Research Report
KTC-07-05/SPR246-02-4F

KENTUCKY TRANSPORTATION CENTER

DETAILED SEISMIC EVALUATION OF BRIDGES ON AND OVER THE PARKWAYS IN WESTERN KENTUCKY

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DETAILED SEISMIC EVALUATION OF BRIDGES ON AND OVER THE PARKWAYS IN WESTERN KENTUCKY

by

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in cooperation with

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Commonwealth of Kentucky

and

Federal Highway Administration
U.S. Department of Transportation

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16. Abstract

This report outlines a rating system and details an evaluation procedure for the seismic evaluation of highway bridges. These processes are later used to investigate the structural integrity of selected highway bridges on and over the parkways in Western Kentucky. A total of 349 bridges were rated with the bridge ranking system, and 17 were selected for detailed seismic evaluation.

The capacity/demand ratio method developed by the Federal Highway Administration is employed for the detailed evaluation of bridges. The evaluation involves creating three-dimensional finite element models for all 17 bridges. Dynamic analysis is performed for a projected 250-year event. Among the four components required for proper evaluation, only expansion joints and bearings, and columns and footings were investigated.

The evaluating results indicate that the rating system is an effective means in determining and prioritizing highway bridges for seismic evaluation and retrofit processes. The different forms of deficiencies of the bridges are presented herein.

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EXECUTIVE SUMMARY

Background

Five parkways in Western Kentucky are located in the region that is greatly influenced by the New Madrid Seismic Zones (NMSZ). The last major earthquake near this region was the Great New Madrid Earthquake of 1811-1812 with a magnitude of 7.5 or greater on the Richter scale. The NMSZ remains active, recording about 200 earthquakes per year, though most of them are too small to be felt by humans. Seismologists, however, believe that there is a high probability of a major earthquake event in the near future. Due to locality and socioeconomic factors, these parkways are listed as the high priority and emergency routes in the region. Hence, it is essential that the parkways remain functional and operational during a major earthquake event.

The primary objective of this study is to investigate the structural integrity, by performing detailed seismic evaluations, of highway bridges which are deemed susceptible to severe damage in a major earthquake event. The objective is achieved by executing the following tasks: (1) select bridges for detailed seismic evaluations based on a seismic rating system; (2) perform detailed seismic evaluation; and (3) summarize and highlight deficiencies, and make retrofitting recommendation if necessary.

Seismic Rating System of Bridges on the Western Kentucky Parkways

The preliminary screening process, or the Seismic Rating System, of bridges is used to identify and prioritize bridges that are in need of a detailed seismic evaluation. One of the tasks in the screening process involves the collection of all structural inventories and site investigations of all bridges on and over parkways in Western Kentucky. The major components in determining the rank of a particular bridge include the following: structural vulnerability, seismic and geotechnical hazards, and socioeconomic factors.

A total of 349 bridges on and over the parkways in Western Kentucky were rated based on the aforementioned procedure. The bridge rank (R) ranges from a low of 0 to a high of 75, based on a scale of 100. For detailed seismic analysis, a total of 17 bridges, including parallel bridges, were selected. The 17-bridges have an average ranking of 58, with a highest bridge rank of 75. All of the selected bridges were constructed in the 1960s, when seismic design was not taken into consideration. The selected bridges are of different construction types: reinforced and prestressed concrete, and steel composites bridges.

Detailed Seismic Evaluation of Bridges on the Western Kentucky Parkways

Selected bridges were evaluated using the Capacity/Demand (C/D) ratio method proposed in the Seismic Retrofitting Manual for Highway Bridges (Buckle and Friedland, 1995). The evaluation of the expansion joints and bearings, and columns and footings, is carried out in this report. The abutments of bridges on the Western Kentucky Parkways are investigated in a separate report which focuses on the seismic or soil stability of the abutments and the potential of liquefaction of the underlying soil.

In this report, the detailed seismic evaluation includes the creation of a 3-dimensional finite element computer model for each of the selected bridges, and a dynamic analysis using a projected 250-year seismic time history.

Results and Summary

Majority of bridges evaluated in this process have some forms of deficiencies and required retrofit. Summary Tables E.1 and E.2 provide details of the deficiencies and recommendations. Of the bridges evaluated, four bridges [30-9005-B00060, 30-9005-B00061 and 42-9003-B00157 (P)] with ranks of 38.0 and 35.1 possess no seismic deficiency.

NOTE: This report is the fourth (4^{th}) in a series of six (6) reports for Project SRP 246: "Seismic Evaluation of Bridges along Western Kentucky Parkways". The six (6) reports are:

Report Number:	Report Title:
(1) KTC-07-02/SPR246-02-1F	Seismic Evaluation of Bridges on and over the Parkways in Western Kentucky – Summary Report
(2) KTC-07-03/SPR246-02-2F	Site Investigation of Bridges on and over the Parkways in Western Kentucky
(3) KTC-07-04/SPR246-02-3F	Preliminary Seismic Evaluation and Ranking of Bridges on and over the Parkways in Western Kentucky
(4) KTC-07-05/SPR246-02-4F*	Detailed Seismic Evaluation of Bridges on and over the Parkways in Western Kentucky
(5) KTC-07-06/SPR246-02-5F	Seismic Evaluation and Ranking of Embankments for Bridges on and over the Parkways in Western Kentucky
(6) KTC-07-07/SPR246-02-6F	Seismic-Hazard Maps and Time Histories for the Commonwealth of Kentucky

* Denote current report

Table E.1: C/D Ratios of Select Bridges on and over Parkways in Western Kentucky.

Bridge Identification Number (BIN)	Capacity/Demand (C/D) ratios of different bridge components											
	Joints and/or Bearings		Columns and/or Footing									Bridge Ranks
	r_{bd}	r_{bf}	r_{ec}	r_{ef}	r_{ca} (Cap)	r_{ca} (Footing)	r_{cs} (Cap)	r_{cs} (Footing)	r_{cc}	r_{cv}	r_{fr}	
38-0051-B00012	1.28	3.97	0.29	0.62	1.00	0.62	-	0.37	0.99	0.29	2.48	75.0
38-0307-B00015	1.28	5.50	0.36	0.78	1.00	0.78	-	0.40	1.17	1.51	3.13	75.0
38-9003-B00053 38-9003-B00053P	1.26	1.56	0.47	0.23	1.00	1.00	-	0.62	1.50	2.37	0.91	75.0
38-9003-B00054 38-9003-B00054P	1.30	0.90	0.23	0.15	1.00	1.00	-	0.30	0.74	1.15	0.58	75.0
38-9003-B00055 38-9003-B00055P	1.19	1.19	0.38	0.33	1.00	1.00	-	0.56	1.21	1.75	0.67	75.0
53-0094-B00050	1.33	5.51	0.35	0.55	1.00	0.55	-	0.39	1.14	1.34	1.09	75.0
53-1529-B00056	1.30	4.42	0.27	0.39	1.00	0.39	-	0.30	0.88	0.76	1.16	75.0
53-9003-B00068	1.07	0.85	0.28	0.39	1.00	0.39	-	0.31	0.91	0.28	0.79	75.0
30-9005-B00060	1.27	1.38	2.20	2.72	1.00	1.00	-	1.65	6.95	3.99	2.72	38.0
30-9005-B00061	1.39	1.20	2.07	2.33	1.00	1.00	-	1.55	6.87	2.48	2.33	38.0
42-9003-B00157 42-9003-B00157P	1.20	1.32	-	-	-	-	-	-	-	-	-	35.1
117-9004-B00071 117-9004-B00071P	0.99	2.15	0.76	1.14	1.00	1.00	-	0.67	2.42	3.78	1.14	8.4

Note: C/D ratio value less than 1.0 is highlighted in the table
Cells indicated with (-) represent C/D ratios are not applicable in these cases
r_{bd} = Joint/bearing displacement C/D ratio
r_{ef} = Footing force C/D ratio
r_{cc} = Transverse conferment C/D ratio
r_{bf} = Joint/bearing force C/D ratio
r_{fr} = Footing rotation/yeiding C/D ratio
r_{ec} = Column force C/D ratio
r_{cs} = Splice C/D ratio of cap or footing
rc_v = Column shear C/D ratio
r_{ca} = Anchorage C/D ratio of cap or footing

Table E.2: Summary of Seismic Deficiencies of Selected Bridges.

Bridge Identification Number (BIN)	Ranking	Seismic Deficiencies
38-0051-B00012	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
38-0307-B00015	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
38-9003-B00053 38-9003-B00053P	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
38-9003-B00054 38-9003-B00054P	75.0	- Bearing seat capacity - Footing flexural capacity - Column shear capacity - Column flexural capacity
38-9003-B00055 38-9003-B00055P	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
53-0094-B00050	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
53-1529-B00056	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
53-9003-B00068	75.0	- Bearing seat capacity - Footing flexural capacity - Column shear capacity - Column flexural capacity
30-9005-B00060	38.0	-
30-9005-B00061	38.0	-
42-9003-B00157 42-9003-B00157P	35.1	-
117-9004-B00071 117-9004-B00071P	8.4	- Bearing seat capacity - Footing flexural capacity - Column flexural capacity

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1. INTRODUCTION

1.1 The New Madrid Seismic Zone

The New Madrid Seismic Zone (NMSZ) extends more than 120 miles southward from Cairo, Illinois, at the junction of the Mississippi and Ohio rivers, into Arkansas and parts of Kentucky and Tennessee.

The greatest earthquake risk east of the Rocky Mountains is along the NMSZ. Damaging earthquakes are not as frequent as in California, but when they do occur, the destruction covers more than 15 times the area because of the underlying geology and soil conditions prevalent in the region (National Earthquake Information Center, 2003). The zone is active, averaging about 200 earthquakes per year, though most of them are too small to be felt by humans.

A damaging earthquake in this area (6.0 or greater on the Richter scale) occurs, on average, once every 80 years – an estimated magnitude 6.4 occurred near Marked Tree, Arkansas, in 1843, and another earthquake with an estimated magnitude of 6.8 occurred near Charleston, Missouri, in 1895. A major earthquake (7.5 or greater) occurs every 200-300 years. It is believed that there is a 10% chance of such a disaster by the year 2000 and a 25% chance by 2040. The last major earthquake was the Great New Madrid Earthquake of 1811-1812. This earthquake occurred over a series of over 2000 tremors in five months, five of which were 8.0 or more in magnitude (National Earthquake Information Center, 2003). Fig. 1.1 shows the Modified Mercalli intensity for the first event of the 1811-1812 New Madrid earthquakes (Bolt, 1993).

1.2 Parkways in Western Kentucky

Kentucky’s parkway system consists of nine highways across the state. There are five parkways located in western of the state, close to the region that is greatly influenced by the New Madrid Seismic Zones (NMSZ). These five parkways are Audubon Parkway, Pennyrile Parkway, Purchase Parkway, Western Kentucky Parkway, and William Natcher Parkway. These parkways were constructed during the 1960s, in which seismic design was not taken into consideration, to augment the state’s interstate highways.

Unlike roads called parkways in other states, Kentucky’s parkways are not closed to commercial traffic. The parkways in Western Kentucky cross seventeen counties in Western Kentucky as shown in Figure 2 and are critical routes. That said plans are for parts of the William Natcher, and Western Kentucky Parkways to become part of Interstate 66 and for parts of the Pennyrile, Western Kentucky, and Purchase Parkways to become part of Interstate 69.

Due to locality and socioeconomic factors, these parkways are listed as the high priority and emergency routes in Western Kentucky. As a result, bridges on and over parkways in Western Kentucky are deemed essential and they must remain open and provide undisrupted access during an earthquake event. It is for this reason that the Commonwealth of Kentucky has sponsored numerous efforts to analyze and examine the structural integrity of these bridges located within the danger zone, primarily those in Western Kentucky, located within the NMSZ.

The primary objective of this study is to perform a detailed seismic evaluation on selected bridges on and over parkways in Western Kentucky; such bridges are considered vulnerable to a seismic event based on a Seismic Rating System. The complete details of a Seismic Rating System and the ranking of all bridges on and over parkways in Western Kentucky are presented in a separate research report. A brief summary, however, of the Seismic Rating System will be described herein. The selected bridges based on this rating system for detailed seismic evaluation will be also included.

1.3 Seismic Rating System

In general, the Seismic Rating System described in this section is used as a basis for selecting bridges for detailed seismic evaluation, which will be described in Chapter 2. The information provided herein is obtained from the Seismic Retrofitting Manual for Highway Bridges (Buckle, I.G. and Friedland, I.M., 1995), published by the Federal Highway Administration (Report No. FHWA-RD-94-052). The Seismic Rating System will be explained with the aid of Figure 3.

Step 1: Determination of Acceleration (A) and Importance (I) coefficients

Peak ground acceleration contour maps, defining seismic zones and response spectra, are given for each Kentucky county basis for the seismic design of new bridges and seismic evaluation of existing bridges. Peak ground acceleration (PGA), as a function of the acceleration (A) coefficient and gravitational acceleration constant (g=9.81 m/sec^2 or 386 in/sec^2), in deferent county is obtained from a time history response spectra (TR-250Y-0.xxg-x) identification map for 50-year event and 250-year event derived by Street et al (1996).

Two categories used to describe the Importance (I) coefficient, as documented in the Seismic Retrofitting Manual are: essential and standard. All bridges on and over parkways in Western Kentucky are essential bridges.

Step 2: Determination of Seismic Performance Category

Table 1 is used to determine the Seismic Performance Category (SPC) based primarily on Acceleration (A) and Importance (I) coefficients as previously described.

Step 3: Determination of Soil Profile Type or Site (S) coefficients

Table 2 shows how the different soil profile type or site (S) coefficient is determined.

Step 4: Determination of Structural Vulnerability Rating (V)

Vulnerability rating (V) is determined based on four bridge components: (a) the connections, bearings, and seats; (b) columns and foundations; (c) abutments; and (d) soils. The flow chart shown in Figure 4 illustrates how V is determined (for further details see the Seismic Retrofitting Manual, Section 2.3.1.1).

Step 5: Determination of Seismic Hazard Rating (E)

Seismic hazard rating (E) is calculated using the following equation:

$$E = 12.5 \cdot A \cdot S \leq 10 \quad \text{(Seismic Retrofitting Manual, Eq. 2-4)}$$

Step 6: Calculation of bridge rank

The bridge rank (R) is calculated by multiplying structural vulnerability rating (V) and seismic hazard rating (E) together:

$$R = V \cdot E \quad \text{(Seismic Retrofitting Manual, Eq. 2-2)}$$

1.4 Bridges Selected for Detailed Seismic Evaluation

The seismic rating or bridge ranking system described in the previous section was used to evaluate 349 bridges on and over parkways in Western Kentucky, near the NMSZ. The rankings (R) of these bridges fall between 0 and 75 on a scale of 100. Based on the ranking system, the bridges, which rank 35 or higher, are selected for detailed seismic evaluation as indicated in Table 3. Moreover, according to the Seismic Retrofitting Manual, some irregular bridges should be processed with detailed seismic evaluation.

2. DETAILED SEISMIC EVALUATION OF SELECTED BRIDGES

2.1 General

The Seismic Retrofitting Manual for Highway Bridges (Buckle, I.G. and Friedland, I.M, 1995), SR Manual hereafter, published by the Federal Highway Administration (Report No. FHWA-RD-94-052), will be used as a guide for detailed seismic evaluation of selected bridges.

The SR Manual proposes two methods – the Capacity/Demand (C/D) ratio method and the Lateral Strength method – for detailed seismic evaluation of bridges requiring a detailed analysis based on the their Seismic Performance Category.

In general, the Lateral Strength method treats the entire bridge system, whether individual segments or frames of the bridge between expansion joints, as a single structural system. The structural system is then evaluated using an incremental collapse mechanism approach (SR Manual, Section 3.3.3).

The Capacity/Demand (C/D) ratio method, on the other hand, evaluates the individual bridge components' (expansion joints, bearings, columns, footings, etc.) ability to resist the design earthquake. In general, the seismic demands (D) of individual components are determined from an elastic spectral analysis. The seismic capacities (C) of individual components are computed at their nominal ultimate values without capacity reduction factors, φ (SR Manual, Section 3.4). The capacities and demands can be forces, displacements, and other quantities that define the performance of the bridge. In this method, a calculated C/D ratio of less than 1.0 indicates that component failure may occur during the design earthquake, and consequently, retrofitting of such components may be required.

The C/D method typically results in conservative retrofitting measures, which lead to higher costs. The lateral strength method, in general, yields more accurate results, hence lower retrofitting costs (Harik et. al., 1997). However, due to the complex nature of the lateral strength method, the C/D method is often preferred, and the latter method is adopted for all bridge analyses performed in this report.

2.2 Capacity/Demand Ratio Method

Bridges components that have the potential of being damaged during an earthquake should be evaluated quantitatively to determine their ability to resist the design earthquake. This should be done by calculating the seismic C/D ratio for each of the potential modes of the following bridge components:

(1) Expansion joints and/or bearings;
(2) Columns, piers and/or footings;
(3) Abutments; and
(4) Liquefaction.

For this investigation, ONLY items (1) and (2) will be evaluated and reported. Items (3) and (4) are investigated and presented in a separate report. The stability analysis of the bridge abutments [Item (3)] and the liquefaction analysis of the foundation soil [Item (4)] will be presented in separate research reports.

To analyze the individual bridge components, the demands (forces and/or displacements) of the individual bridge components must first be calculated. In general, 3 dimensional bridge models are created for finite element analysis. This process is performed with the aid of a commercially available structural analysis computer program, e.g. SAP2000 (Wilson E.L., 1998), from which the demands of the components are derived. A schematic showing the three orthogonal directions of a bridge is presented in Figure 5.

In general, the longitudinal direction is assumed to lie along the centerline of the bridge, and the transverse direction is then the perpendicular direction to the longitudinal axis, as shown in Figure 5. Once seismic demands are calculated in each direction for specific individual bridge component, the demands are then combined to produce an overall demand (D) on the individual component. The combination of orthogonal seismic force and/or displacement demands is required to account for the directional uncertainty of earthquake motions and the simultaneous occurrence of earthquakes in two perpendicular horizontal directions (SR Manual, Section 3.3.2.4). The larger of the following two combinations of seismic demands are used for further analysis:

Combination 1: 100% of longitudinal demands plus 30% of transverse demands;
Combination 2: 100% of transverse demands plus 30% of longitudinal demands.

Guidelines for the capacity of individual bridge components are given in Section 3.6 and Appendix A of the SR Manual. A list of the capacity/demand ratios for the detailed seismic evaluation is presented in Table 4.

