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15. SUPPLEMENTARY NOTES

This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. ABSTRACT

The response of bridges when subjected to seismic excitation can be evaluated by a number of analysis methods. The traditional approach to seismic analysis focuses on forces (so-called force-based methods of analysis) while current design practice is moving towards an increased emphasis on displacements (so-called displacement-based methods of analysis). The primary objective of this research project was to evaluate the effectiveness of various commercially-available computer programs for performing practical displacement-based seismic analysis of highway bridges. A secondary objective was to identify the fundamental differences between force-based and displacement-based methods of analysis, particularly as they apply to highway bridges. The objectives of the project were met by utilizing four different computer programs to evaluate the seismic response of a simple two-span highway bridge. The seismic response was evaluated using two force-based methods of analysis (response spectrum and time-history) and two displacement-based methods (capacity spectrum and inelastic demand spectrum). Furthermore, the effects of two different abutment and bent foundation support conditions were evaluated. The experience gained by utilizing the computer software revealed that some programs are well suited to displacement-based analysis, both from the point-of-view of being efficient and providing insight into the behavior of plastic hinges. The results of the seismic analyses demonstrated that force-based methods of analysis may be conveniently used to prioritize cases under which displacement-based methods of analysis should be applied. Furthermore, the displacement-based methods of analysis that were used produced different predictions of nonlinear response with neither method being regarded as producing accurate results due to a number of simplifications inherent in the methods. Finally, the displacement-based methods of analysis appear to be attractive to practicing engineers in the sense that they emphasize a graphical evaluation of seismic performance.

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TABLE OF CONTENTS

LIST OF FIGURES

Figure **Page**

LIST OF TABLES

EXECUTIVE SUMMARY

The response of bridges when subjected to seismic excitation can be evaluated by a number of analysis methods. The traditional approach that has been presented in seismic codes and specifications is to utilize linear static analysis procedures. However, there is currently a shift towards the use of new methods of displacement-based analysis in which the potential damage to a bridge is evaluated based on displacement-related quantities.

As a result of the recent shift in interest toward displacement-based seismic analysis, there is a need to identify which analysis methods and computer software are most effective for practical displacement-based seismic analysis of highway bridges and to identify the fundamental differences between traditional force-based methods of analysis and the new displacement-based methods of analysis. This study was conducted to fulfill this need.

This study was initiated by conducting a survey of State Departments of Transportation and consulting engineering firms to identify the analysis methods and associated software that are most commonly employed by practicing bridge engineers. Results of the survey showed that modal response spectrum analysis is the most common method of analysis used by practicing bridge engineers while the SAP2000 software program is most widely used in practice. Based on the survey results and other criteria, four different software programs (SC-Push3D, SAP2000, GT-STRUDL, and ADINA) were selected for evaluation in this study.

Seismic analysis of a simple two-span highway bridge was performed to evaluate the effectiveness of the four different software programs for performing displacement-based seismic analysis. The bridge was designed in a previous study for the Federal Highway Administration. Two different support conditions were considered, one employing seat-type abutments with rigid bent foundations (Basic Support Condition) and the second employing stub wall abutments with flexible bent foundations (Spring Support Condition). The effect of the support conditions on the need for displacement-based seismic analysis is presented.

Four methods of analysis were used to evaluate the seismic performance of the bridge when subjected to three different earthquake records in both the longitudinal and transverse directions. The analysis methods included two force-based linear dynamic methods (modal response spectrum analysis and time-history analysis) and two displacement-based nonlinear static methods (capacity spectrum analysis and inelastic demand spectrum analysis).

The force-based analyses showed that the bridge abutment and bent foundation support conditions resulted in significant differences in the dynamic response of the bridge, especially in the longitudinal direction. Furthermore, force-based analyses were used to identify which bridge configuration to examine via nonlinear displacement-based seismic analysis (i.e., the bridge with the Basic Support Conditions when subjected to the two strongest earthquake records in the longitudinal direction). Two methods of displacement-based analyses, the Capacity Spectrum Analysis method and the Inelastic Demand Spectrum analysis method, were utilized to determine the seismic performance of the bridge. The displacement-based analyses demonstrated that the bridge would indeed experience damage when subjected to the two strongest earthquake records. Interestingly, due to a number of simplifications inherent in the methods, neither method is expected to produce accurate results. However, the methods are apparently attractive to practicing engineers due to their emphasis on a graphical evaluation of seismic performance.

Based on the experience gained by the authors in utilizing the four software programs, SAP2000 is recommended over GT-STRUDL, SC-Push3D, and ADINA since it has several features that make it particularly useful for practical displacement-based seismic analysis of simple highway bridges. In particular, SAP2000 is the only program that completes the displacement-based analysis by overlaying the capacity and demand curve and identifying the performance point; the other programs are only capable of generating the pushover curve. However, it should be recognized that, if detailed information on the behavior of plastic hinges is sought, the GT-STRUDL program should be used since it uses a more advanced hinge plasticity model.

INTRODUCTION

BACKGROUND

The seismic design of typical highway bridges in the United States is governed by the American Association of Highway and Transportation Officials (AASHTO) Bridge Design Specifications. The specifications permit the designer to utilize a variety of methods for seismic analysis, from simple equivalent static analysis to complex nonlinear dynamic analysis. For simple bridges, it is common practice to utilize methods that employ elastic analysis, accounting for inelastic response through the application of force reduction factors. Such an approach may be regarded as *force-based* since the primary emphasis of the methods is on the forces within the structure. In addition to an elastic analysis, it is common practice to perform a capacity analysis associated with the desired inelastic response in which ductile flexural response occurs at selected plastic hinge regions within the structure. The plastic hinge regions are detailed to ensure plastic behavior while inhibiting nonductile failure modes.

In recent years, there has been a strong shift in seismic design and retrofit philosophy towards so-called *performance-based* seismic design. Under such a design philosophy, the primary objective is to design/retrofit structures such that they have predictable level of damage (seismic performance) when subjected to a range of ground motion intensities. To achieve such an objective, the inelastic behavior of the structure must be understood over a wide range of structural performance levels, rather than only at first yield or near collapse. A more explicit understanding of the inelastic response of a structure to seismic loading can be obtained through *displacement-based* seismic analysis; an analysis method in which the primary emphasis is on displacements rather than forces.

PROBLEM STATEMENT

A variety of software programs are available for displacement-based seismic analysis of structures. However, it is unclear which programs are most effective for practical analysis of typical highway bridges. The lack of consensus on which software programs are most effective results in an impediment to the use of displacement-based seismic design. Furthermore, the lack of knowledge on the relation between traditional force-based and newer displacement-based methods of analysis further impedes the use of the displacement-based methods. The impediments to the use of the displacement-based methods have recently become a more serious issue since the seismic design of structures is rapidly shifting toward the use of performance-based design specifications. Such specifications involve a more careful examination of damage potential to structures. The damage potential can usually be directly related to the inelastic displacement-capacity of the structure. Thus, the ability to obtain reasonably accurate estimates of displacement capacity is becoming increasingly important. In summary, the problem that bridge engineers face is that there is no clear consensus on how to perform reasonably efficient practical displacement-based seismic analysis. The major objective of this study is to solve this problem by identifying effective computer software tools for displacement-based seismic analysis. Furthermore, the primary differences between traditional force-based analysis and the new displacement-based analysis methods will be identified.

