.394 in)
Figure III.4.3. Displacement Time History for the 5.25 m_{h,kg} (4.6 M_{t}) Local Event (Note: 1 cm = 0.394 in)
Figure III.4.4. Transverse Earthquake Spectrum for the 5.25 m$_{b,L}$ (4.6 M$_{s}$) Local Event (Note: 1 in = 2.54 cm)
Figure III.4.5. Vertical Earthquake Spectrum for the 5.25 m$_{b,lg}$ (4.6 M$_{s}$) Local Event (Note: 1 in = 2.54 cm)
Figure III.4.6. Longitudinal Earthquake Spectrum for the 5.25 m_h.Lg (4.6 M_s) Local Event (Note: 1 in = 2.54 cm)
APPENDIX IV: REFERENCES


7.- Drake, D. (1815). "Natural and statistical view, a picture of Cincinnati and the Miami country, illustrated by maps, with an appendix, containing observations on the late earthquake, the aurora borealis, and the south-west wind", *Looker and Wallace, Cincinnati,* Ohio, 251 p.


Additional References