2.3 Capacity/Demand Ratios for Expansion Joint and/or Bearing

In general, two C/D ratios, the displacement C/D ratio, r_{bd}, and the force C/D ratio, r_{bf}, must be checked for expansion joints and/or bearings as proposed by the SR Manual. The procedures of determining these ratios are described as follows:

2.3.1 Displacement C/D Ratios for Expansion Joint and/or Bearing

The displacement C/D ratio, r_{bd}, calculation is explained with the aid of Figure 6. Section A.4.2 of the SR Manual proposes two methods to calculate the displacement C/D ratios, methods 1 and 2. The lesser of the C/D ratios calculated by methods 1 and 2 is used for the expansion joint and/or bearing. When the calculated r_{bd} is less than 1, retrofitting measures must be taken.

2.3.2 Force C/D Ratios for Expansion Joint and/or Bearing

The force C/D ratios, r_{bf}, for bearings and expansion joint restrainers are discussed in Section A.4.3 of the SR Manual. Specifically, the force demand, $V_b(d)$, is calculated by multiplying the elastic analysis value by 1.25. For cases where elastic analysis has not been carried out, it can be assumed that the force demand is 20 percent of the dead load of the superstructure. The bearing force capacity, $V_b(c)$, depends on the type of bearing supports. For instance, the bearing capacity may be the shear resistance provided by the shear key or the frictional force provided by the bearing pads.

2.4 Capacity/Demand Ratios for Column and Footing

2.4.1 Force C/D Ratios for Column and Footing

The determination of the column and footing C/D ratios, r_{ec} and r_{ef}, is explained in this section. Firstly, the moment demands of the columns and footings, $M_c(d)$ and $M_f(d)$, of substructures are determined by elastic analysis for the seismic load combinations described in Section 2.2. The elastic moment demands may be taken as the sum of the absolute values of the earthquake and dead load moments as described in the SR manual. The nominal ultimate moment capacities for both the column and footing, $M_c(c)$ and $M_f(c)$, are then calculated from the axial loads due to the earthquake and the self-weight of the structure. Lastly, the column and footing force C/D ratios can be determined using the following expressions:

$$r_{ec} = \frac{M_c(c)}{M_c(d)} \quad \text{– Column force C/D ratio}$$

$$r_{ef} = \frac{M_f(c)}{M_f(d)} \quad \text{– Footing force C/D ratio}$$

2.4.2 Anchorage of Longitudinal Reinforcement

A sudden loss of column flexural strength can occur if longitudinal reinforcement is not properly anchored. The determination of the anchorage ratio, r_{ca}, of longitudinal reinforcement is explained with the aid of Figure 7.

2.4.3 Splices in Longitudinal Reinforcement

Longitudinal reinforcements that are not well confined by closely spaced transverse reinforcement have the potential of losing flexural strength near or within the yielding zone. The procedure used to determine the adequacy of splice in longitudinal reinforcement is illustrated in Figure 8.

2.4.4 Column Shear

Column shear failure occurs when column shear capacity is exceeded. To illustrate how the C/D ratio of column shear is calculated, Figure 9 is presented.

2.4.5 Transverse Confinement Reinforcement

Adequate transverse confinement reinforcement in columns must be present to prevent buckling of the main reinforcement and crushing of concrete in compression, which ultimately leads to loss of strength and serviceability. The degree to which degradation is prevented depends largely on the amount and spacing of transverse reinforcement and the adequacy of the anchorage of this reinforcing. The transverse confinement C/D ratio, r_{cc}, can be determined by multiplying the C/D ratio of column, r_{ec}, with a ductility indicator, μ (for further details, see SR Manual Section A.5.4). For a conservative estimate, a ductility indicator of 2 may be used as indicated in the SR Manual. Note that the transverse confinement C/D ratio, r_{cc}, should only be investigated when the column force C/D ratio, r_{ec}, is less than 0.8, as proposed in the SR Manual (Cases III and IV).

2.4.6 Footing Rotation and/or Yielding

The seismic C/D ratio for footing rotation and/or yielding, r_{fr}, can be determined by multiplying the C/D ratio of footing, r_{ef}, with the ductility indicator, μ (SR Manual Section A.5.5). The ductility indicator, μ, is dependent on the type of footing and the mode of footing failure. The ductility indicator, μ, can be determined from Table 5 as proposed by the SR Manual. The ratio, r_{fr}, should only be calculated when r_{ef} is less than 0.8 (Cases II and IV in the SR Manual).

3. FINITE ELEMENT ANALYSIS METHOD WITH SAP 2000

When using Capacity/Demand Ratio Method for detailed seismic evaluation of selected bridges, there are many terms should be calculated beforehand with Finite Element Method (FEM). Base on this reason, the finite element analysis method with SAP 2000 is presented here for illustrative purpose.

3.1 About SAP 2000

The SAP name has been synonymous with State-of-the-art analytical methods since its introduction over 30 years ago. SAP2000 follows in the same tradition featuring a very sophisticated, intuitive and versatile user interface powered by an unmatched analysis engine and design tools for engineers working on transportation, industrial, public works, sports, and other facilities.

From its 3D object based graphical modeling environment, to the wide variety of analysis and design options completely integrated across one powerful user interface, SAP2000 has proven to be the most integrated, productive and practical general purpose structural program on the market today.

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English as well as SI and MKS metric units can be used to define the model geometry and to specify design parameters.

3.2 Procedure of Finite Element Analysis Method

SAP 2000 provides several analysis types, the procedures of which are different from each other. According to the Seismic Retrofitting Manual, the Capacity/Demand method need a time history seismic analysis. The procedure of finite element analysis method will be explained as following:

Step 1: Set up 3-D Model

New models may be created with very little effort using pre-programmed bridge template. In order to use the bridge template, the following information must first be found in the drawing plan: *Number of Plans*, *Number of Girders*, *Number of Columns*, *Span Length*, *Girder Spacing*, *Column Spacing*, *Column Height* and *Skew angle*.

All the dimensional values, except the column spacing, should be assigned according to the as-built plan. Column spacing is adjusted to be normal to girder. Parameters of the selected bridges are summarized and tabulated in Table 6.

Step 2: Define Materials

The materials properties in the FEM analysis should be found from the bridge drawing plan. All the 17 selected bridges were constructed in the 1960s, and the materials, including concrete, reinforcing steel, structural steel and anchorage bolt, were selected almost the same. The design stresses of these materials are presented here as following:
f_s = Reinforcing yielding strength = 20, 000 psi
f_c' = Concrete compression strength (cylinder) = 3, 000 psi
E_s = Coefficient of thermal expansion = 5.500E-6
f_{sb} = Yielding strength of anchorage bolt = 60, 000 psi

Step 3: Define Sections and Assign

The 3-D bridge model is composed from several elements, including some frame elements such as girders, diaphragm beams, columns (piers) and caps, and some shell elements such as bridge decks and pier-walls. These frame sections and shell sections should be defined as the design dimensions and assigned to the elements in the 3-D model.

Step 4: Define and Assign Static Loads

In any type of analysis, dead load must be considered. So, the dead load is one of the static loads. Generally, the dead load need not assign to the 3-D model because the materials and sections are defined and assigned to the elements in the model. However, the masses of such components as barriers, which do not considered in building the bridge model for the reason of simplification, should be manually assigned to appropriate joints.

In order to get the displacement of bearings due to temperature, another static load case should be defined and all the elements in the model should be assigned a temperature of 20 degree by element in this load case.

<u>Step 5</u>: Define Time History Functions and Time History Cases

The Time History Response Spectra are quoted from the Report KTC-96-4 for the 250-year earthquake event. The earthquake duration is 20.5 seconds consisting of 4,100 data points at 0.005-second intervals. Different bridge should take different functions according to the county where the bridge located. Figure 10 is the identification map of 250-year earthquake event for Kentucky. From Figure 10, the Time History Response Spectra of all the selected bridges can be identified and tabulated in Table 7. The acceleration time history pictures for the direction of Component-1 (transverse), Component-2 (vertical) and Component-3 (longitudinal) of different identifications are illustrated in Figure 11 through Figure 22.

Two time history cases are defined to account for the directional uncertainty of earthquake motions and the simultaneous occurrences of earthquake forces to two perpendicular horizontal directions. Time History Case I is defined as Component-1, Component-3 and Component-2 for the three orthogonal directions and Time History Case II is defined as Component-3, Component-1 and Component-2.

<u>Step 6</u>: Set Analysis Options and Run Analysis

In order to get the dynamic analysis results, *Dynamic Analysis* option should be checked in Analysis Options form in SAP 2000, and the dynamic parameters should be set in Dynamic Analysis Parameters form.

Upon completing all the above steps, it should be taken a couple of minutes for SAP 2000 to run analysis before the analysis results are available. These analysis results include displacement of joints resulting from temperature and earthquake, reaction force of bearing seat, force of frame sections such as axial load, shear force and moment in different load case.

4. DETAIL SEISMIC EVALUATION EXAMPLE OF ONE BRIDGE OVER AUDUBON PARKWAY

For illustrative purposes, the Lyddane Br. Rd. Bridge over Audubon Parkway in Daviess County, KY, is selected for detailed seismic evaluation:

4.1 Bridge Description

Figure 10 shows a three-dimensional view of the Example Bridge over Audubon Parkway in Daviess County, KY. The continuous structure, with two equal spans of 104 ft, was constructed in 1967. The superstructure consists of four steel plate I-girders supporting an eight-inch concrete bridge deck. The substructure – pier – is made up of three columns supported on a pile footing (Figure 11). The footing pedestal has a thickness equal to that of a column, 36 in. Soft to medium-stiff clays and sands were found at the bridge site.

4.2 Bridge Classification and Analysis Procedure

Based on the acceleration contour map, the 250-year design acceleration coefficient for Daviess County is A = 0.15g. Since the bridge is located along a priority route, this bridge is viewed as "Essential" based on AASHTO specifications. This combination of acceleration coefficient and importance classification gives the seismic performance category (SPC) of C (refer to SR Manual Section 1.5).

Section 3.3.2.1 of the SR Manual specifies the minimum dynamic analysis required for a bridge. Lyddane Br. Rd. Bridge is a "regular" bridge by SR Manual definition. Based on the criterion set forth in the SR Manual, a regular bridge has less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry, and no large changes in these parameters from span-to-span or support-to-support. Therefore, a uniform-load or single-mode spectral method should be specified as the minimum required analysis.

4.3 Bridge Components that Require Seismic Evaluation

Table 4 in Section 2.2 lists the bridge components required for seismic evaluation wherever is applicable. For this bridge, almost all C/D ratios listed in Table 4 will be investigated.

Seismic demands of individual bridge components are determined using a structural analysis computer program, SAP 2000. A three dimensional bridge model was built in SAP 2000 for this purpose.

Details and results of the computer analysis are excluded in this example. The seismic demands in the subsequent section are obtained from results generated by the computer analysis.

4.4 Capacity/Demand Ratios for Expansion Joint and/or Bearing

4.4.1 Displacement C/D Ratios

Two methods are outlined to determine the displacement C/D ratios, r_{bd}. The value, r_{bd}, is the lesser of the values calculated using the following two methods.

Method 1:

$$r_{bd} = \frac{N(c)}{N(d)} = 1.27 \qquad \text{(SR Manual, Eq. A-3)}$$

Where
N(c) = the support length provided = 25.48 in (from the bridge drawing)
N(d) = the minimum support length (see Sect. A.3 of SR Manual) = 12 + 0.03L + 0.12H

Where
L = Length, in ft, of the bridge deck from the support under consideration to the adjacent expansion joint or to the end of the bridge deck = 2 $\times$ 104 ft = 208 ft (use length of the entire bridge deck)
H = Height, in ft, of columns supporting the bridge deck = 15.58 ft (from top of footing to the center of bent cap)
Hence, N(d) = 20.11 in

Method 2:

$$r_{bd} = \frac{\Delta_s(c) - \Delta_i(d)}{\Delta_{eq}(d)} = 37.30 \qquad \text{(SR Manual, Eq. A-4)}$$

Where
$\Delta_s(c)$ = available support length for movement, $\Delta_s(c)$ = 20.48 in
$\Delta_i(d)$ = maximum possible movement resulting from temperature, shrinkage, and creep shortening. Because field measurements of available capacity in older bridge are used for $\Delta_s(c)$, only temperature effects need to be considered. Hence, $\Delta_i(d) = \alpha L \Delta T$ = 0.30 in (assumed temperature change of 20 degree)
$\Delta_{eq}(d)$ = maximum calculated relative displacement due to earthquake loading for the load cases described in section 2.2, $\Delta_{eq}(d)$ = 0.54 in (from SAP 2000, using Response Spectral Analysis)

Thus, rbd is equal to 1.27 from Method 1. Since, r_{bd} is greater than 1.0, support lengths of the expansion joints and/or bearings are adequate.

4.4.2 Force C/D Ratios

The force C/D ratio of joints and/or bearing can be determined as:

$$r_{bf} = \frac{V_b(c)}{V_b(d)} = 1.38 \qquad \text{(SR Manual, Eq. A-5)}$$

Where

$V_b(c)$ = nominal ultimate capacity of expansion joints and/or bearings, $V_b(c) = \mu R_s$ = 18.56 kips. For this bridge, the bearing type is elastometric. The coefficient of friction, μ, for elastomeric type bearing is assumed to be 0.6. R_s is the average vertical reaction at supports due to self-weight of superstructure, 30.94 kips (SAP 2000).

$V_b(d)$ = Seismic force acting on joints and/or bearings = elastic force determined from analysis or 20% of R_s, whichever is larger = 13.41 kips (SAP 2000).

Since, r_{bf} is greater than 1.0, the joint and/or bearing capacity is adequate.

4.5 Capacity/Demand Ratios for Column and Footing

4.5.1 Column Force C/D Ratio

The column force ratio can be determined as:

$$r_{ec} = \frac{M_c(c)}{M_c(d)} = 2.20$$

Where

$M_c(c)$ = nominal capacity of column = 369.92 kip-ft (see Figure 12 for column cross section)

$M_c(d)$ = elastic moment determined from analysis using CQC method = 167.78 kip-ft (SAP 2000)

Since, r_{ec} is greater than 1.0, strengthening or retrofitting of columns is not required.

4.5.2 Footing Force C/D Ratio

The footing force ratio can be determined as:

$$r_{ef} = \frac{M_f(c)}{M_f(d)} = 2.72$$

Where

$M_f(c)$ = nominal capacity of footing = 551.27 kip-ft (also see Figure 13)

$M_f(d)$ = elastic force determined from the analysis = 202.71 kip-ft (SAP 2000)

Since, both r_{ec} and r_{ef} are greater than 0.8, Case I is the proper designation according to the SR Manual, Section A.5. As a result, C/D ratios of anchorage and splice of columns should be determined. However, in order to describe the seismic evaluation method in detail, the calculating procedure of confinement C/D ratio of column is also included in this example.

4.5.3 Anchorage Length C/D Ratio

The following terms must first be calculated before determining the anchorage length ratio:

$l_a(c)$ = effective anchorage length of longitudinal reinforcement = 35 in (Bent Cap) & 85 in (Footing)

$l_a(d)$ = required effective anchorage length of longitudinal reinforcement, for straight anchorage, is larger of $\frac{k_s d_b}{(1+2.5c/d_b + k_{tr})\sqrt{f_c'}}$ = 3.41 in (SR Manual, Eq. A-6), and $30d_b$ = 26.25 in (Control)

Where

k_s = constant of reinforcing steel = 1875

d_b = nominal longitudinal bar diameter = 0.875 in (# 7 rebar shown in drawing)

f_c' = ultimate concrete compression strength = 3000 psi

c = clear concrete cover = 2.5 in

k_{tr} = conservatively assumed = 0.63

In both cases $l_a(c)$ is greater than $l_a(d)$, the anchorage length C/D ratios are 1.0 for bent cap and footing, according to Section A5.1 of the SR Manual.

4.5.4 Splices Length C/D Ratio

The following terms must first be calculated before determining the anchorage length ratio:

$A_{tr}(c)$ = area of transverse reinforcement = 0.20 in^2 (the clear spacing between spliced bars, 10 in, is greater than $4d_b$, 3.5 in)

$A_{tr}(d)$ = the minimum area of transverse reinforcement required, calculated per

$$A_{tr}(d) = \frac{sf_y}{l_s f_{yt}} A_b = 0.21 \text{ in}^2 \qquad \text{(SR Manual, Eq. A-14)}$$

Where

s = spacing of transverse reinforcement = 12 in

l_s = splice length = 35 in (shown in drawing)

f_y = yield stress of the longitudinal reinforcement = 20,000 psi (shown in drawing)

f_{yt} = yield stress of the transverse reinforcement = 20,000 psi (shown in drawing)

A_b = area of the spliced bar = 0.60 in^2 (# 7 rebar shown in drawing)

Because splice located in a zone of yielding, and s > 6 in, Case A is the proper designation according to the SR Manual, Section A.5.2. As a result, C/D ratio of splice length, r_{cs}, is smaller of $r_{cs} = \frac{A_{tr}(c)}{A_{tr}(d)}\left[\frac{\left(\frac{6}{s}\right)l_s}{\left(\frac{1860}{\sqrt{f_c'}}\right)d_b}\right]r_{ec}$ = 1.26 (SR Manual, Eq. A-15) and $\frac{A_{tr}(c)}{A_{tr}(d)}r_{ec}$ = 2.14, and need not taken as less than 0.75 r_{ec} = 1.65 (Control).

4.5.5 Shear Strength C/D Ratio

Column shear failure will occur when shear demand exceeds shear capacity. According to the SR Manual, the sample columns may experience flexure yielding, as the column force ratios (r_{ec}) are less than 1.0. For this particular scenario, shear strength C/D ratio must be identified and determined from one of the three cases presented (see Figure 9). The following terms must first be calculated:

$V_e(d)$ = elastic shear demand from analysis = 70.197 kips (SAP 2000)
$V_u(d)$ = maximum shear demand due to plastic hinging = $1.3\Sigma M_u/L_c$ = 156.096 kips

where
L_c = unsupported length of column
M_u = column moment at the location where shear strength is considered

$V_i(c)$ = initial shear resistance of the undamaged column (AASHTO Section 8.16.6)

$$= 3.5\sqrt{f_c'}\left(0.8A_g\right) + A_v f_y \frac{d}{s} + 0.2P = 280.012 \text{ kips}$$

$V_f(c)$ = final shear resistance of the damaged column (Section A.5.3 of SR Manual)

$$= 2.0\sqrt{f_c'}\left(A_c\right) + A_v f_y \frac{d}{s} + 0.2P = 173.381 \text{ kips}$$

where
A_c = concrete core area confined by transverse reinforcement
A_g = gross cross section of column
A_v = leg area of transverse reinforcement
d = effective length of column cross section
s = spacing of transverse reinforcement
f_y = yield strength of transverse reinforcement
P = applied axial load on the column

Since $r_{cv} > 1.0$, this is Case D as specified in the SR Manual. For Case D, the C/D ratio for column shear, r_{cv}, is:

$$r_{cv} = \frac{V_i(c)}{V_e(d)} = 3.99 \qquad \text{(SR Manual, Eq. A-17)}$$

Since r_{cv} is greater than 1.0, the column possesses adequate shear strength.