RESEARCH OBJECTIVES AND SCOPE

The major objective of this research was to identify which analysis methods and computer software are most effective for practical displacement-based seismic analysis of highway bridges. A secondary objective was to identify the primary differences between traditional force-based analysis and more recently developed displacement-based analysis methods.

To achieve these objectives, the following major tasks were completed:

1. A literature review was conducted to evaluate the fundamental differences between force-based and displacement-based analysis.
2. A survey of selected bridge engineering consulting firms and State Departments of Transportation (DOT's) was conducted to identify the most commonly used seismic analysis methods and associated software.
3. Selected computer software were utilized to perform force-based and displacement-based seismic analyses of a typical highway bridge.

REVIEW OF PREVIOUS WORK AND CURRENT PRACTICE

LITERATURE REVIEW

The seismic demand on a highway bridge can be determined by several methods. These methods include: linear static procedures (e.g., equivalent static analysis), linear dynamic procedures (e.g., modal analysis [response spectrum or time-history] and direct time-history analysis), non-linear static procedures (e.g., direct displacement-based analysis, capacity spectrum analysis, and inelastic demand spectrum analysis), and nonlinear dynamic procedures (e.g., nonlinear time–history analysis). The conventional approach to seismic design of simple highway bridges employs linear static procedures in which the lateral seismic forces are initially determined by an elastic analysis and subsequently reduced to inelastic design force levels via a response modification factor. This approach has several shortcomings, which have been accepted due to its simplicity and a lack of alternative practical approaches (Imbsen et al. 1996).

Recently, there has been a shift of attention away from traditional elastic force-based methods of seismic analyses since damage potential and ultimate failure can usually be directly related to the inelastic displacement capacity of the structure (Chandler and Mendis 2000). Thus, displacement-based methods of analysis are needed that are capable of realistically predicting the deformations imposed by earthquakes on structures. In response to this need, new nonlinear static analysis procedures have recently appeared in national resource documents such as the ATC-40 report (ATC 1996a) on seismic evaluation and retrofit of concrete buildings and the FEMA-273 NEHRP (National Earthquake Hazard Reduction Program) Guidelines for the Seismic Rehabilitation of Buildings (FEMA 1997a and 1997b). Such displacement-based methods of analysis are useful for predicting inelastic displacement capacities while simultaneously offering a compromise between the oversimplification of linear static analysis and the inherent complexity of nonlinear time-history analysis. For example, the *Capacity Spectrum Method* for nonlinear static analysis provides insight into potential failure mechanisms, ductility demands and stability under large drifts.

The Capacity Spectrum Method, originally developed by Freeman (Freeman et al. 1975 and Freeman 1978), is one of the more frequently utilized methods of displacement-based seismic analysis and is the principal method described in the ATC-40 report (ATC

1996a). One of the strong appeals of this method is that it is a graphical procedure in which the capacity of a structure is directly compared to the seismic demand on the structure. As described by Yu et al. (1999), the Capacity Spectrum Method begins with a nonlinear static pushover analysis which results in a graphical depiction of the global force-displacement relation for the structure (i.e., the pushover curve). Note that pushover analysis procedures are described in a number of references (e.g., see Krawinkler and Senevirtana (1998) and Kim and D'Amore (1999). The demand on the structure is then represented graphically by elastic spectra with equivalent viscous damping (i.e., the demand curve). According to Krawinkler (1995), however, a suitable value for the equivalent viscous damping is difficult to ascertain since a stable relationship between the hysteretic energy dissipation associated with the maximum excursion and equivalent viscous damping does not exist.

Several methods have been proposed to overcome the deficiencies of the original version of the Capacity Spectrum Method. In the FEMA-273 document, the *Displacement Coefficient Method* is used to characterize the displacement demand. In this method, the demand is represented by inelastic displacement spectra which are obtained from the elastic displacement spectra using a number of correction factors, which in principle are expected to be more accurate than elastic spectra with equivalent viscous damping (Fajfar 1999). Bertero (1995) recommended the use of smoothed inelastic design response spectra to represent the demand. Chopra and Goel (1999) developed the *Capacity-Demand-Diagram Method* that utilizes a constant-ductility design spectrum for the demand diagram. According to Chopra and Goel (1999), for SDOF systems, the *Capacity-Demand-Diagram Method* produces results that are significantly more accurate than those obtained using the approximate procedures described in the ATC-40 report (ATC 1996a).

Finally, it should be recognized that efforts continue to be made to improve upon current forms of displacement-based seismic analysis (e.g., the ATC-55 Project entitled "Evaluation and Improvement of Inelastic Seismic Analysis Procedures") and such efforts should be monitored to keep abreast of the most recent developments on this evolving topic. Furthermore, it is noted that displacement-based methods of seismic design appear in the National Cooperative Highway Research Program (NCHRP) Report 472 entitled "Comprehensive Specifications for the Seismic Design of Highway Bridges" (ATC/MCEER 2002) which forms the basis for the "Recommended LRFD Guidelines for the Seismic Design of Highway Bridges" that is under preparation within the ATC-49 Project (ATC 2001a). AASHTO is considering incorporating the ATC-49 provisions either within a Guide Specification or within the AASHTO LRFD Specifications (AASHTO 1998).

SOFTWARE SURVEY

Several commercial software packages are available for determining the demand and capacity of a structure subjected to earthquake loading. Some software has special features that make it more appropriate for one type of analysis over another. In order to determine the most commonly used analysis methods and associated software, a survey of bridge engineering consulting firms and State DOT's was conducted. The state DOT's were located in regions of high seismic risk as indicated by the acceleration contour map of the AASHTO bridge design specifications (AASHTO 1992). The survey, which was in the form of a questionnaire, included the following seismic analysis methods: AASHTO single mode methods (Uniform Load Method or Single-Mode Spectral Method), ATC-32 (ATC 1996b) equivalent static method, modal response spectrum analysis, elastic time-history analysis, inelastic static analysis (demand defined by inelastic response spectrum), capacity spectrum analysis (demand defined by modified elastic response spectrum), and inelastic time-history analysis. A total of twenty-two state DOT's and thirty-one bridge engineering consulting firms were contacted to fill out the survey. Among those, seventeen state DOT's and nineteen consulting firms replied to the survey, a return of about sixty-eight percent. A summary of the most common software for each analysis method is provided in Table 1 while details of the survey are provided in Appendix A. Note that in a separate survey conducted as part of the ATC-55 Project entitled "Evaluation and Improvement of Inelastic Seismic Analysis Procedures" (ATC 2001b), the most common software used by practicing engineers for displacement-based seismic analysis of buildings was SAP2000 (used by 39% of 79 respondents).