4.5.6 Confinement C/D Ratio

Inadequate transverse confinement reinforcement will cause rapid loss of flexural capacity due to buckling of the main reinforcement and crushing of the concrete in compression. The confinement C/D ratio of transverse reinforcement shall be determined as:

$$r_{cc} = \mu r_{ec} \qquad \text{(SR Manual, Eq. A-21)}$$

where

$$\mu = 2 + 4\left(\frac{k_1 + k_2}{2}\right)k_3 \qquad \text{(SR Manual, Eq. A-22)}$$

where

$$k_1 = \frac{\rho(c)}{\rho(d)\left(0.5 + \dfrac{1.25P_c}{f_c'A_g}\right)} \le 1$$

$k_2 = 6d_b \le 1$ or $0.2b_{min}/s \le 1$, whichever is smaller

k_3 = effectiveness of transverse bar anchorage. This will be 1.0 unless transverse bars are poorly anchored.

$\rho(c)$ = volumetric ratio of existing transverse reinforcement

$\rho(d)$ = required volumetric ratio of transverse reinforcement determined in accordance with the provisions of section 7.6 Division I-A of the AASHTO Specifications

P_c = axial compressive load on the column

f_c' = compressive strength of the concrete

A_g = gross area of column

s = spacing of transverse steel

d_b = diameter of longitudinal reinforcement

b_{min} = minimum width of the column cross section = 36 in (from drawing)

For this particular case, if μ is assumed to be 2 (most conservative), the confinement ratio, $r_{cc} = 2 \times 2.20 = 4.40$.

Since r_{cc} is greater than 1.0, it can be concluded that the confinement provided for the columns is adequate.

4.5.7 Footing Rotation C/D Ratio

Since r_{ef} is greater than 0.8, the footing rotation and/or yielding ratio will not be investigated. However, in order to describe the seismic evaluation method, the calculating procedure of footing rotation C/D ratio is also presented here.

Column footings may rotate and/or yield before columns can yield. This can occur due to any one of several failure modes. The amount of rotation and/or yielding allowed in the footing will depend on the mode of failure. The seismic C/D ratio for these type of footing failures, r_{fr}, are calculated as follows:

$$r_{fr} = \mu r_{ef} \qquad \text{(SR Manual, Eq. A-23)}$$

where

μ = the ductility indicator, the most conservative value is 1.0.

Hence, $r_{fr} = \mu r_{ef} = 1.0 \times 2.72 = 2.72$

Since r_{fr} is greater than 1.0, it can be concluded that the footing can not rotate and/or yield before columns can yield.

5. RESULTS OF DETAILED SEISMIC EVALUATION ON SELECTED BRIDGES

Based on the procedure described above, the detailed seismic evaluations were carried out on all the 17 selected bridges. The results of every bridge are summarized and tabulated in Table 8 through Table 19. Summary of all 17 bridges is provided in Table 20. Deficiencies of bridges under the projected 250-year seismic event are highlighted in Table 21.

6. SUMMARY

The five parkways in Western Kentucky lie near the New Madrid Seismic Zone (NMSZ). The zone remains active with an average of nearly 200 seismic events recorded annually. Due to their locality and socioeconomics factors, these parkways are designated as high priority and emergency routes in Western Kentucky, which must remain functional and operational after an earthquake event.

The primary objective of this study is to investigate the structural integrity of selected highway bridges on and over the parkways in Western Kentucky by performing detailed seismic evaluation of bridges that are deemed susceptible to damage during a major earthquake event.

There are 389 highway bridges on and over the parkways in Western Kentucky. To select and prioritize these bridges for detailed seismic evaluation, a Seismic Rating System was used. The rating and ranking of bridges employed by this system depended on several factors: structural vulnerability, seismic and geotechnical hazards, and socioeconomic factors. Included in this process was the collection of structural inventory and site investigations of all bridges. The ranks of these bridges range from 0 (no or minimal risk) to 75, based on a scale of 100. A total of 17 bridges, parallel bridges included, ranked 35 or higher, were selected. The average rank (R) of the 17 bridges was 48. The selected bridges were constructed in the 1960s where seismic designs were not taken into consideration. The selected bridges have different construction types: reinforced and prestress concrete, and steel composites bridges.

Selected bridges were evaluated using the Capacity/Demand (C/D) ratio method proposed in the Seismic Retrofitting Manual for Highway Bridges (Buckle and Friedland, 1995). The evaluation of the expansion joints and bearings, and columns and footings, is carried out in this report. The abutments of bridges on the Western Kentucky Parkways are investigated and presented in a separate report, whose focus is on the seismic or soil stability of the abutments and the potential of liquefaction of the underlying soil.

In this report, the detailed seismic evaluation includes the creation of a 3-dimensional finite element computer model for each of the selected bridges, and a dynamic analysis using a projected 250-year seismic time history.

Majority of bridges evaluated in this process have some forms of deficiencies and required retrofit. Of the bridges evaluated, four bridges [30-9005-B00060, 30-9005-B00061 and 42-9003-B00157 (P)] with ranks of 38.0 and 35.1 possess no seismic deficiency.

Table 1: Classification of Seismic Performance Category (SPC)
(Seismic Retrofitting Manual, Table 1)

Acceleration Coefficient	Importance Classification	
	Essential	Standard
$A \leq 0.09$	B	A
$0.09 < A \leq 0.19$	C	B
$0.19 < A \leq 0.29$	C	C
$0.29 < A$	D	C

Table 2: Soil Profile Type and Site Coefficient (S)
(Seismic Retrofitting Manual, Table 3)

Soil Type	Soil Profile	Site Coefficient
I	Rock or stiff soils Soil depth less than 60 m (200 ft)	1.0
II	Stiff cohesive or deep cohesionless soil Soil depth exceeds 60 m (200 ft)	1.2
III	Soft to medium stiff clays and sands Soil depth exceeds 9 m (30 ft)	1.5
IV	Soft clays or silts Soil depth exceeds 12 m (40 ft)	2.0

Table 3: Priority Bridges to Be Evaluated

No.	Parkway	County	BIN Number	SPC	Drawing Number	R
1	Purchase	Fulton	38-0051-B00012	D	16696	75.0
2			38-0307-B00015	D	16649	75.0
3			38-9003-B00053	D	16694	75.0
4			38-9003-B00053P	D		75.0
5			38-9003-B00054	D	16695	75.0
6			38-9003-B00054P	D		75.0
7			38-9003-B00055	D	16561	75.0
8			38-9003-B00055P	D		75.0
9	Purchase	Hickman	53-0094-B00050	D	16566	75.0
10			53-1529-B00056	D	16567	75.0
11			53-9003-B00068	D	16565	75.0
12	Audubon	Daviess	30-9005-B00060	C	17494	38.0
13			30-9005-B00061	C	17464	38.0
14	Purchase	Graves	42-9003-B00157	C	16527	35.1
15			42-9003-B00157P	C		35.1
16	Pennyrile	Webster	117-9004-B00071[a]	B	16858	8.4
17			117-9004-B00071P	B		8.4

Note: a) Irregular Bridge

Table 4: Capacity/Demand Ratios for Detailed Seismic Evaluation

No.	Symbol	Definition	SR Manual
1	r_{bd}	Displacement ratio for bearing/joint	Sections 3.6.2 & A.4.2
2	r_{bf}	Force ratio for bearing/joint	Sections 3.6.2 & A.4.3
3	r_{ec}	Force ratio for column	Sections 3.6.3 & A.5
4	r_{ef}	Force ratio for footing	Sections 3.6.3 & A.5
5	$r_{ca(cap)}$	Anchorage length ratio for bent cap	Sections 3.6.3 & A.5.1
6	$r_{ca(footing)}$	Anchorage length ratio for footing	Sections 3.6.3 & A.5.1
7	r_{cs}	Splice length ratio for column	Sections 3.6.3 & A.5.2
8	r_{cv}	Shear ratio for column	Sections 3.6.3 & A.5.3
9	r_{cc}	Confinement ratio for transverse reinforcement	Sections 3.6.3 & A.5.4
10	r_{fr}	Footing rotation and/or yielding ratio	Sections 3.6.3 & A.5.5

Table 5: Footing Ductility Indicators
(Seismic Retrofitting Manual, Table 8)

Type of Footing	Factor Limiting the Capacity	μ
Spread Footing	Soil Bearing Failure	4
	Reinforcing Steel Yielding in the Footing	4
	Concrete Shear or Tension in the Footing	1
Pile Footing	Pile Overload (Compression or Tension)	3
	Reinforcing Steel Yielding in the Footing	4
	Pile Pullout at Footing	2
	Concrete Shear or Tension in the Footing	1
	Flexural Failure of Piling	4
	Shear Failure of Piling	1

Table 6: Dimensional Parameters of Selected Bridges

BIN	Number of Spans	Number of Girders	Number of Columns	Span Length / ft.	Girder Spacing / ft.	Column Spacing / ft.	Column Height / ft.	Skew Angle
38-0051-B00012	2	9	7	80-80	9	12	19	$4^{o}23'$
38-0307-B00015	2	7	5	80-80	8	12	19	-4^{o}
38-9003-B00053 38-9003-B00053P	3	5	3	49-51-49	8.75	17.5	27	0^{o}
38-9003-B00054 38-9003-B00054P	3	5	3	45-51-45	9	18	26	$21^{o}55'$
38-9003-B00055 38-9003-B00055P	3	5	3	67-92-72	8	16	29-36	$-25^{o}52'$
53-0094-B00050	2	4	3	88-88	8	12	19	25^{o}
53-1529-B00056	2	4	3	80-80	8	12	18.5	$8^{o}58'$
53-9003-B00068	2	4	3	91-91	8	12	18.5	30^{o}
30-9005-B00060	2	4	3	104-104	9	13.5	18.8	-27^{o}
30-9005-B00061	2	5	3	108-108	9	18	19	$-31^{o}17$
42-9003-B00157 42-9003-B00157P	4	5	9	48.5-53.25 53.25-48.5	8.75	4.38	22	0
117-9004-B00071 117-9004-B00071P	7	4	3/7	48.5- 5*53.3- 48.5	8	12	53-56	0

Table 7: Identification of Time History Response Spectra for Selected Bridges

No.	Parkway	County	BIN	Identification	Time Step / s	Number of Steps
1	Purchase	Fulton	38-0051-B00012	0.40g-1	0.005	4100
2			38-0307-B00015	0.40g-1	0.005	4100
3			38-9003-B00053	0.40g-1	0.005	4100
4			38-9003-B00053P	0.40g-1	0.005	4100
5			38-9003-B00054	0.40g-1	0.005	4100
6			38-9003-B00054P	0.40g-1	0.005	4100
7			38-9003-B00055	0.40g-1	0.005	4100
8			38-9003-B00055P	0.40g-1	0.005	4100
9	Purchase	Hickman	53-0094-B00050	0.40g-1	0.005	4100
10			53-1529-B00056	0.40g-1	0.005	4100
11			53-9003-B00068	0.40g-1	0.005	4100
12	Audubon	Daviess	30-9005-B00060	0.15g-1	0.005	4100
13			30-9005-B00061	0.15g-1	0.005	4100
14	Purchase	Graves	42-9003-B00157	0.19g-1	0.005	4100
15			42-9003-B00157P	0.19g-1	0.005	4100
16	Pennyrile	Webster	117-9004-B00071	0.09g-2	0.005	4100
17			117-9004-B00071P	0.09g-2	0.005	4100

Table 8: C/D Ratios for the US 51 Bridge over Purchase Parkway

1. Title Summary of the detailed seismic evaluation of bridge No. 38-0051-B00012 US 51 bridge over Purchase Parkway (Fulton Country) 80 ft – 80 ft continuous RC box girder span		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.28	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	3.97	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.29	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	0.62	*Capacity is not adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	0.62	*Capacity is not adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A* *
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	0.37	*Capacity is not adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	0.99	*Capacity is not adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	0.29	*Capacity is not adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	2.48	*Capacity is adequate*

*: There is no steel splice at bent cap.

Table 9: C/D Ratios for the KY 307 Bridge over Purchase Parkway

1. Title Summary of the detailed seismic evaluation of bridge No. 38-0307-B00015 KY 307 bridge over Purchase Parkway (Fulton Country) 80 ft – 80 ft continuous RC box girder spans		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.28	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	5.50	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.36	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	0.78	*Capacity is not adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	0.78	*Capacity is not adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A**
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	0.40	*Capacity is not adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	1.17	*Capacity is adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	1.51	*Capacity is adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	3.13	*Capacity is adequate*

*: There is no steel splice at bent cap.

Table 10: C/D Ratios for the Purchase Parkway Bridge over KY 116

1. Title Summary of the detailed seismic evaluation of bridge No. 38-9003-B00053(P) Purchase Parkway bridge over KY 116 (Fulton Country) 48 ft – 51 ft – 48 ft continuous pre-stressed R.C.D.G. spans		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.26	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	1.56	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.47	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	0.23	*Capacity is not adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.00	*Capacity is adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A*[*]
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	0.62	*Capacity is not adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	1.50	*Capacity is adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	2.37	*Capacity is adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	0.91	*Capacity is not adequate*

*: There is no steel splice at bent cap.

Table 11: C/D Ratios for the Purchase Parkway Bridge over KY 166

1. Title Summary of the detailed seismic evaluation of bridge No. 38-9003-B00054(P) Purchase Parkway bridge over KY 166 (Fulton Country) 44 ft – 51 ft – 44 ft continuous pre-stressed R.C.D.G. spans		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.30	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	0.90	*Capacity is not adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.23	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	0.15	*Capacity is not adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.00	*Capacity is adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A**
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	0.30	*Capacity is not adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	0.74	*Capacity is not adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	1.15	*Capacity is adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	0.58	*Capacity is not adequate*

*: There is no steel splice at bent cap.

Table 12: C/D Ratios for the Purchase Parkway Bridge over I.C.R.R

1. Title Summary of the detailed seismic evaluation of bridge No. 38-9003-B00055(P) Purchase Parkway bridge over I.C.R.R (Fulton Country) 66'-92'-71'+41'+70'-77'-60' pre-stressed R.C.D.G. spans		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.19	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	1.19	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.38	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	0.33	*Capacity is not adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.00	*Capacity is adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A**
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	0.56	*Capacity is not adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	1.21	*Capacity is adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	1.75	*Capacity is adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	0.67	*Capacity is not adequate*

*: There is no steel splice at bent cap.

Table 13: C/D Ratios for the KY 94 Bridge over Purchase Parkway

1. Title Summary of the detailed seismic evaluation of bridge No. 53-0094-B00050 KY 94 bridge over Purchase Parkway (Hickman Country) 88 ft – 88 ft continuous RC box girder spans		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.33	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	5.51	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.35	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	0.55	*Capacity is not adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	0.55	*Capacity is not adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A**
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	0.39	*Capacity is not adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	1.14	*Capacity is adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	1.34	*Capacity is adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	1.09	*Capacity is adequate*

*: There is no steel splice at bent cap.

Table 14: C/D Ratios for the KY 1529 Bridge over Purchase Parkway

1. Title Summary of the detailed seismic evaluation of bridge No. 53-1529-B00056 KY 1529 bridge over Purchase Parkway (Hickman Country) 80 ft – 80 ft continuous RC box girder spans		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.30	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	4.42	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.27	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	0.39	*Capacity is not adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	0.39	*Capacity is adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A* *
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	0.30	*Capacity is not adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	0.88	*Capacity is not adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	0.76	*Capacity is not adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	1.16	*Capacity is adequate*

*: There is no steel splice at bent cap.

Table 15: C/D Ratios for the Holland Lane Bridge over Purchase Parkway

1. Title Summary of the detailed seismic evaluation of bridge No. 53-9003-B00068 Holland lane bridge over Purchase Parkway (Hickman Country) 91 ft – 91 ft continuous RC box girder spans		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.07	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	0.85	*Capacity is not adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.28	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	0.39	*Capacity is not adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	0.39	*Capacity is not adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A* *
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	0.31	*Capacity is not adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	0.91	*Capacity is not adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	0.28	*Capacity is not adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	0.79	*Capacity is not adequate*

*: There is no steel splice at bent cap.

Table 16: C/D Ratios for the Lyddane Br. Rd. Bridge over Audubon Parkway

1. Title Summary of the detailed seismic evaluation of bridge No. 30-9005-B00060 Lyddane Br. Rd. bridge over Audubon Parkway (Daviess Country) 104 ft – 104 ft continuous comp. welded girder spans		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.27	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	1.38	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	2.20	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	2.72	*Capacity is adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.00	*Capacity is adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A* *
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	1.65	*Capacity is adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	6.95	*Capacity is adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	3.99	*Capacity is adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	2.72	*Capacity is adequate*

*: There is no steel splice at bent cap.

Table 17: C/D Ratios for the KY 279 Bridge over Audubon Parkway

1. Title Summary of the detailed seismic evaluation of bridge No. 30-9005-B00061 KY 279 bridge over Audubon Parkway (Daviess Country) 108 ft – 108 ft continuous comp. welded girder spans		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.39	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	1.20	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	2.07	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	2.33	*Capacity is adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.00	*Capacity is adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A*[*]
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	1.55	*Capacity is adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	6.87	*Capacity is adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	2.48	*Capacity is adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	2.33	*Capacity is adequate*

*: There is no steel splice at bent cap.

Table 18: C/D Ratios for the Purchase Parkway Bridge over Mayfield Creek

<table>
<tr><td colspan="3">1. Title

Summary of the detailed seismic evaluation of bridge No. 42-9003-B00157(P)
Purchase Parkway bridge over Mayfield Creek (Graves Country)
48'-53'-53'-48' pre-stressed R.C.D.G. spans</td></tr>
<tr><td colspan="3">CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS</td></tr>
<tr><td colspan="2"></td><td>Comment:</td></tr>
<tr><td>2. Displacement Capacity/Demand Ratio
r_{bd}</td><td>1.20</td><td>Capacity is adequate</td></tr>
<tr><td>3. Force Capacity/Demand Ratio
r_{bf}</td><td>1.32</td><td>Capacity is adequate</td></tr>
<tr><td colspan="3">CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING</td></tr>
<tr><td colspan="2"></td><td>Comment:</td></tr>
<tr><td>4. Force Capacity/Demand Ratio for Column
r_{ec}</td><td>-</td><td>N/A*</td></tr>
<tr><td>5. Force Capacity/Demand Ratio for Footing
r_{ef}</td><td>-</td><td>N/A*</td></tr>
<tr><td>6. Anchorage Capacity/Demand Ratio at Bent Cap
$r_{ca(Cap)}$</td><td>-</td><td>N/A*</td></tr>
<tr><td>7. Anchorage Capacity/Demand Ratio at Footing
$r_{ca(Footing)}$</td><td>-</td><td>N/A*</td></tr>
<tr><td>8. Splice Capacity/Demand Ratio at Bent Cap
$r_{cs(Cap)}$</td><td>-</td><td>N/A*</td></tr>
<tr><td>9. Splice Capacity/Demand Ratio at Footing
$r_{cs(Footing)}$</td><td>-</td><td>N/A*</td></tr>
<tr><td>10. Transverse Confinement Capacity/Demand Ratio
r_{cc}</td><td>-</td><td>N/A*</td></tr>
<tr><td>11. Column Shear Capacity/Demand Ratio
r_{cv}</td><td>-</td><td>N/A*</td></tr>
<tr><td>12. Footing Rotation and/or Yielding Ratio
r_{fr}</td><td>-</td><td>N/A*</td></tr>
</table>

*: The substructure – pier – is made up of 16’’ reinforced concrete pre-cast concrete piles.

Table 19: C/D Ratios for the Pennyrile Parkway Bridge over Deer Creek

1. Title Summary of the detailed seismic evaluation of bridge No. 117-9004-B00071 Pennyrile Parkway bridge over Deer Creek (Webster Country) 48'-53'-53'-53'-53'-53'-48' continuous pre-stressed R.C.D.G. spans		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	0.99	*Capacity is not adequate*
3. Force Capacity/Demand Ratio r_{bf}	2.15	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.76	*Capacity is not adequate*
5. Force Capacity/Demand Ratio for Footing r_{ef}	1.14	*Capacity is adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.00	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.00	*Capacity is adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	-	*N/A*[*]
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	0.67	*Capacity is not adequate*
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	2.42	*Capacity is adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	3.78	*Capacity is adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	1.14	*Capacity is adequate*

*: There is no steel splice at bent cap.

Table 20: C/D Ratios of Select Bridges on and over Parkways in Western Kentucky.

Bridge Identification Number (BIN)	Capacity/Demand (C/D) ratios of different bridge components											
	Joints and/or Bearings		Columns and/or Footing									Bridge Ranks
	r_{bd}	r_{bf}	r_{ec}	r_{ef}	r_{ca} (Cap)	r_{ca} (Footing)	r_{cs} (Cap)	r_{cs} (Footing)	r_{cc}	r_{cv}	r_{fr}	
38-0051-B00012	1.28	3.97	0.29	0.62	1.00	0.62	-	0.37	0.99	0.29	2.48	75.0
38-0307-B00015	1.28	5.50	0.36	0.78	1.00	0.78	-	0.40	1.17	1.51	3.13	75.0
38-9003-B00053 38-9003-B00053P	1.26	1.56	0.47	0.23	1.00	1.00	-	0.62	1.50	2.37	0.91	75.0
38-9003-B00054 38-9003-B00054P	1.30	0.90	0.23	0.15	1.00	1.00	-	0.30	0.74	1.15	0.58	75.0
38-9003-B00055 38-9003-B00055P	1.19	1.19	0.38	0.33	1.00	1.00	-	0.56	1.21	1.75	0.67	75.0
53-0094-B00050	1.33	5.51	0.35	0.55	1.00	0.55	-	0.39	1.14	1.34	1.09	75.0
53-1529-B00056	1.30	4.42	0.27	0.39	1.00	0.39	-	0.30	0.88	0.76	1.16	75.0
53-9003-B00068	1.07	0.85	0.28	0.39	1.00	0.39	-	0.31	0.91	0.28	0.79	75.0
30-9005-B00060	1.27	1.38	2.20	2.72	1.00	1.00	-	1.65	6.95	3.99	2.72	38.0
30-9005-B00061	1.39	1.20	2.07	2.33	1.00	1.00	-	1.55	6.87	2.48	2.33	38.0
42-9003-B00157 42-9003-B00157P	1.20	1.32	-	-	-	-	-	-	-	-	-	35.1
117-9004-B00071 117-9004-B00071P	0.99	2.15	0.76	1.14	1.00	1.00	-	0.67	2.42	3.78	1.14	8.4

Note: C/D ratio value less than 1.0 is highlighted in the table
Cells indicated with (-) represent C/D ratios are not applicable in these cases

r_{bd} = Joint/bearing displacement C/D ratio
r_{ef} = Footing force C/D ratio
r_{cc} = Transverse conferment C/D ratio
r_{bf} = Joint/bearing force C/D ratio
r_{fr} = Footing rotation/yeiding C/D ratio
r_{ec} = Column force C/D ratio
r_{cs} = Splice C/D ratio of cap or footing
rc_v = Column shear C/D ratio
r_{ca} = Anchorage C/D ratio of cap or footing

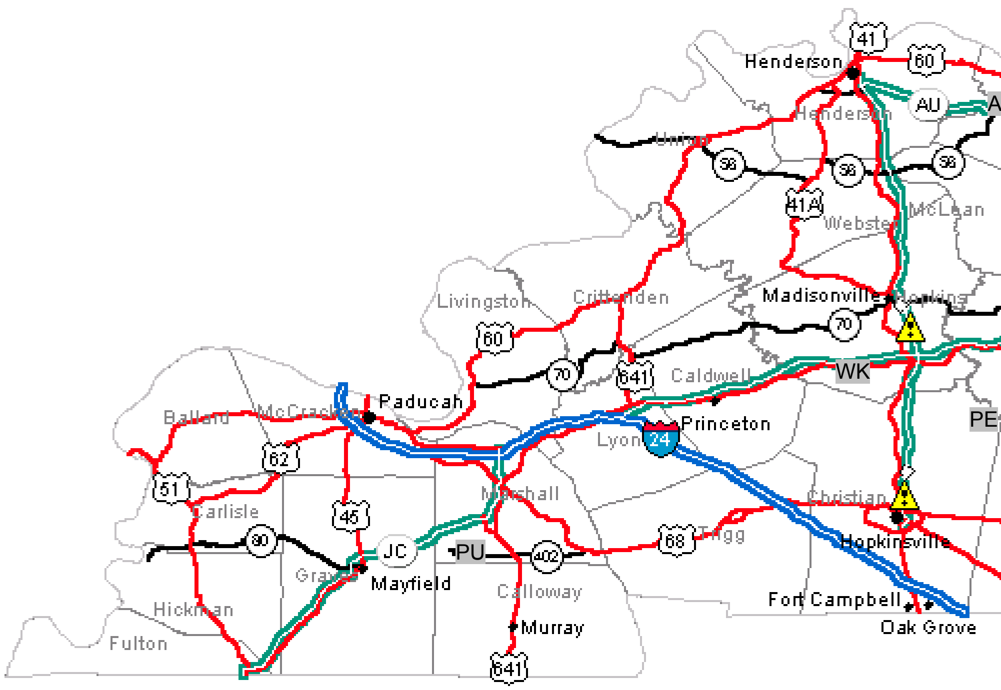
Table 21: Summary of Seismic Deficiencies of Selected Bridges

Bridge Identification Number (BIN)	Ranking	Seismic Deficiencies
38-0051-B00012	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
38-0307-B00015	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
38-9003-B00053 38-9003-B00053P	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
38-9003-B00054 38-9003-B00054P	75.0	- Bearing seat capacity - Footing flexural capacity - Column shear capacity - Column flexural capacity
38-9003-B00055 38-9003-B00055P	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
53-0094-B00050	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
53-1529-B00056	75.0	- Footing flexural capacity - Column shear capacity - Column flexural capacity
53-9003-B00068	75.0	- Bearing seat capacity - Footing flexural capacity - Column shear capacity - Column flexural capacity
30-9005-B00060	38.0	-
30-9005-B00061	38.0	-
42-9003-B00157 42-9003-B00157P	35.1	-
117-9004-B00071 117-9004-B00071P	8.4	- Bearing seat capacity - Footing flexural capacity - Column flexural capacity

Modified Mercalli Intensity Scale

INTENSITY	EFFECTS	AVE. PEAK ACCEL.
VI Strong	Felt by all. Damage slight	0.06-0.07g
VII Very Strong	Everybody runs outdoors. Considerable damage to poorly designed buildings	0.10-0.15g
VIII Destructive	Considerable damage to ordinary buildings	0.25-0.30g
IX Ruinous	Great damage to ordinary buildings	0.50-0.55g
X Disastrous	Many buildings destroyed	> 0.60g
XI Disastrous	Few, if any, structures remain standing	

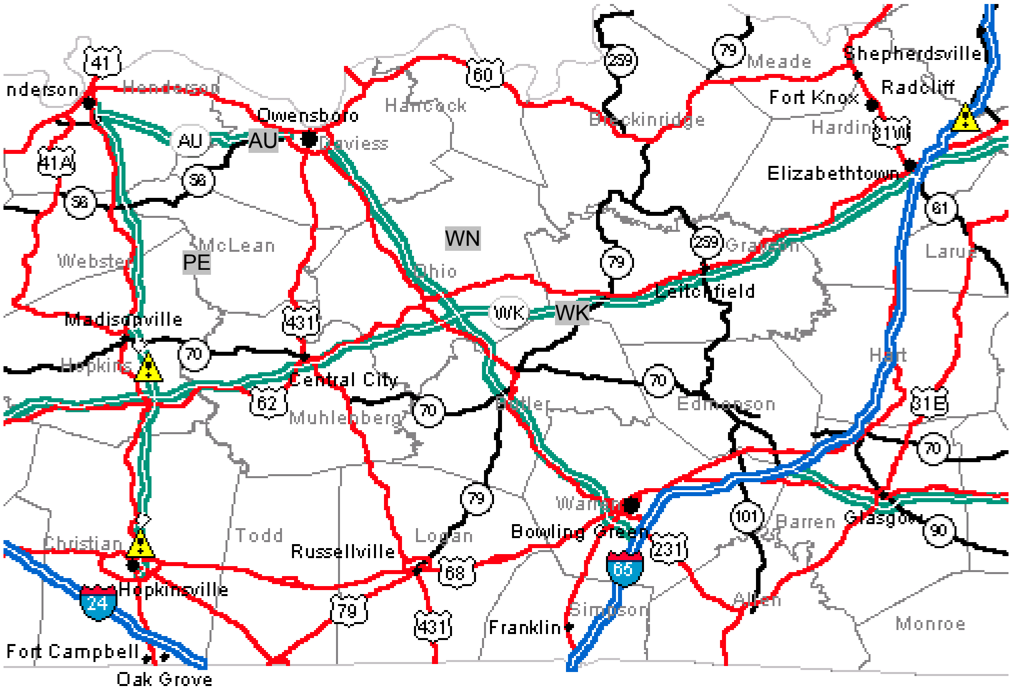
Figure 1: Isoseismal Map for the Arkansas Earthquake of December 16, 1811 (Bolt, 1993)



(a) Far West Kentucky

(Note: PU-Purchase Parkway; WK-Western Kentucky Parkway; AU-Audubon Parkway; PE-Pennyrile Parkway; WN-William Natcher Parkway)

Figure 2: The Parkways in Western Kentucky



(b) Western Kentucky

(Note: PU-Purchase Parkway; WK-Western Kentucky Parkway; AU-Audubon Parkway; PE-Pennyrile Parkway; WN-William Natcher Parkway)

Figure 2 Continued: The Parkways in Western Kentucky

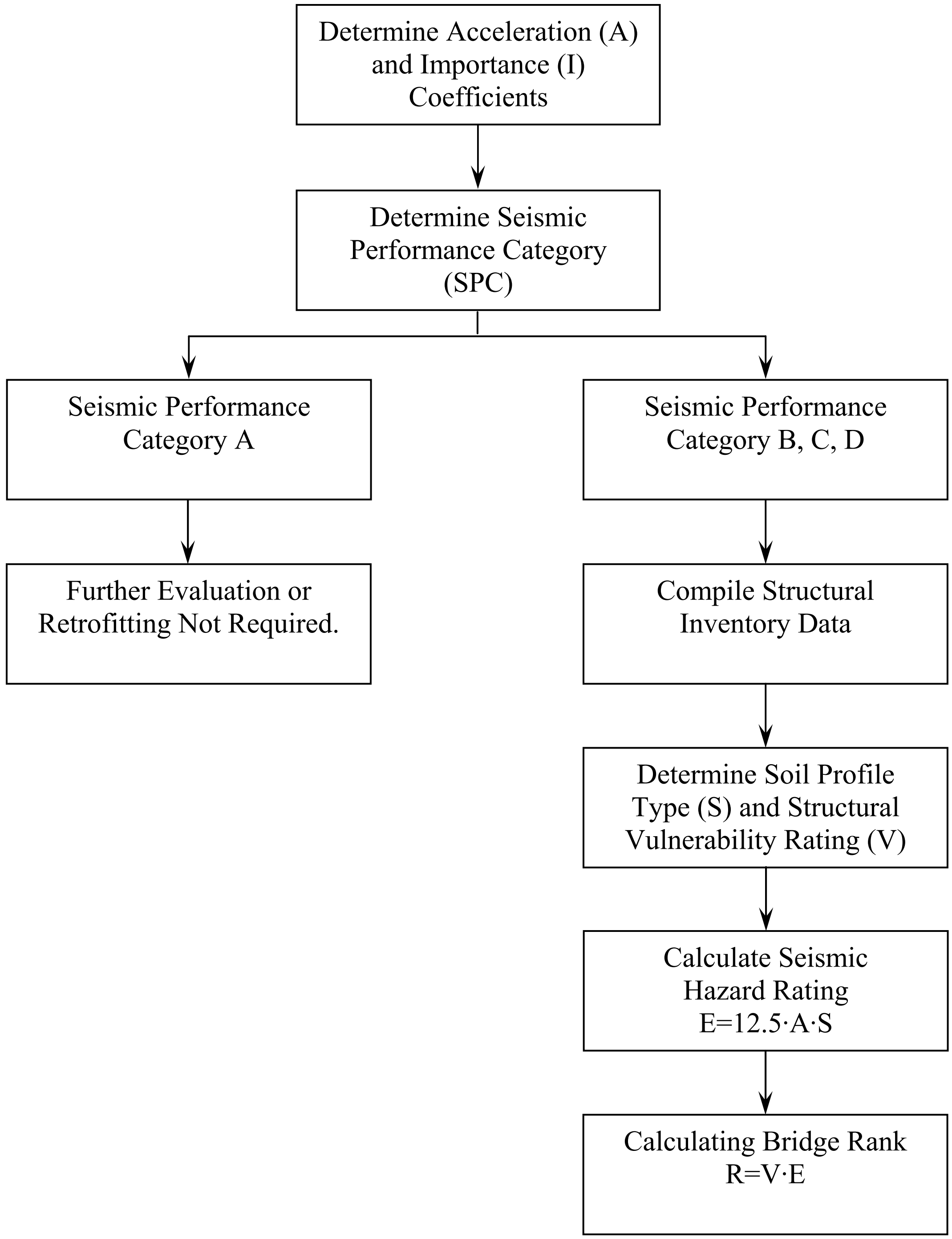


Figure 3: Seismic Ranking System
(Seismic Retrofitting Manual, Figure 6)

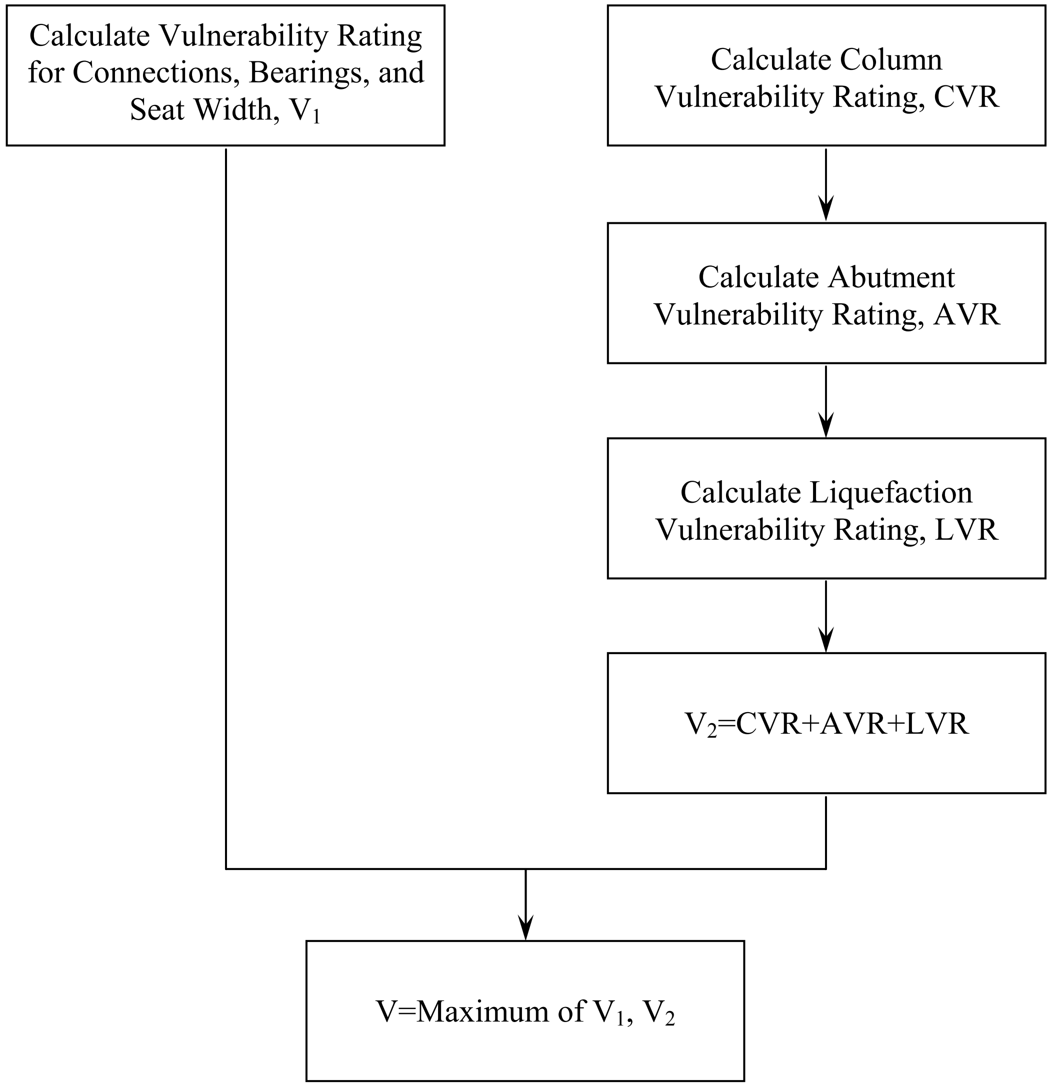


Figure 4: Flow Chart for Calculation of Bridge Vulnerability Rating (V)