BRIDGE MODELS, SEISMIC EXCITATION, AND SELECTED SOFTWARE

BRIDGE MODELS

The highway bridge under study is taken from Design Example No. 1 of the Federal Highway Administration (FHWA) Seismic Design of Bridges Series (Mast et al. 1996a). The bridge was selected since it has characteristics that are similar to highway bridges in Washington State (in particular, the selected bridge has rigid connections between the bent columns and spread footings). The bridge consists of a two-span continuous cast-in-place, post-tensioned, reinforced concrete box girder with a three-column integral bent, and spread footings. Figure 1 shows plan and elevation views of the bridge, Figure 2 shows a typical cross-section of the bridge, Figures 3 and 4 show abutment support details for two different support conditions, and Figure 5 shows a cross-section of the abutments. The bridge is a non-essential bridge (Importance Classification II) located at a site with Soil Profile II (250 ft deep glacial sand and gravel) and an Acceleration Coefficient of 0.28. Thus, the bridge is assigned to Seismic Performance Category C. The weight of the bridge is 4876 kips which includes half of the bent weight.

Table 1 Summary of Software Survey Results.

Analysis Method	Most Common Software
AASHTO single mode methods	SEISAB
ATC-32 equivalent static method	Hand Calculations
Modal response analysis	SAP2000 and SEISAB
Elastic time-history analysis	SAP2000
Inelastic static analysis	SAP2000
Capacity spectrum method	WFRAME
Inelastic time-history analysis	ADINA

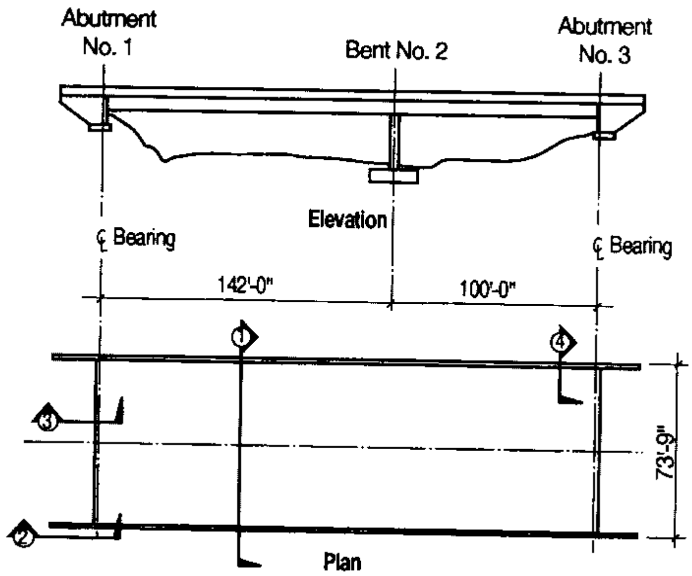


Figure 1 Plan and Elevation Views of Bridge (from Mast et al. 1996b).

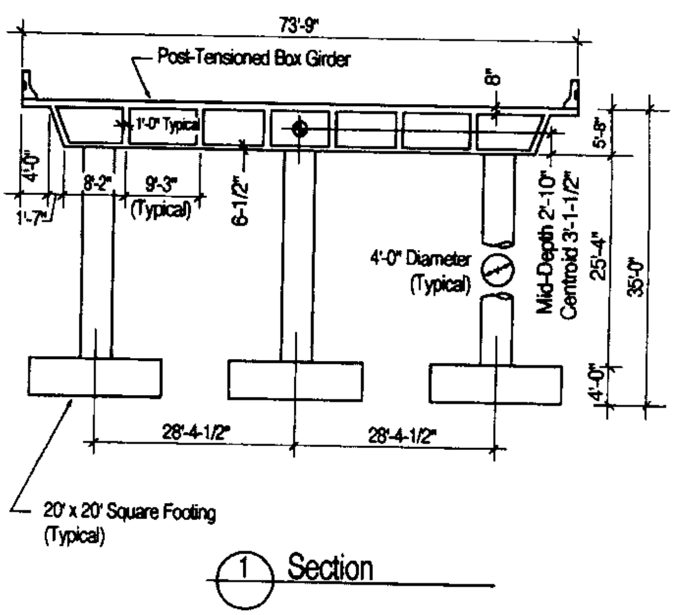


Figure 2 Cross-Section of Bridge (from Mast et al. 1996b).

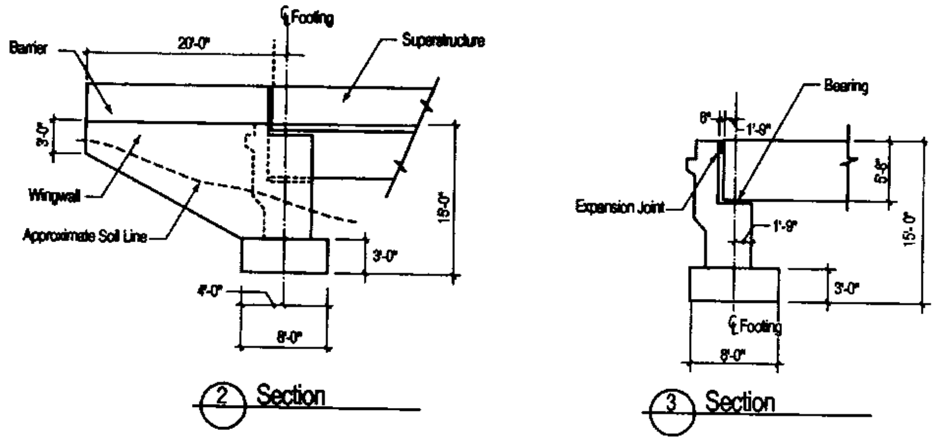


Figure 3 Seat-Type Abutment Details for Basic Support Model (from Mast et al. 1996b).

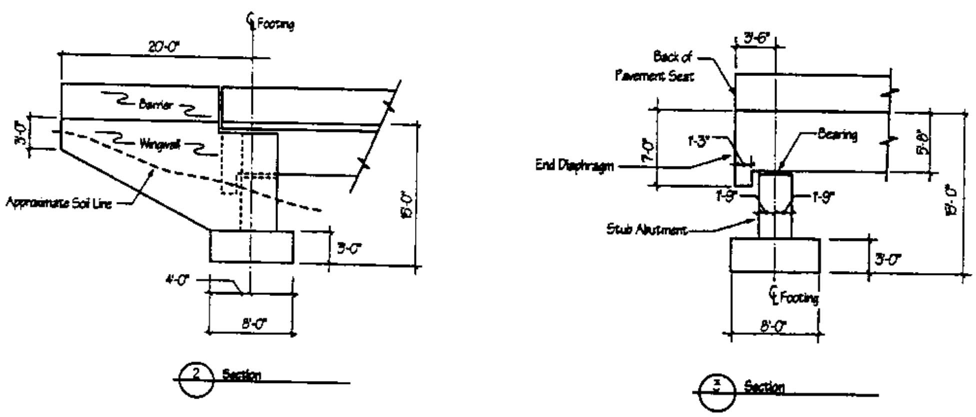


Figure 4 Stub Wall Abutment Details for Spring Support Model (from Mast et al. 1996a).

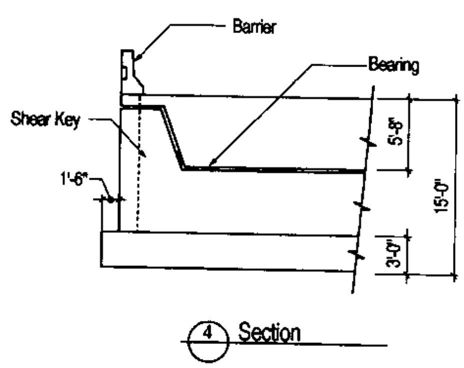


Figure 5 Abutment Cross-Section Showing Bearing Support and Shear Key (from Mast et al. 1996b).

Two different bridge configurations were considered. In the first configuration, the *Basic Support Configuration*, the bridge has seat-type abutments which allow limited longitudinal movement of the superstructure due to the gap between the superstructure and the abutment back wall (see Figure 3). The support provided by the abutment is assumed to be fixed against translation in the vertical and transverse directions and fixed against rotation about the longitudinal axis while the column bent footings are considered to be fixed against both translation and rotation (see Figure 6). The lateral load behavior of the bridge in the longitudinal and transverse directions is depicted in Figure 7. Note that in the longitudinal direction the bent resists the lateral force while in the transverse direction most of the lateral force is taken by the abutments. In addition, for the Basic Support Configuration, the gross (uncracked) column moment of inertia was used in the seismic analysis.

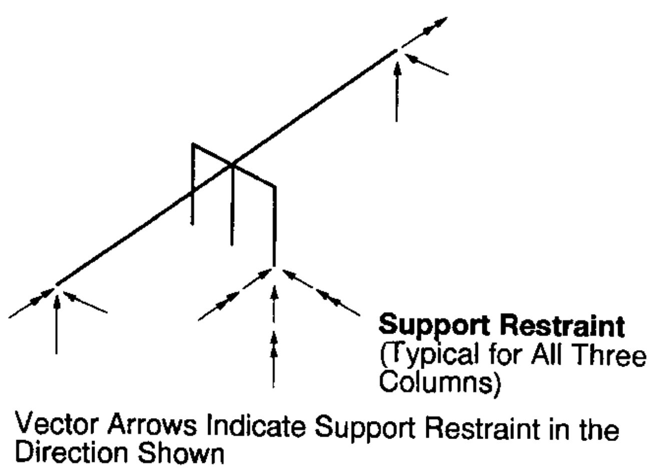


Figure 6 Support Conditions for Basic Support Model (from Mast et al. 1996b).

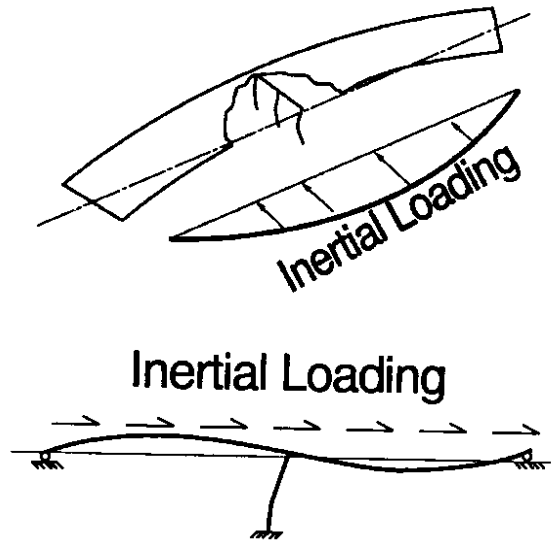


Figure 7 Transverse and Longitudinal Seismic Response of Bridge with Basic Support Conditions (adapted from Mast et al. 1996b.

In the second configuration, the *Spring Support Configuration*, the bridge has stub wall abutments which are restrained in the longitudinal and transverse directions due to end diaphragm and wing wall interaction with the soil, respectively (see Figure 4). The support provided by the abutment is assumed to be fixed against translation vertically, fixed against rotation about the longitudinal axis of the superstructure, and has translational springs in the longitudinal and transverse directions while the column bent footings were restrained against translation and rotation with springs provided in each orthogonal direction to account for soil flexibility (see Figure 8). Note that, for the column bent footings, the Spring Support Configuration presented in Mast et al. (1996a) does not include the translational springs or the vertical rotational spring that are included in this study. For the abutment supports, the stiffness of the translational springs was determined in accordance with the California Department of Transportation (Caltrans) bridge design specifications (Caltrans 1994). The resulting values were K_a = 83,250 kip/ft for the longitudinal stiffness and K_w = 53,200 kip/ft for the transverse stiffness. For the bent footings, McGuire et al. (1994) recommend the use of spring-only foundations (i.e., no mass or dashpot elements) for spread footings constructed on intermediate and stiff soils. The stiffness of the translational and rotational springs for the column footing supports was determined based on two studies which are presented in the NEHRP Guidelines for the Seismic Rehabilitation of Buildings (Gazetas 1991 and Lam et al. 1991). The resulting values were $K_x = K_y$ = 90,684 kip/ft for the transverse and longitudinal translational springs, K_z = 133,016 kip/ft for the vertical translational spring, $K_{\theta x} = K_{\theta y}$ = 9,930,00 kip-ft/rad for the transverse and longitudinal rotational springs, and $K_{\theta z}$ = 13,307,000 kip-ft/rad for the vertical rotational spring. In addition, for the Spring Support Configuration, an effective (cracked) column moment of inertia was used as explained subsequently.

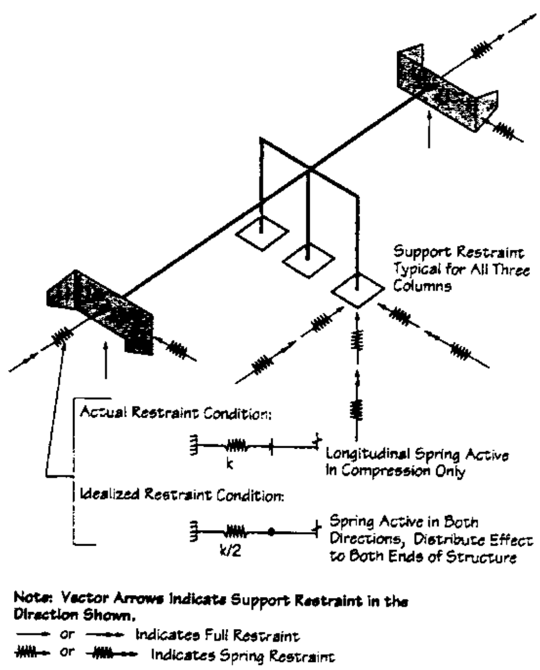


Figure 8 Support Conditions for Spring Support Model (adapted from Mast et al. 1996a).

The idealized mathematical model of the bridge is shown in Figure 9. The superstructure is represented by a single line of multiple three-dimensional frame elements (i.e., a spine-type configuration) which pass through the mid-depth (approximately the centroid) of the superstructure. Each of the columns and the cap beam are represented by single three-dimensional frame elements which pass through the geometric center and mid-depth, respectively. A single element was deemed sufficient for the columns since they are relatively short and prismatic. Since the cross-section of the superstructure is uniform, it was deemed sufficient to locate nodes at the tenth points of each span. The mass was specified per unit length of the members with half the member mass being subsequently assigned to each node. The superstructure cross-sectional area is 120 ft^2 and the moment of inertia about the strong and weak cross-sectional axes are 51,000 ft^4 and 575 ft^4, respectively. The box-girder is assumed to be integral with the bent and thus full continuity is employed at the superstructure-bent intersection. The torsional rigidity about the longitudinal axis of the elements representing the superstructure is increased by several orders of magnitude over the actual value to reflect the physical torsional restraint due to the spatial support of the superstructure on the bent and abutments. The cap beam cross-sectional area is 25 ft^2 and the moment of inertia about the strong and weak cross-sectional axes and the torsional rigidity about the longitudinal axis were increased by several orders of magnitude over the actual values to ensure that, as would expected for the actual bridge, the columns attract approximately equal forces. The circular columns were initially sized with a diameter of 4 ft resulting in a cross-sectional area of 12.6 ft^2 and a gross moment of inertia of 12.6 ft^4. As shown in Figure 10, a rigid end zone for the elements representing the columns was used to account for the offset between the centerline of the cap beam and the soffit of the box-girder. It is assumed that the concrete has a nominal 28-day compressive strength of 4 ksi and a modulus of elasticity of 3,600 ksi and that the transverse and longitudinal reinforcement has a nominal yield strength of 60 ksi.