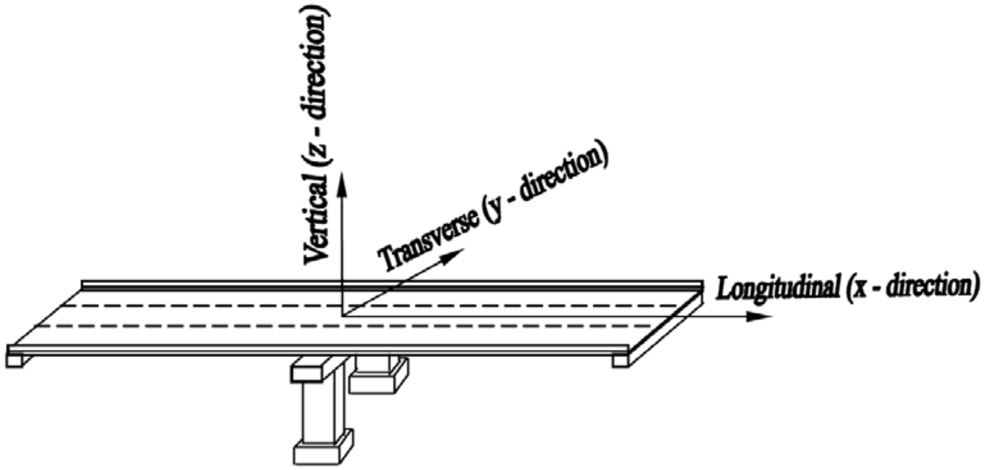


Figure 5: Longitudinal, Transverse, and Vertical Directions of a Bridge

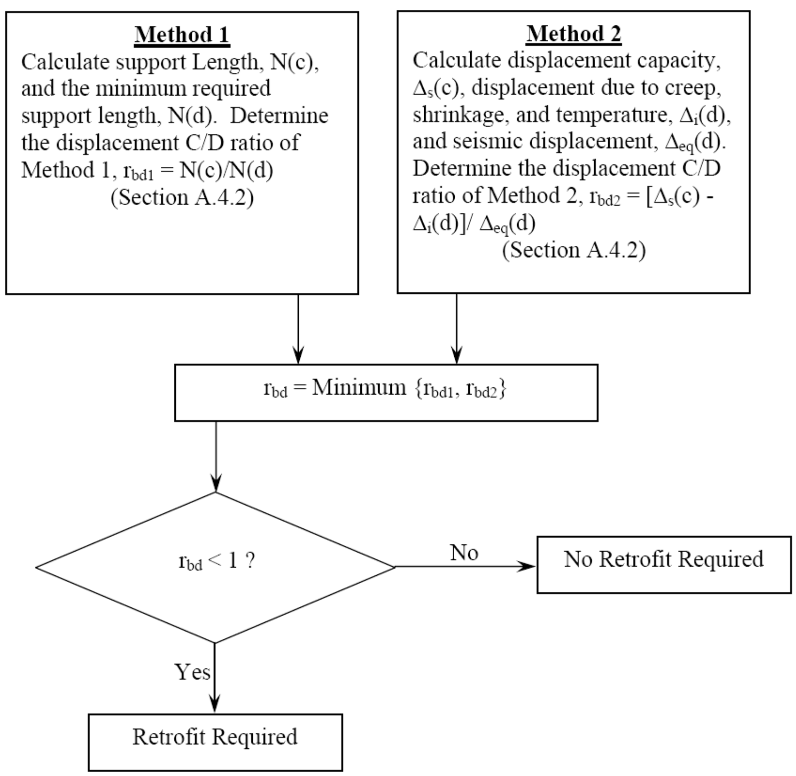
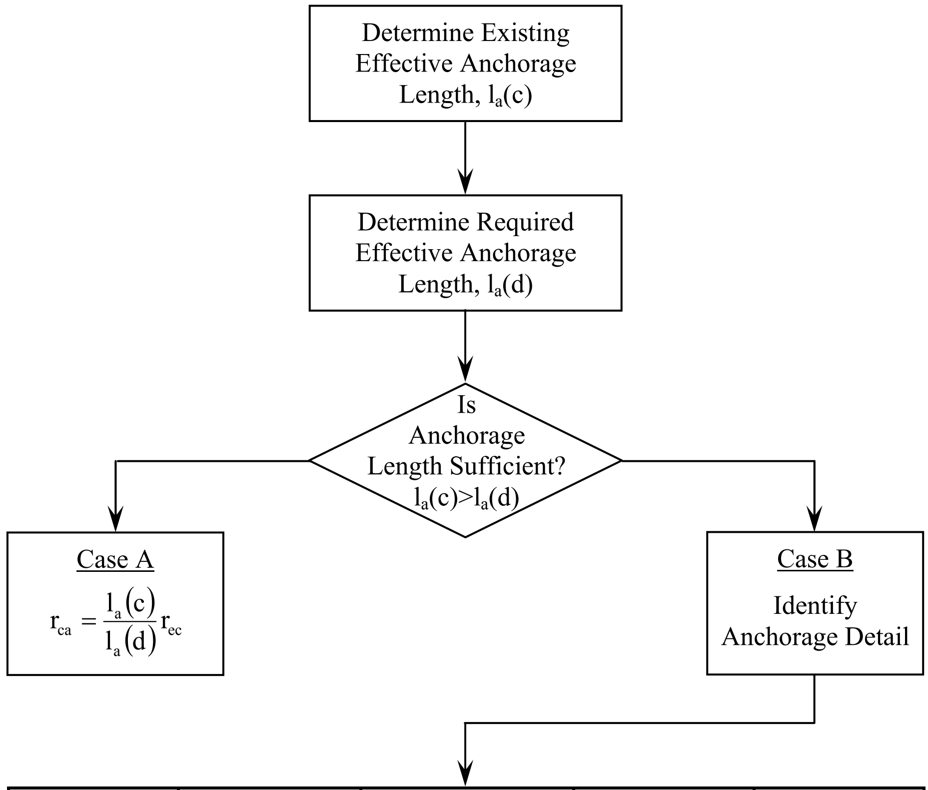


Figure 6: Displacement Capacity/Demand Ratios for Expansion Joints and/or Bearings



Detail No.	Location	Anchorage Type	Top Footing Reinforcing	C/D Ratio
1	Footing	Straight	No	$r_{ca} = r_{ef}$
2	Footing	90^o hook away from centerline	No	$r_{ca} = 1.3r_{ef}$
3	Footing	90^o hook toward centerline	No	$r_{ca} = 2.0r_{ef}$
4	Footing	Straight	Yes	$r_{ca} = 1.5r_{ef}$
5	Footing	90^o hook	Yes	1.0
6	Bent cap	—	—	1.0

Figure 7: Anchorage Capacity/Demand Ratio of Longitudinal Reinforcement
(Seismic Retrofitting Manual, Figure 78)

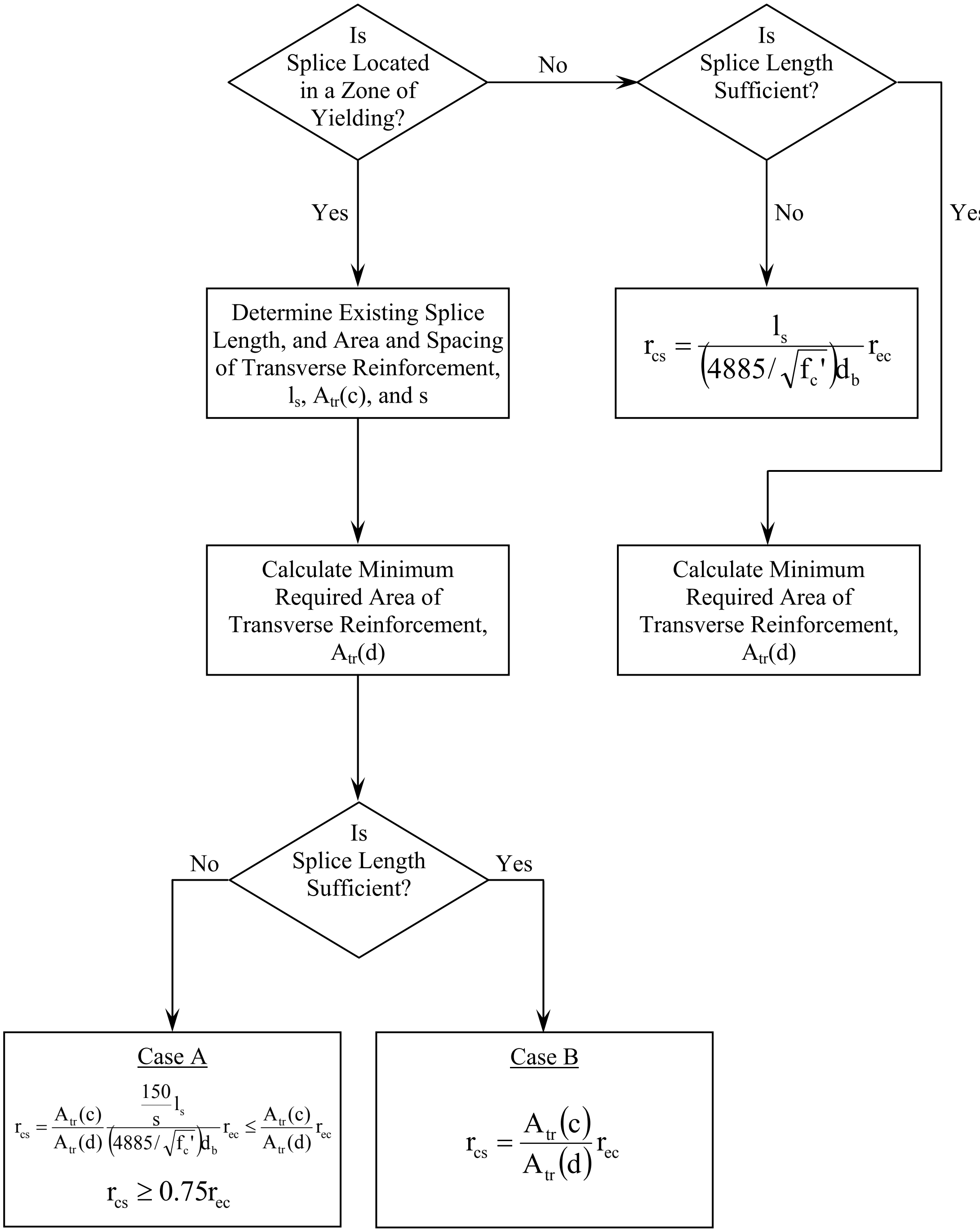


Figure 8: Procedure for Determining C/D Ratios for Splices in Longitudinal Reinforcement

(Seismic Retrofitting Manual, Figure 80)

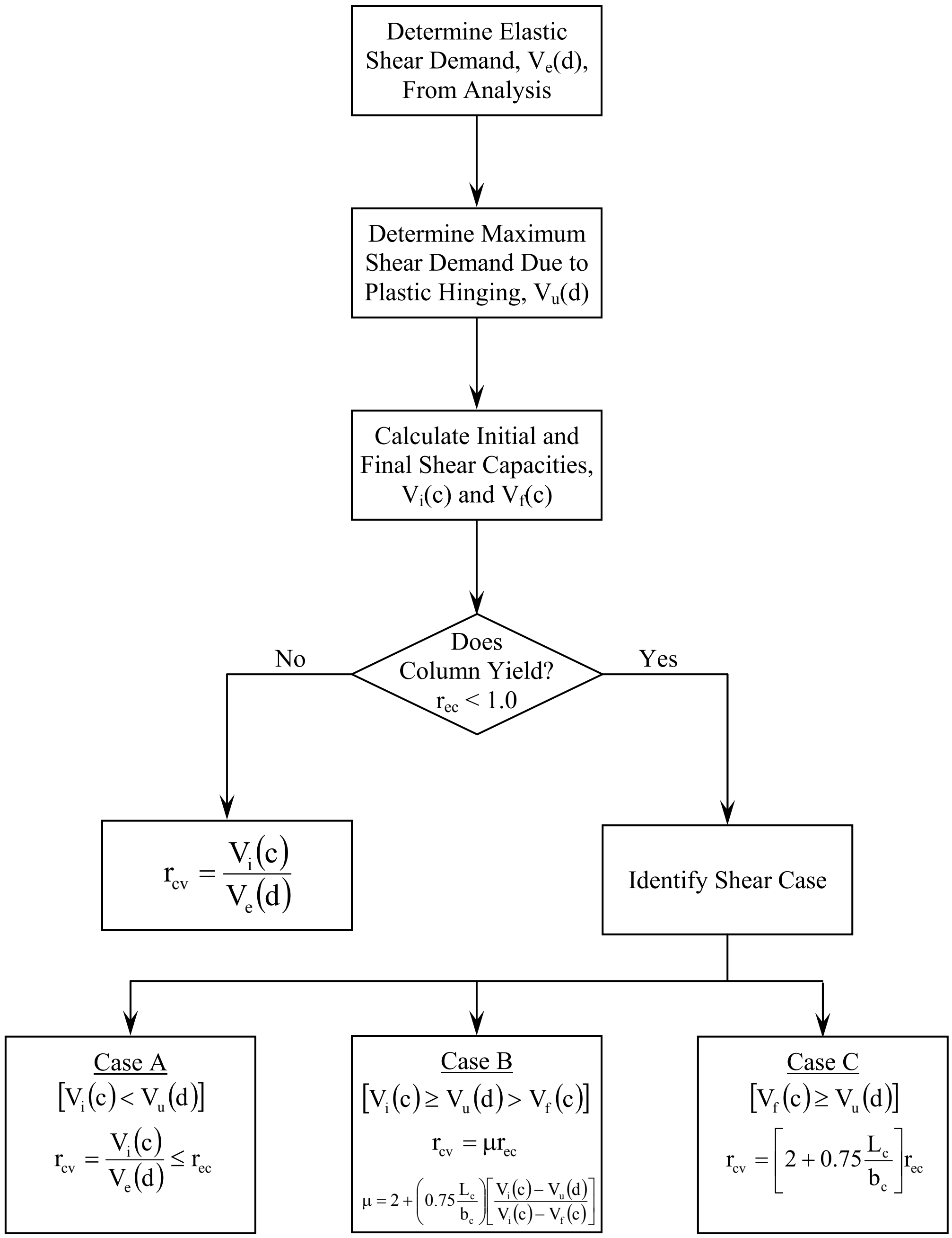


Figure 9: Procedure for Determining Capacity/Demand Ratios for Column Shear

(Seismic Retrofitting Manual, Figure 81)

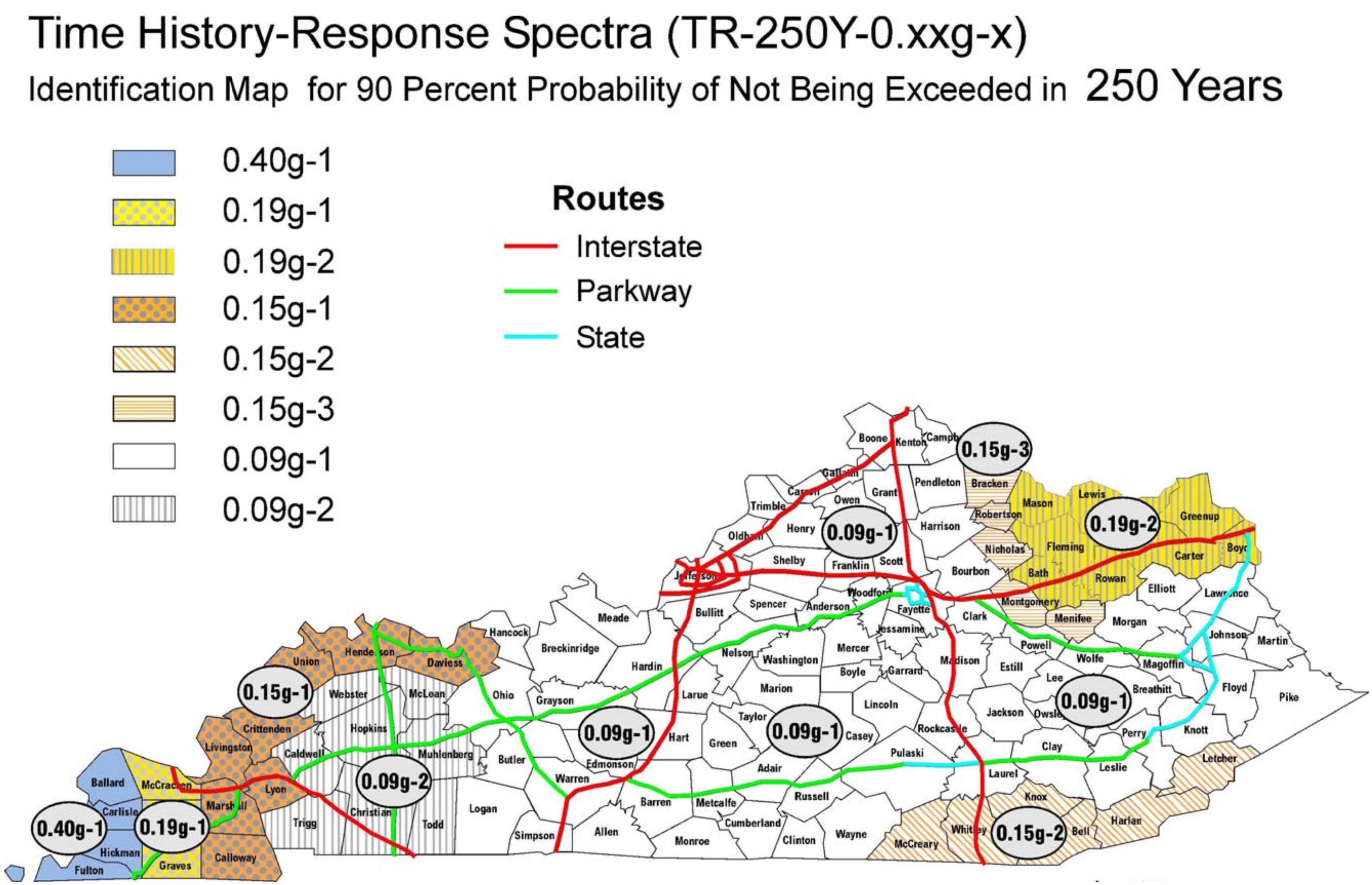


Figure 10: Time History Response Spectra Identification Map of 250-year Earthquake Event for Kentucky

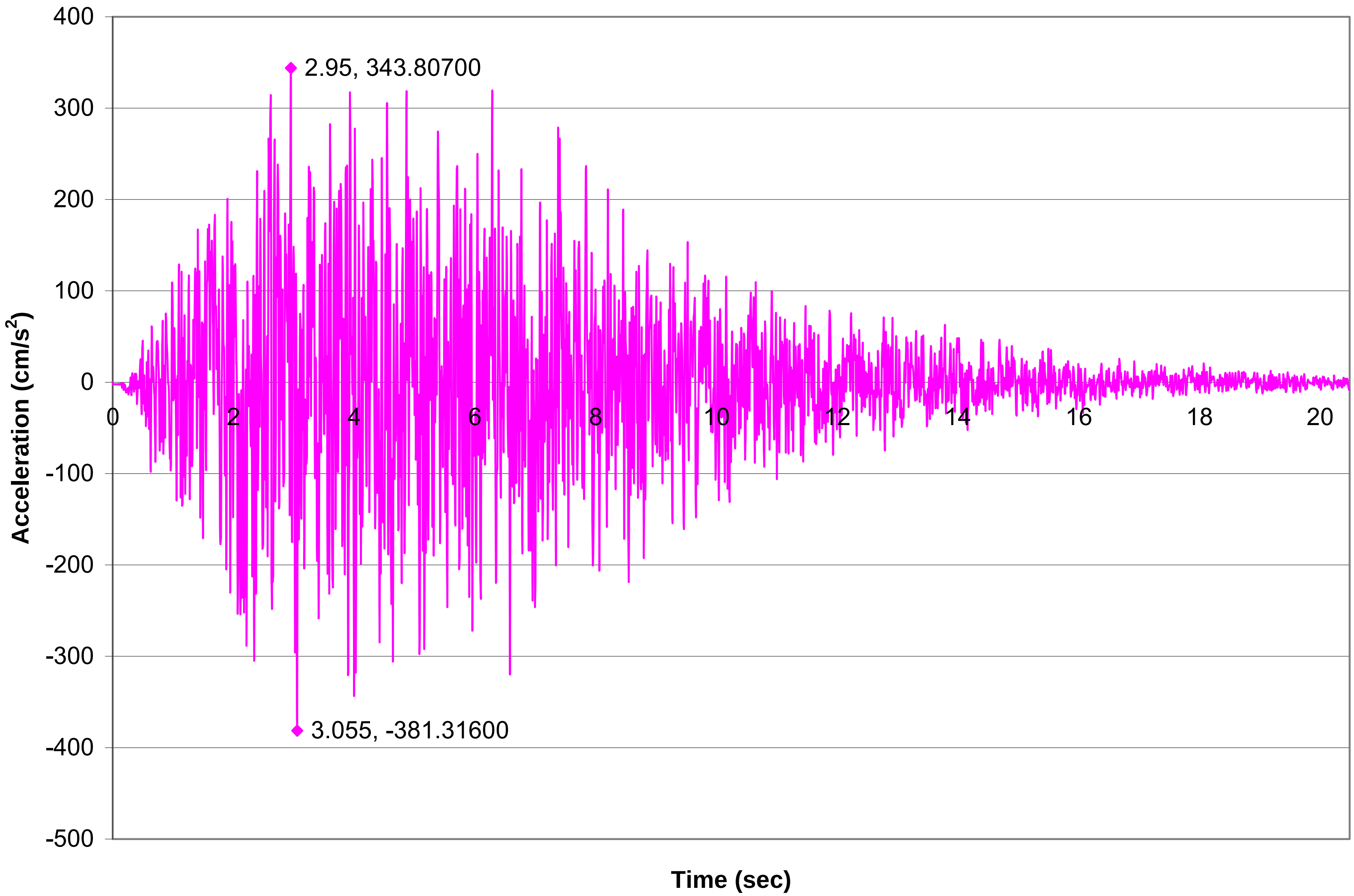


Figure 11: Acceleration Time History for Component-1 (Transverse) of the 250-Year Event for Counties Identified by 0.4g-1

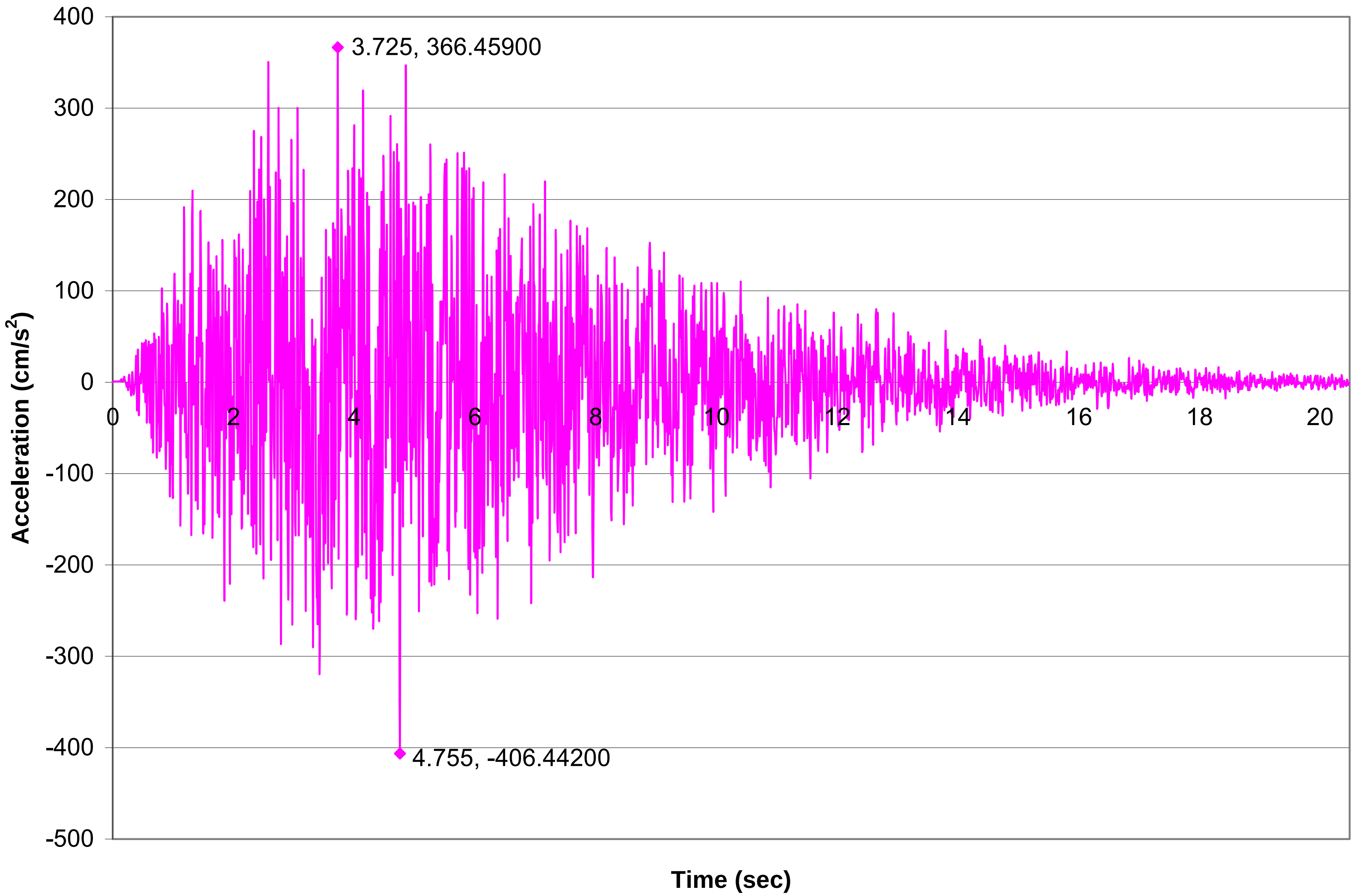


Figure 12: Acceleration Time History for Component-2 (Vertical) of the 250-Year Event for Counties Identified by 0.4g-1

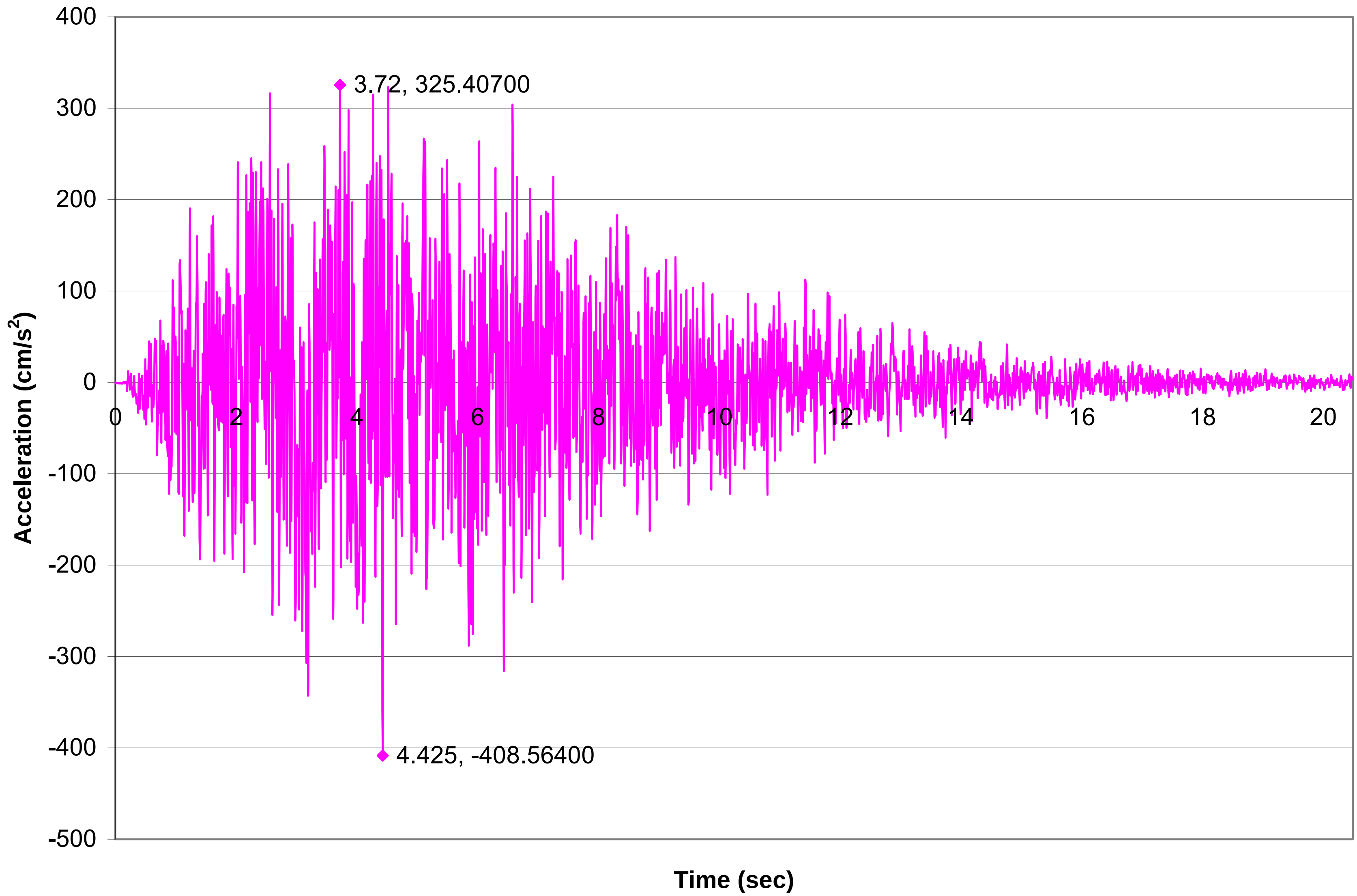


Figure 13: Acceleration Time History for Component-3 (Longitudinal) of the 250-Year Event for Counties Identified by 0.4g-1

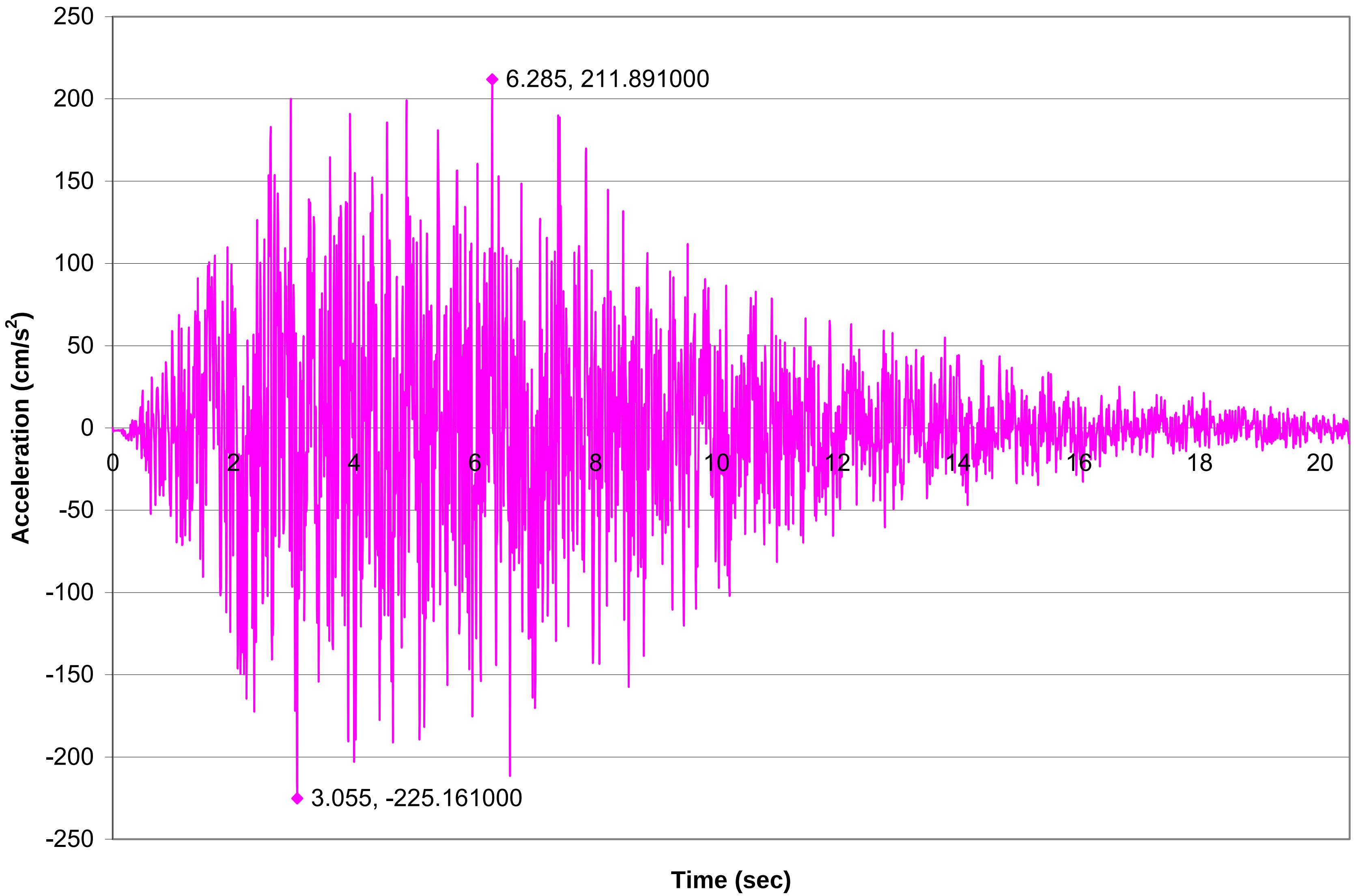


Figure 14: Acceleration Time History for Component-1 (Transverse) of the 250-Year Event for Counties Identified by 0.19g-1

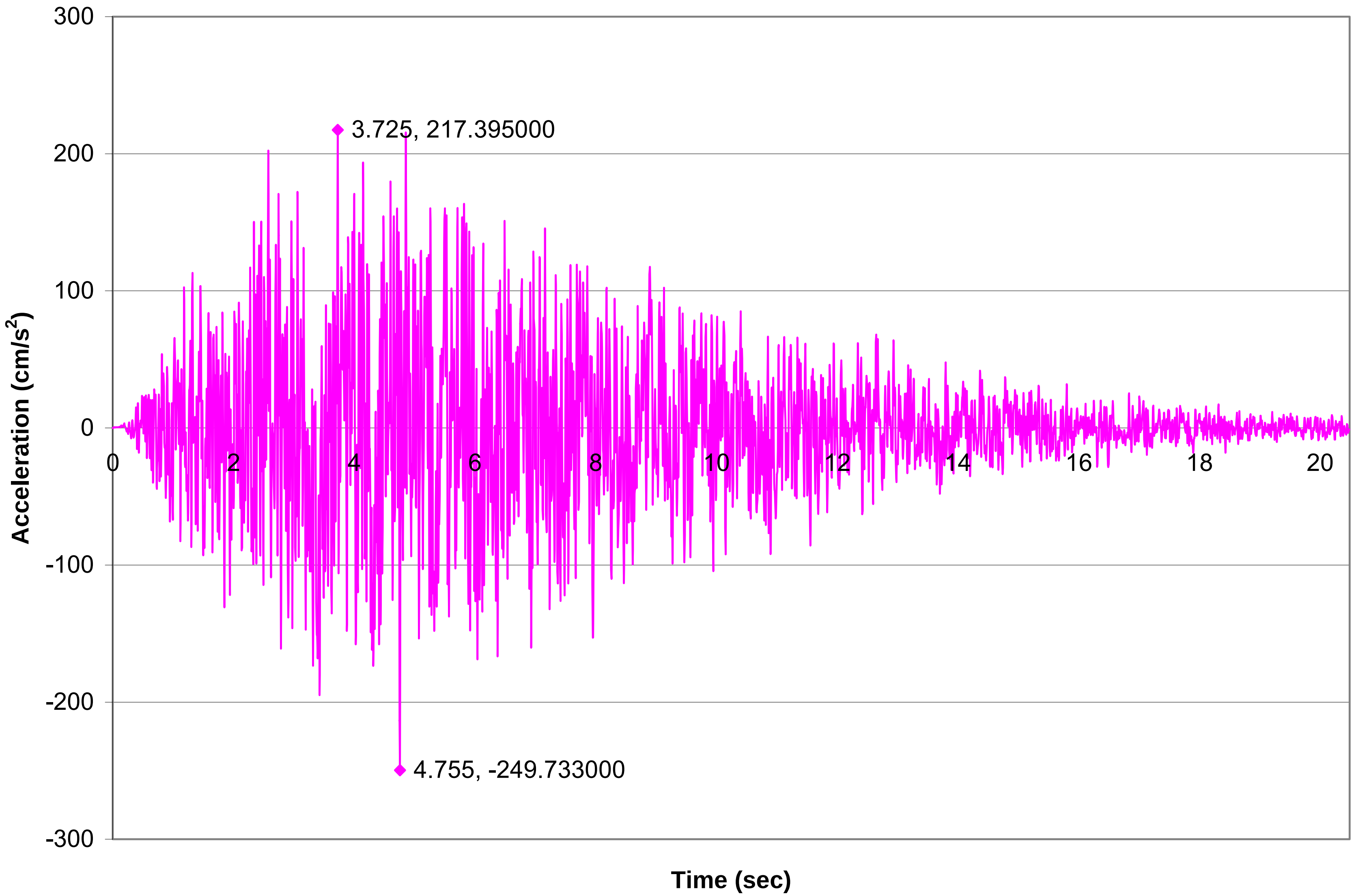


Figure 15: Acceleration Time History for Component-2 (Vertical) of the 250-Year Event for Counties Identified by 0.19g-1