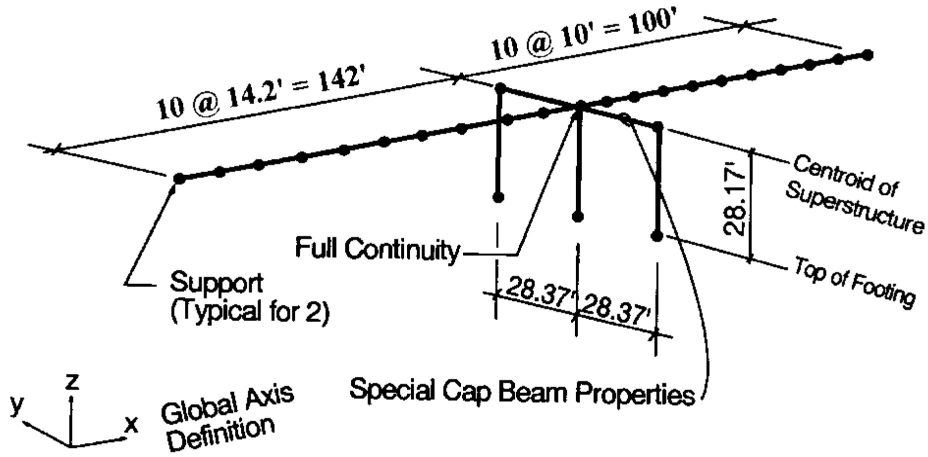


Figure 9 Computer Model of Bridge (adapted from Mast et al. 1996b).

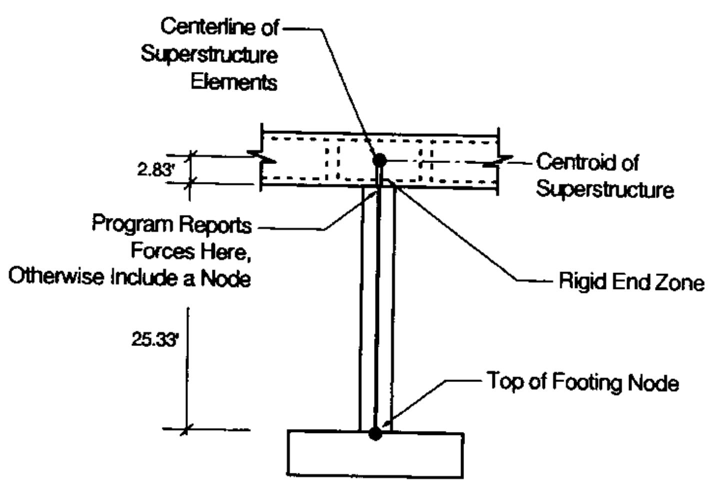


Figure 10 Modeling Details for Column Connections to Cap Beam and Footing (adapted from Mast et al. 1996b)

The bridge was designed (see Mast et al. 1996a) using the Single-Mode Spectral Method of the 15th Edition of the *AASHTO Standard Specifications for Highway Bridges,* Division I, as amended by the Interim Specifications-Bridges-1993 through 1995. (AASHTO 1992). In addition, the design complied with the proposed revisions to the *AASHTO Standard Specifications for Highway Bridges, Division I-A: Seismic Design* which were developed within Task 45 of the NCHRP Project 20-7 and adopted by AASHTO in 1994. For the Basic Support Condition, the design resulted in 4-ft diameter columns with a longitudinal steel reinforcement ratio of 1.9% (see Figure 11) while for the Spring Support Condition the design resulted in 3-ft diameter columns with a longitudinal steel reinforcement ratio of 1.0%. In this study, for both the Basic and Spring Support Conditions, the 4-ft diameter column design with a longitudinal steel reinforcement ratio of 1.9% was used (see Figure 11). Note that, as per standard practice, the bridge columns were designed to preclude shear failures in favor of flexural failures.

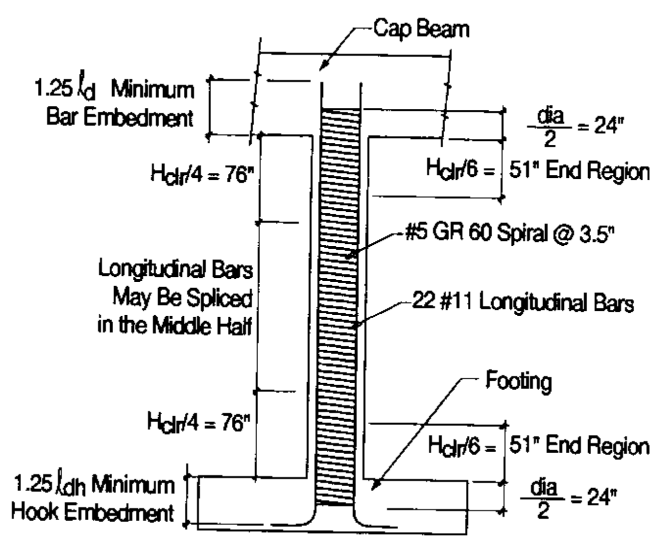


Figure 11 Column Reinforcement Details (from Mast et al. 1996b).

Since the bridge columns are expected to respond inelastically under the input ground motions, effective column properties were used to reflect concrete cracking and reinforcement yielding. As shown in Figure 12, the effective flexural stiffness (I_{eff}) of the bridge columns depends on the axial load ratio ($P/f_c'A_g$) and the longitudinal reinforcement ratio (A_{st}/A_g) where P is the axial load, f_c' is the concrete compressive strength, A_g is the gross area of the section, and A_{st} is the area of the longitudinal steel (Priestley et al. 1996). Based on an axial dead load of 1100 kips (estimated by assuming that the dead load is distributed equally to each of the three columns), a column diameter of 4 ft, and a concrete compressive strength of 4 ksi, the axial load ratio is 0.15. The columns of the bridge have a longitudinal reinforcement ratio of 1.9% and thus Figure 12 indicates that the effective moment of inertia of the column is approximately 50% of the gross section moment of inertia (I_g). Note that, as was done in the *Bridge Design Example* (Mast et al. 1996a), the effective flexural stiffness of $0.5I_g$ was used for the Spring Support Conditions while the gross (uncracked) flexural stiffness was used for the Basic Support Conditions. Due to the presence of prestress forces, the effective flexural stiffness of the multi-cell box girder is assumed to be equal to the uncracked flexural stiffness (Priestley et al. 1996).