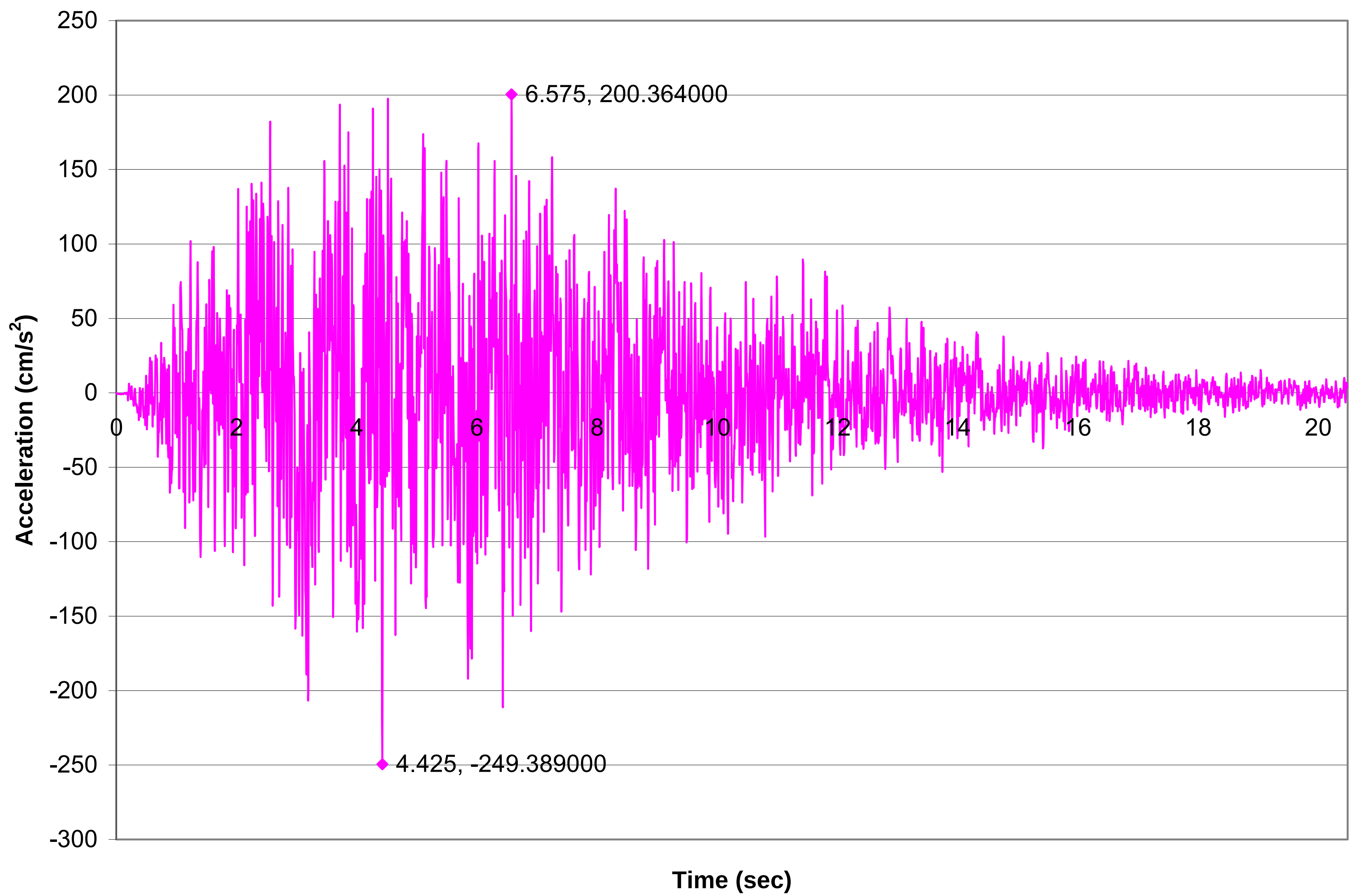


Figure 16: Acceleration Time History for Component-3 (Longitudinal) of the 250-Year Event for Counties Identified by 0.19g-1

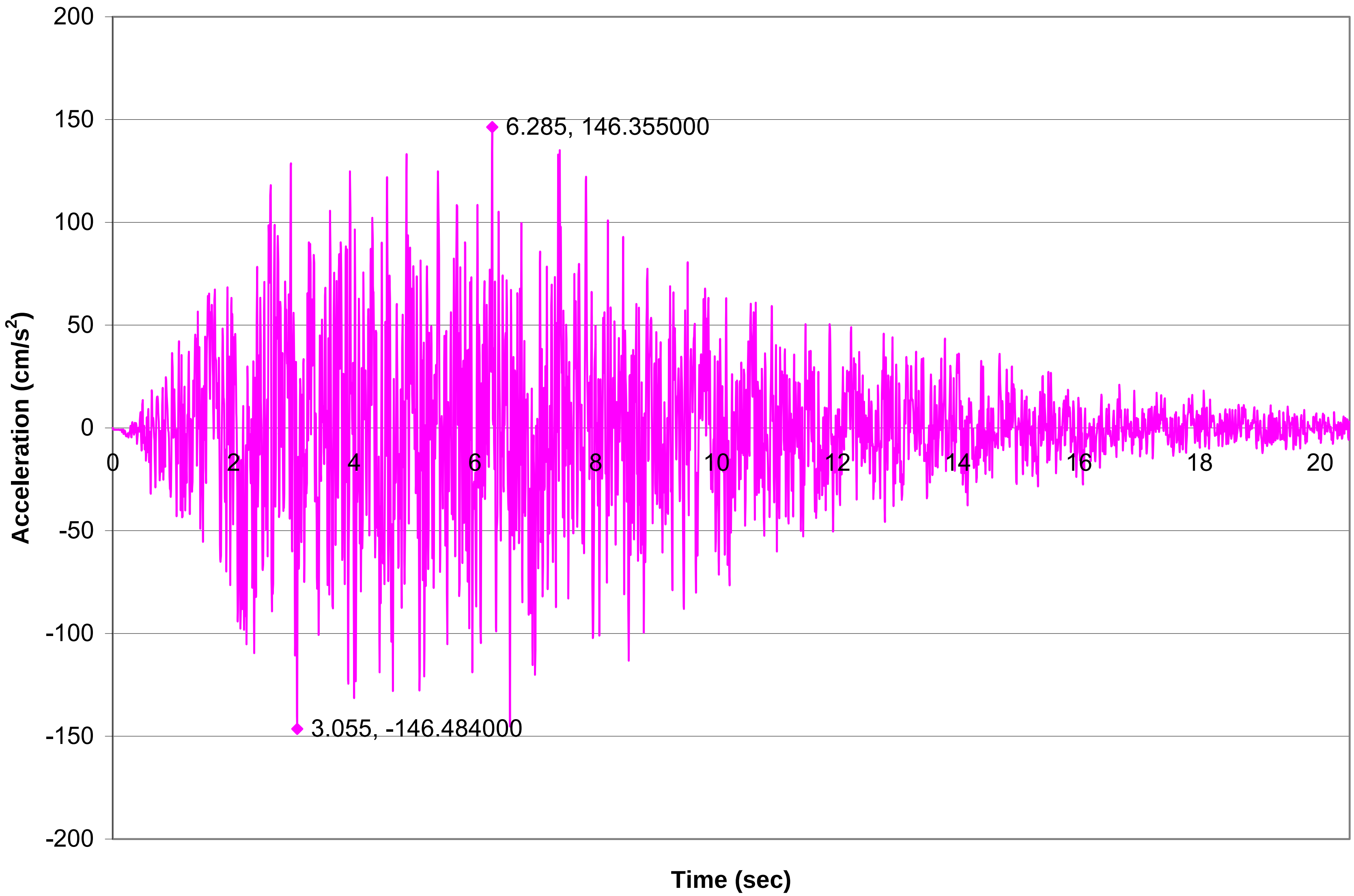


Figure 17: Acceleration Time History for Component-1 (Transverse) of the 250-Year Event for Counties Identified by 0.15g-1

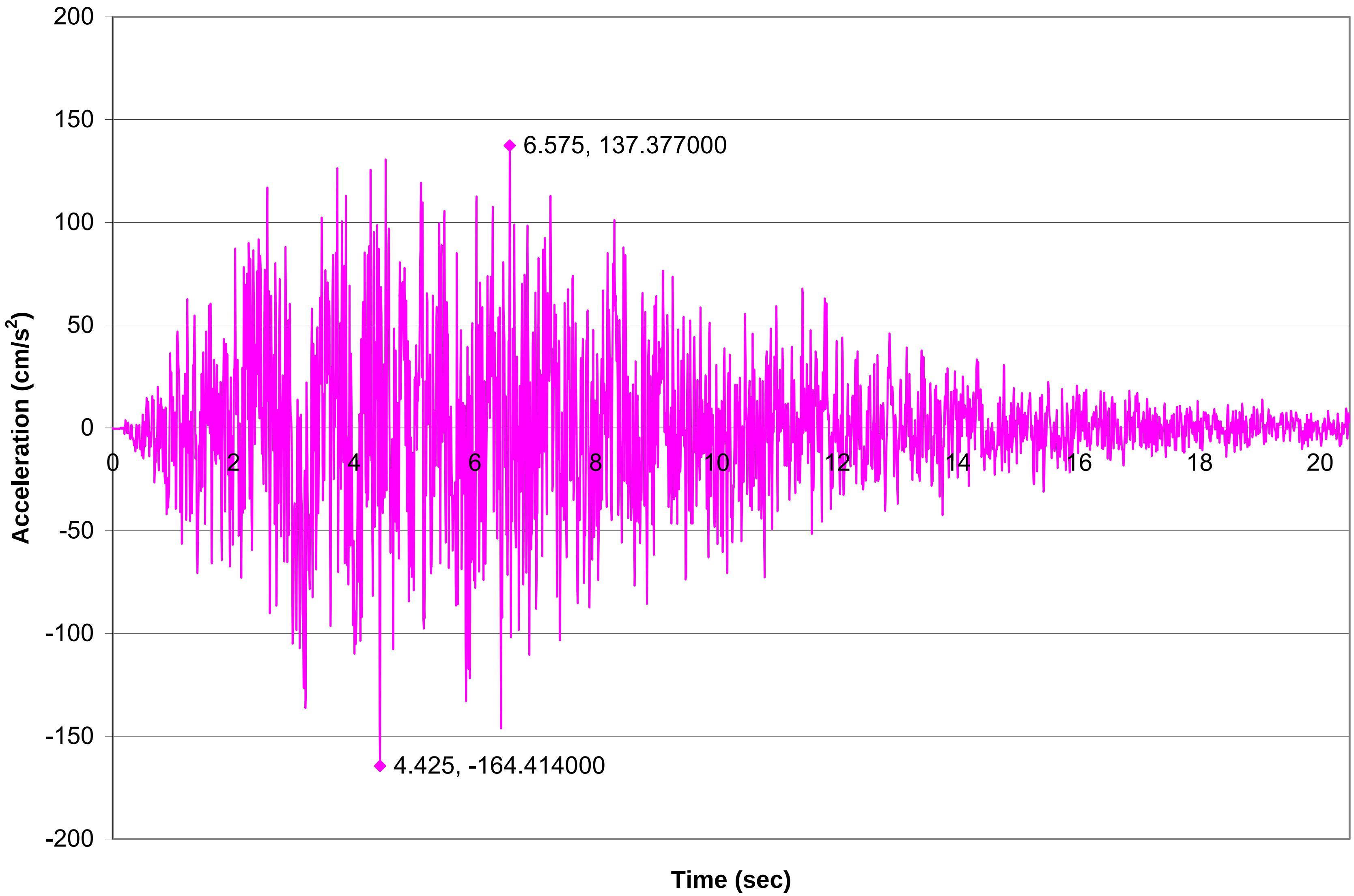


Figure 18: Acceleration Time History for Component-2 (Vertical) of the 250-Year Event for Counties Identified by 0.15g-1

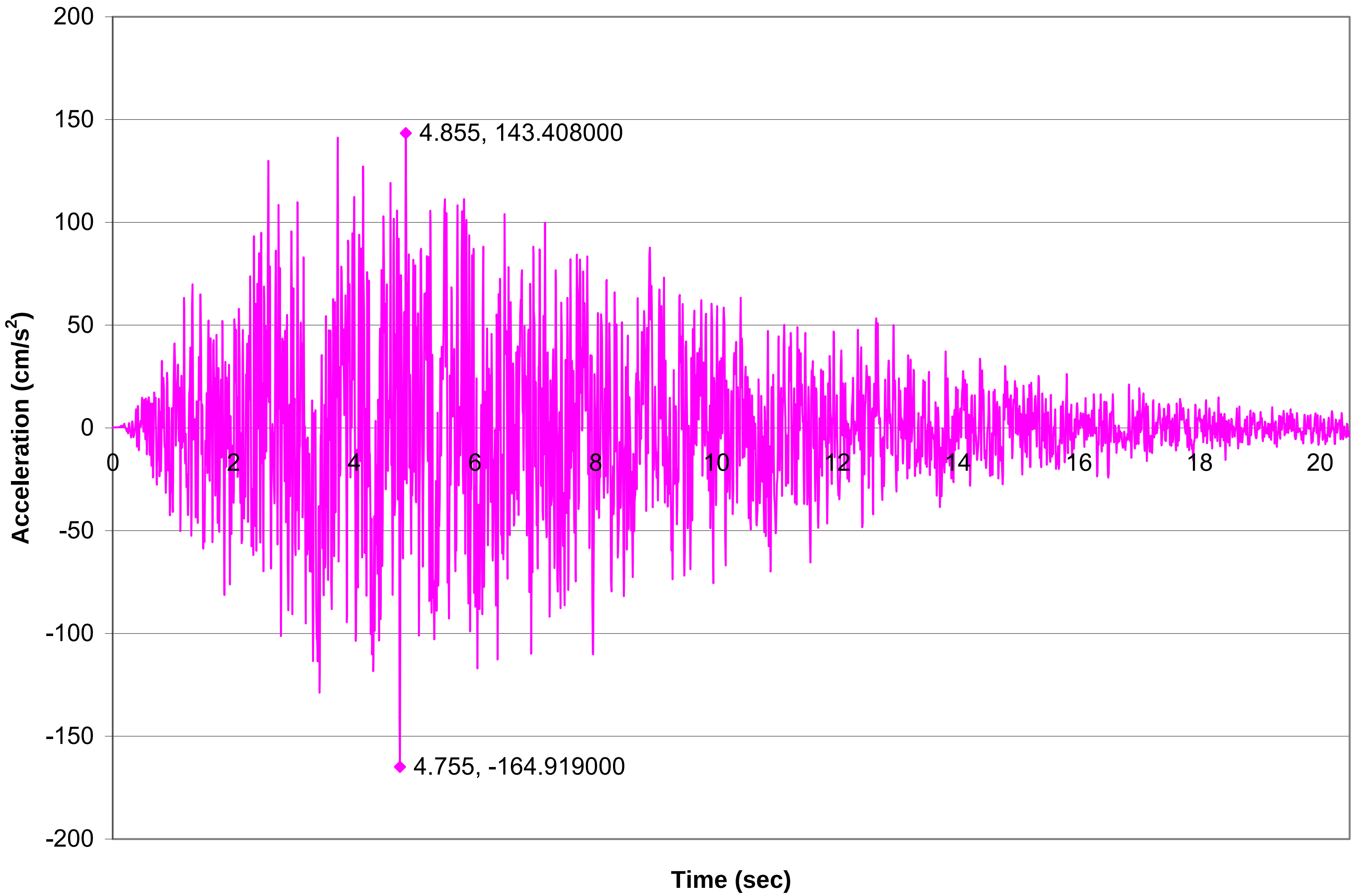


Figure 19: Acceleration Time History for Component-3 (Longitudinal) of the 250-Year Event for Counties Identified by 0.15g-1