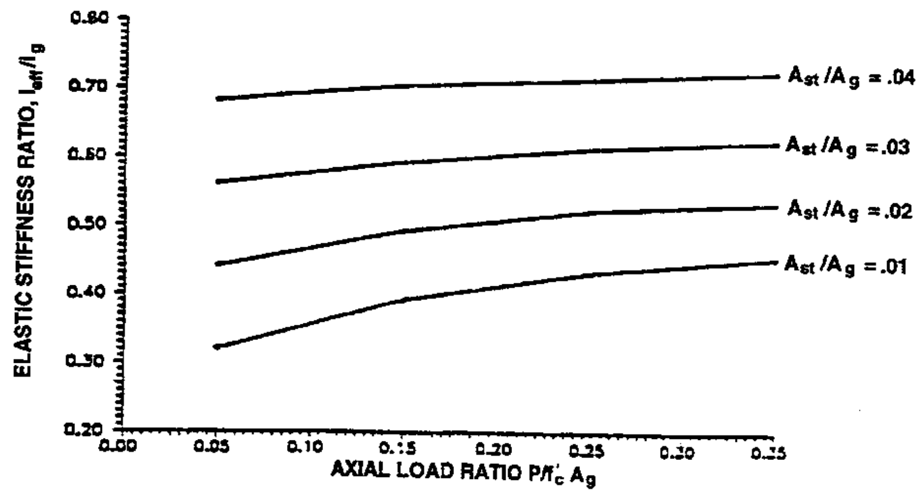


Figure 12 Effect of axial load ratio and longitudinal steel ratio on effective flexural stiffness for circular cross sections (adapted from Priestley et al. 1996)

SEISMIC GROUND MOTIONS

In this study, the bridge was analyzed using linear dynamic analysis and nonlinear static analysis with the seismic ground motion represented by three different historical earthquake records. Three records were selected since this represents the minimum number of records recommended by current seismic design codes for time history analysis (e.g., AASHTO 1992). The three earthquake records selected were as follows: 1) the S00E component of the El Centro record of the 1940 Imperial Valley, California Earthquake; 2) the N86E component of the Olympia (Washington DOT Highway Test Lab) record of the 1949 Western Washington Earthquake; and 3) the N90E component of the Sylmar Hospital Parking Lot record of the 1994 Northridge, California Earthquake. The ground acceleration records and associated pseudo-acceleration response spectra for each earthquake, all plotted to the same scale, are shown in Figure 13 along with selected characteristics of the records in Table 2. As is evident from Figure 13 and Table 2, the characteristics of these records are quite different in terms of amplitude, frequency content, and duration. The rationale for selecting these three records was as follows: the El Centro record was selected since the spectral characteristics of this record are known to closely match the design spectral shapes adopted in the Uniform Building Code on firm soil sites (Chandler and Mendis 2000), the Olympia record was selected since it corresponds to a relatively strong earthquake in the geographical region of interest, and the Sylmar record is from the Northridge Earthquake, an earthquake which caused the partial or complete collapse of five bridges while damaging approximately 200 others (EERI 1995).

SELECTED SEISMIC ANALYSIS SOFTWARE

The following four different software programs were utilized in this study: GT-STRUDL, SAP2000, ADINA and SC-Push3D. The motivation for utilizing these particular software programs was as follows: GT-STRUDL was selected since the Washington State Department of Transportation (WSDOT) bridge design office regularly utilizes this program for force-based analysis; SAP2000 was selected because the survey results (see Table 1) indicated that this program is widely used in practice and the WSDOT bridge design office recently acquired this program; ADINA was selected because of interest expressed by the WSDOT bridge design office in expanding their use of this program; and SC-Push3D was selected because it was specifically developed for pushover analysis. Each program is briefly described bebw with detailed descriptions provided throughout the report as needed.

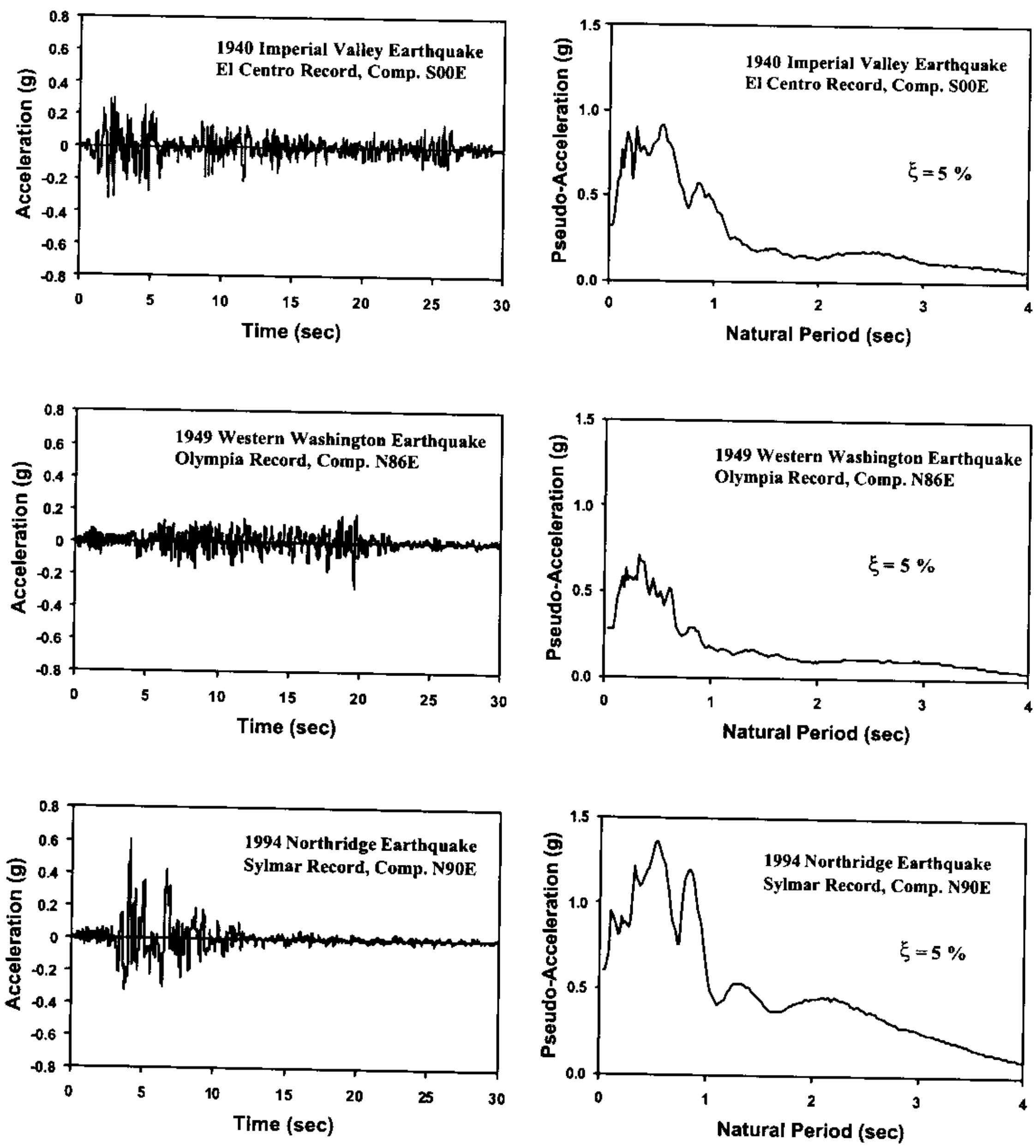


Figure 13 Earthquake Ground Acceleration Records used for Seismic Analysis and Associated 5%-Damped Pseudo-Acceleration Response Spectrums.

Table 2 Characteristics of earthquake ground motions used in seismic analysis.

Earthquake	Station	Comp.[1]	Ms[2]	ED[3] (km)	PGA[4] (g)
1940 Imperial Valley	El Centro	S00E	6.7	8.2	0.319
1949 Western Washington	Olympia Highway Test Lab	N86E	7.1	26.2	0.280
1994 Northridge	Sylmar Hospital Parking Lot	N90E	6.8	19.0	0.604

[1] Component; [2] Surface Wave Magnitude; [3] Epicentral Distance; [4] Peak Ground Acceleration

GT-STRUDL (Georgia Tech STRUctural Design Language) was developed in 1975 and is a general purpose finite element analysis program for static and dynamic analysis of two- and three-dimensional linear and nonlinear structures. The program continues to be developed by the Georgia Tech CASE (Computer-Aided Structural Engineering) Center. Version 25, which is compatible with the Microsoft Windows operating system (GT-STRUDL 2000), was utilized in this study. Features and enhancements in version 25 that are relevant to this study include pushover analysis and nonlinear dynamic analysis. Over the duration of this study, version 26 was released with the release of version 27 pending. Enhancements that are planned beyond version 27 include graphical display of plastic hinge status and other pushover analysis information.

SAP2000 is a general purpose finite element analysis program for static and dynamic analysis of two- and three-dimensional linear and nonlinear structures with a particular emphasis on dynamic loading and earthquake loading. The software is developed by Computers and Structures, Inc. and is the first version of the SAP programs that is completely integrated with the Microsoft Windows operating system (CSI 1997). SAP2000 is available as a suite of programs that includes SAP2000, SAP2000 Plus, and

SAP2000 Nonlinear. In particular, the program used for this study, SAP2000 Nonlinear, is capable of performing pseudo-static nonlinear pushover analysis and nonlinear time-history analysis. In addition, SAP2000 can perform Capacity Spectrum Analysis using design spectra.

ADINA (Automatic Dynamic Incremental Nonlinear Dynamic Analysis), developed by ADINA R & D, Inc., is a general purpose finite element analysis program for static and dynamic analysis of two- or three-dimensional linear and nonlinear systems (ADINA 2000). With regard to seismic response analysis, ADINA is capable of performing response spectrum analysis and time-history analysis. In addition, ADINA can perform pushover analysis, although there are no specific utilities available to aid the user in performing such an analysis.

SC-Push3D, developed by SC Solutions, Inc., is a three-dimensional finite element program that is capable of performing linear and nonlinear static analysis of frame-type structures. The program emphasizes the ability to perform pseudo-static pushover analysis. The latest version of SC-Push3D (version 2.0*)* is a DOS-based program (SC Solutions 1998).

A summary of the selected software programs, including the version number and platform used, is shown in Table 3. In addition, Table 3 indicates which types of analyses were performed with each program.

Table 3 Software Utilized for Seismic Analysis of Bridge Structure.

Software	Version and Platform	Force-Based: Time-History Analysis	Force-Based: Response Spectrum Analysis	Displacement-Based: Inelastic Demand Spectrum Analysis	Displacement-Based: Capacity Spectrum Analysis
GT-STRUDL	V. 25.0 Win95	X	X	X	X
SAP2000 Nonlinear	V. 7.0 Win95	X	X	X	X
ADINA	900 Node Win95	X	X	---	---
SC-Push3D	V. 2.0 DOS	---	---	X	X

Notes: X = Used, --- = Not Used

DESCRIPTION OF SEISMIC ANALYSIS PROCEDURES

LINEAR STATIC ANALYSIS (Force-Based Methods)

The simplest procedure for seismic analysis of highway bridges is linear static analysis. In this procedure, the bridge is assumed to remain elastic while being subjected to a static force load distribution that is approximately equivalent to the inertial force distribution associated with the fundamental mode of vibration. Within the AASHTO Bridge Design Specifications, there are two methods available for linear static analysis: the *Uniform Load Method* and the *Single-Mode Spectral Method.* The two methods differ in the form of the lateral force distribution. The static lateral forces are applied independently along the two principal axes (i.e., the longitudinal and transverse axes) and the resulting internal forces are combined using approximate combination rules. To account for inelastic response, the internal forces are reduced by response modification factors. To ensure ductile response under strong earthquake motions, a collapse mechanism is postulated resulting in the identification of plastic hinge locations (so-called *Capacity Design*). The plastic hinge locations are then subjected to special detailing requirements so as to ensure ductile flexural response while inhibiting nonductile failure modes such as brittle shear failures. The emphasis of linear static analysis is on the forces that develop within the bridge as thus such methods of analysis may be regarded as force-based methods.

LINEAR DYNAMIC ANALYSIS (Force-Based Methods)

As a result of recent developments in desktop computing capabilities and seismic analysis software, there has been a shift among practicing engineers toward the routine application of linear dynamic analysis rather than linear static analysis for typical highway bridges. The application of linear dynamic analysis is favored due to the insight it can provide into the dynamics of the bridge and its ability to explicitly account for the effects of multiple modes of vibration. Furthermore, the results of linear dynamic analysis can be used to determine whether significant inelastic behavior is likely to occur and thus can be used to determine whether more complex static or dynamic nonlinear analysis is warranted.

Linear dynamic analysis procedures include response spectrum analysis and time-history analysis. The former is a method for obtaining an approximate solution of the coupled, second-order, linear differential equations of motion under forced vibration. The response spectrum analysis begins with determining the natural frequencies and mode shapes via an eigenvalue analysis. The coupled equations of motion are then decoupled via a modal transformation wherein the principle of orthogonality of the mode shapes with respect to the mass, damping, and stiffness matrices is applied. Each decoupled equation corresponds to the equation of motion of a single-degree-of-freedom (SDOF) system associated with a mode of vibration. The peak response of each single degree-of-freedom system is obtained through the use of elastic-response spectra. Since the peak response in each mode does not occur at the same time, the peak response with contributions from all modes is estimated via the application of modal combination rules that are based on random vibration theory. Note that the AASHTO Bridge Design Specifications refer to the response spectrum analysis method as the *Multimode Spectral Method*.

Whereas response spectrum analysis results in estimates of peak response, a time-history analysis provides a method for obtaining the "exact" response of a structure as a function of time. The response-history is normally determined using step-by-step numerical integration of the equation of motion. Consequently, time history analysis is performed using computer software where the ground acceleration is divided into small time steps and the response is calculated at the end of each time step while satisfying dynamic equilibrium. In general, the ground acceleration is only available at discrete points in time separated by a fixed time step while the solution may be sought at points in time other than at integer multiples of the time step. Thus, the ground acceleration must be interpolated, with linear interpolation often being adequate.