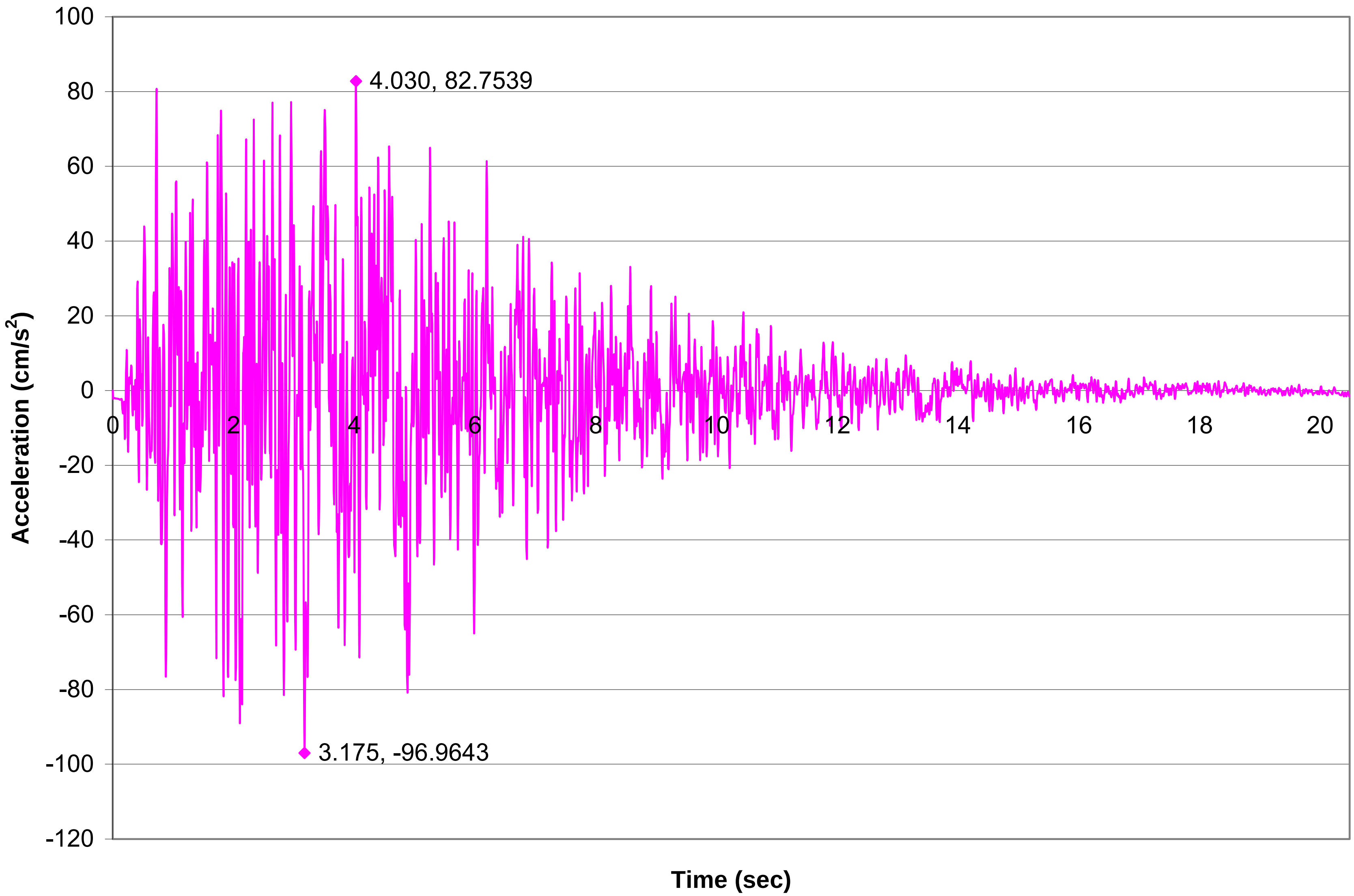


Figure 20: Acceleration Time History for Component-1 (Transverse) of the 250-Year Event for Counties Identified by 0.09g-2

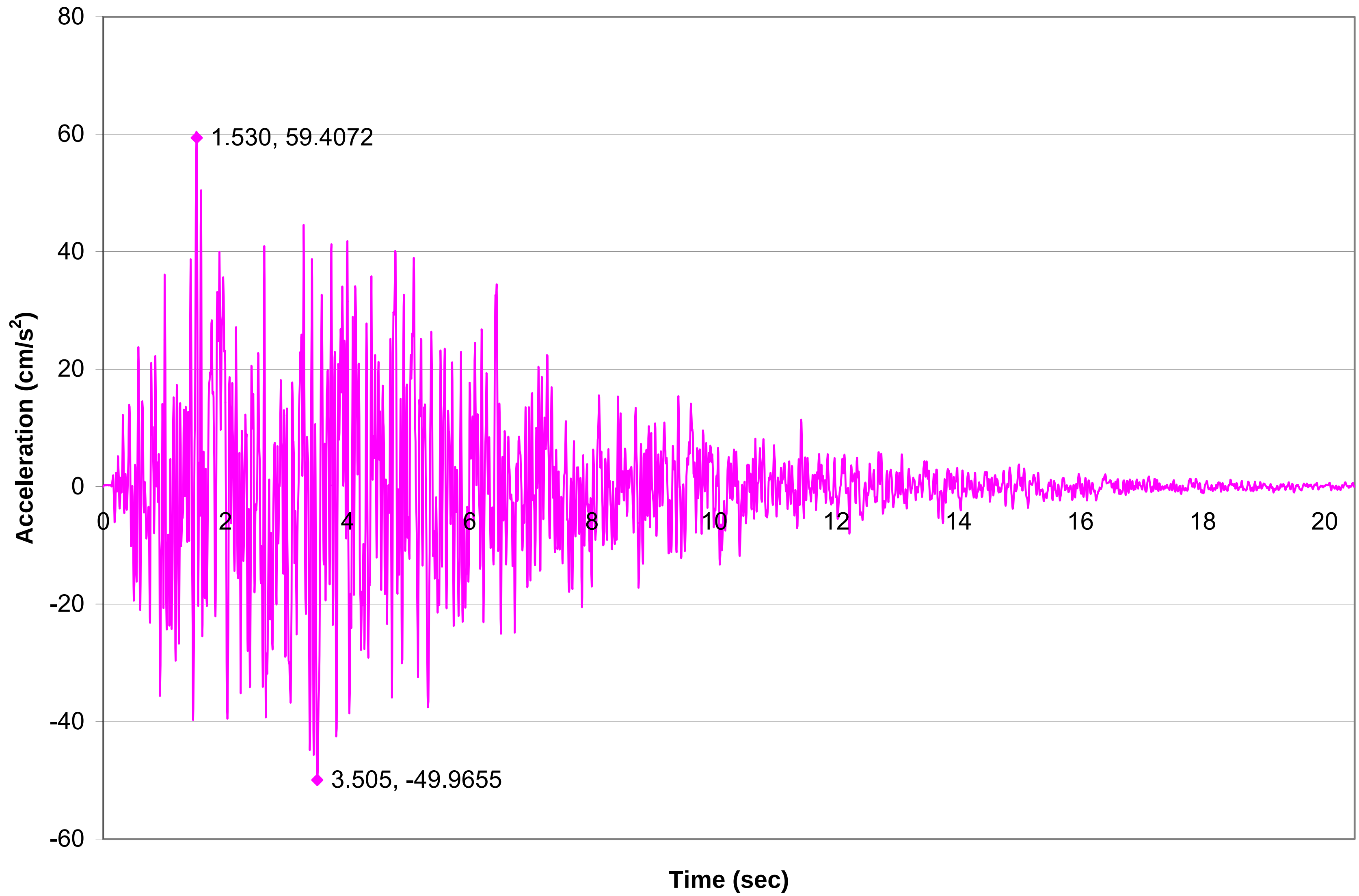


Figure 21: Acceleration Time History for Component-2 (Vertical) of the 250-Year Event for Counties Identified by 0.09g-2

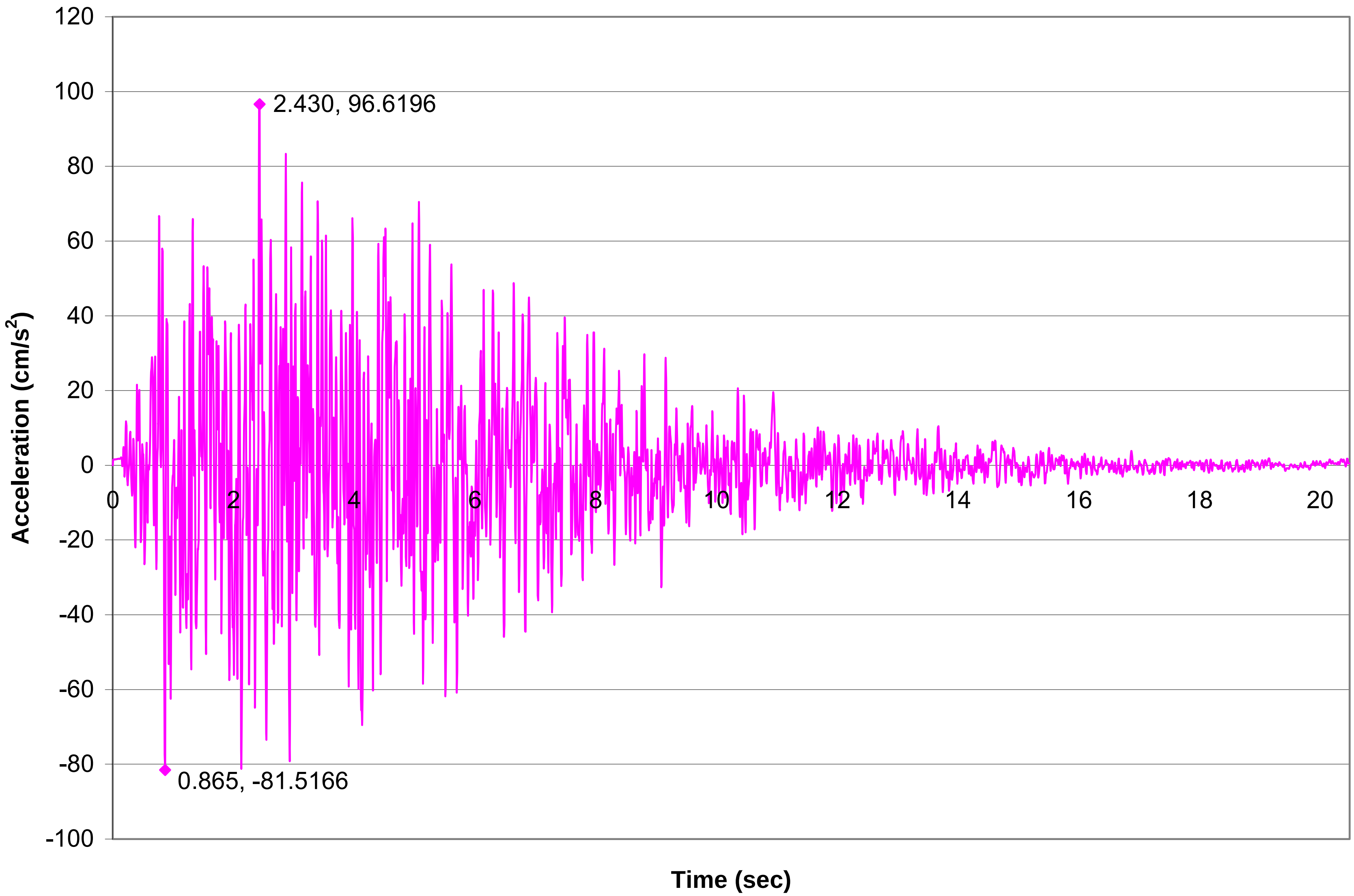


Figure 22: Acceleration Time History for Component-3 (Longitudinal) of the 250-Year Event for Counties Identified by 0.09g-2

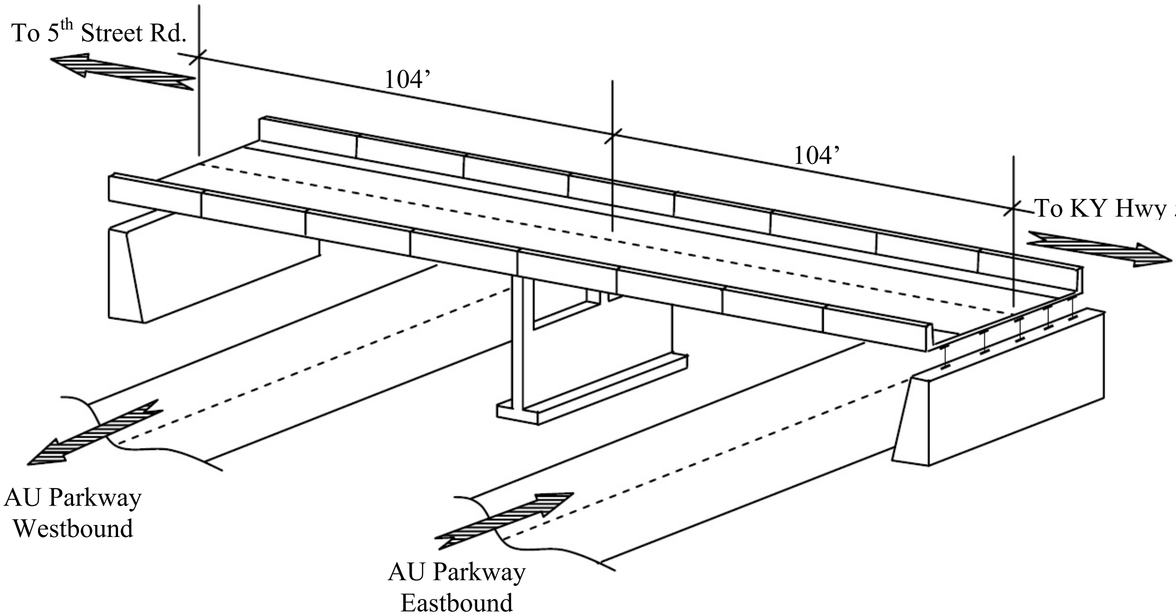


Figure 23: Example Bridge over Audubon Parkway in Daviess County, KY

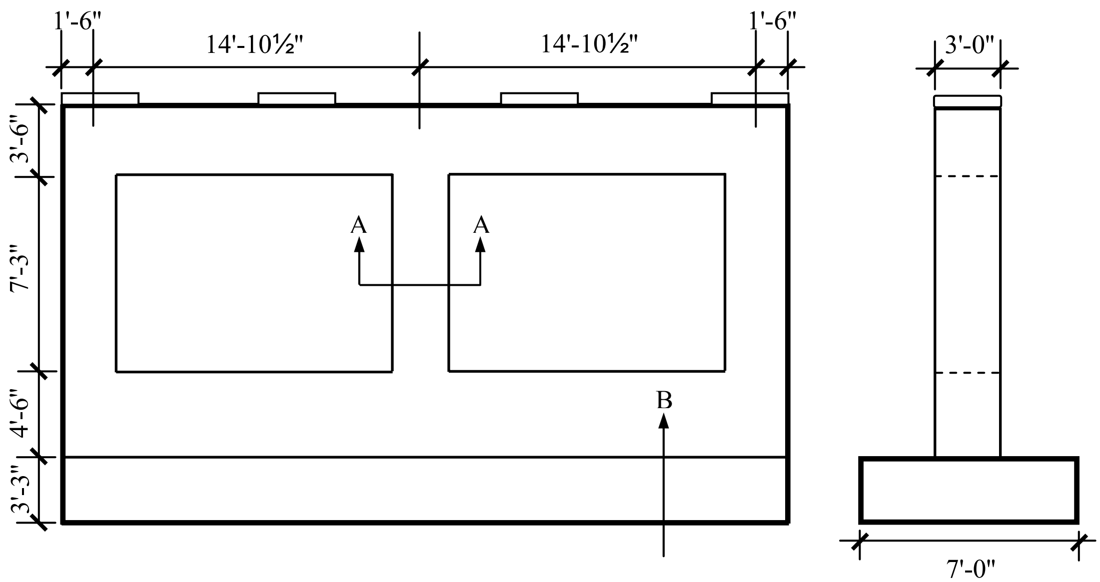


Figure 24: Dimension of the Substructure of the Example Bridge

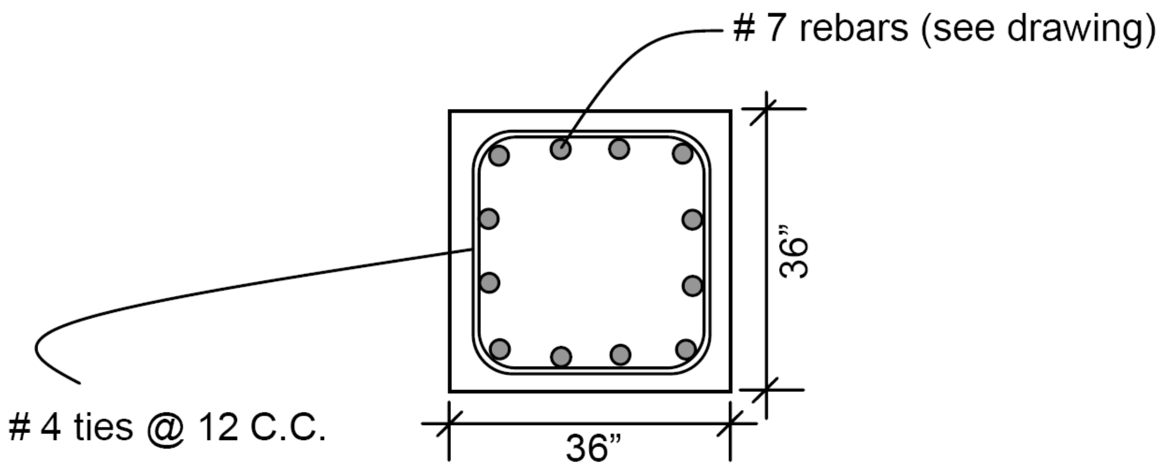


Figure 25: Section A-A of Columns of the Example Bridge

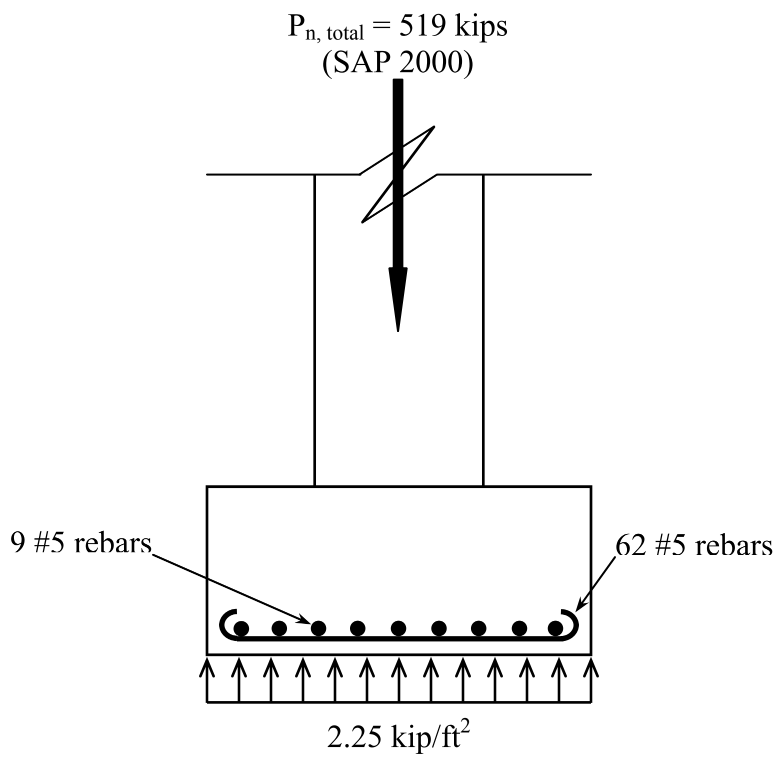


Figure 26: Section B-B of Column Footing: Applied Load and Soil Reaction

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