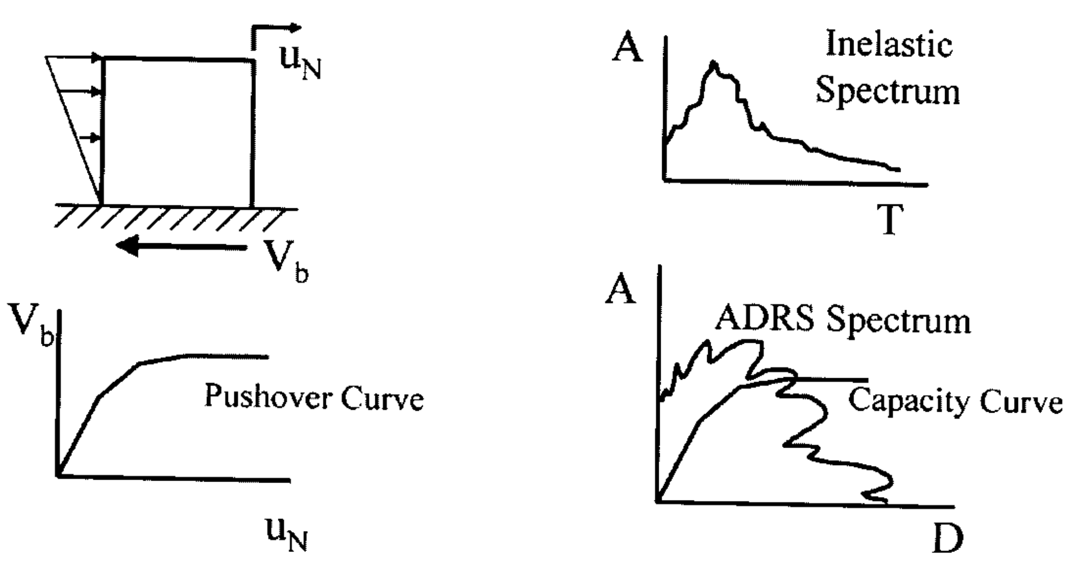
As alluded to above, the primary difference between time-history analysis and response spectrum analysis is that time-history analysis includes explicit consideration of the time domain whereas the response spectrum analysis does not. For time-history analysis, the peak response can be obtained directly from the absolute maximum value on a response-history plot. In contrast, for response spectrum analysis, the peak response is estimated by combining modal peak responses and thus a single value, rather than an entire time-history, is obtained for each response quantity. Several methods can be used for combining the modal peak responses, the most well-known being the Square Root of the Sum of the Squares (SRSS) method and the Complete Quadratic Combination (CQC) method. The CQC method was used in this study for combining the modal responses since it takes into account the statistical coupling between closely spaced modes due to modal damping.

The *elastic design forces* along the two principal axes of the bridge that have been determined from longitudinal and transverse linear dynamic analysis are combined using a 100/30 percentage rule. Next, *modified design forces* are determined by taking into account the potential for inelastic behavior during moderate to strong ground shaking. Specifically, the modified design forces are obtained by applying response modification reduction factors to the elastic design forces. The response modification factors account for both ductility and overstrength. Finally, the modified design forces are used to design the bridge components or, for existing bridges, to check the vulnerability of components. As explained for linear static analysis, a Capacity Design is performed to ensure ductile failure modes.

The linear dynamic analysis procedures may be regarded as *force-based* since the emphasis is on forces rather than displacements. In fact, it is unlikely that the displacements will be properly estimated. It might also be pointed out that linear dynamic analysis is not well suited for structures of irregular configuration (e.g., highly skewed bridges in which nonlinear behavior is expected due to nonuniform distribution of seismic forces) (FEMA 1997a) or for evaluating the damage susceptibility of existing structures subjected to significant levels of seismic force (Deierlein and Hseih 1990).

NONLINEAR STATIC ANALYSIS (Displacement-Based Methods)

Damage to bridge structures may be most directly related to displacements and displacement-related quantities (e.g., ductility). Thus, one may approach the seismic analysis of a structure with a primary emphasis on displacements rather than forces. Such an approach is referred to as displacement-based seismic analysis. Although a number of methods have been proposed for performing displacement-based seismic analysis and design, two methods are emphasized in this report, one that employs an elastic demand spectrum (Capacity Spectrum Analysis) and another that utilizes an inelastic demand spectrum (Inelastic Demand Spectrum Analysis). A graphical depiction of these two methods is shown in Figure 14.



(a) Structure Capacity

(b) Seismic Demand and Performance Evaluation

Figure 14 Graphical Depiction of Displacement-Based Seismic Analysis

Displacement-based seismic analysis generally begins with a pseudo-static multi-degree-of-freedom (MDOF) pushover analysis of the bridge to establish the *pushover curve* which is, in turn, transformed to a *capacity curve* that characterizes the bridge response in its fundamental mode of vibration (see Figure 14). Note that the terminology for displacement-based analysis is still evolving and thus the terms used above are not necessarily consistent with those found in other related documents. The pushover analysis requires the selection of a lateral force distribution (often being proportional to the fundamental mode shape) and a control node (often the center of mass of the bridge superstructure). The force distribution is applied to the bridge in an incremental fashion while monitoring the occurrence of nonlinear behavior and plotting the base shear (V_b) versus control node displacement (u_N). Note that gravity loads should be applied to the bridge prior to the application of lateral loads. The pushover analysis is stopped when the bridge reaches either a pre-defined displacement limit or the ultimate capacity is reached. The ultimate capacity may correspond to either a localized failure (e.g., a single plastic hinge reaching its curvature capacity) or the development of a global collapse mechanism (i.e., sufficient plastic hinges developing to render the structure unstable). The resulting base shear versus control node displacement [multi-degree-of-freedom (MDOF) pushover curve] is then converted to a *capacity curve* for an equivalent SDOF system by converting the base shear to the fundamental mode pseudo-spectral acceleration (A) and the control node displacement to the fundamental mode spectral displacement (D). The equations for converting the MDOF pushover curve to the SDOF capacity curve are obtained from basic principles of modal analysis (see, for example, Chopra 2001):

$$A = \frac{V_b}{M_1^*} \tag{1}$$

$$D = \frac{u_N}{\phi_{N1}\Gamma_1} \tag{2}$$

where M_1^* is the effective modal mass for the fundamental mode, ϕ_{N1} is the modal amplitude of the control node in the fundamental mode, and Γ_1 is the modal participation factor for the fundamental mode. The effective modal mass and modal participation factor for the fundamental mode are given by:

$$M_1^* = \frac{\left(\sum_{j=1}^{n} m_j \phi_{j1}\right)^2}{\sum_{j=1}^{n} m_j \phi_{j1}^2} \tag{3}$$

$$\Gamma_1 = \frac{\sum_{j=1}^{n} m_j \phi_{j1}}{\sum_{j=1}^{n} m_j \phi_{j1}^2} \tag{4}$$

where m_j is the lumped mass at the j-th node, ϕ_{j1} is the j-th node modal amplitude in the fundamental mode, and n is the number of modes.

As noted above, the transformation equations from the base shear – control node displacement domain (i.e., the pushover curve) to the fundamental mode spectral acceleration – spectral displacement domain (i.e., the capacity curve) are based on modal analysis principles. Modal analysis inherently involves the superposition of modal responses and thus, strictly speaking, only applies to linear systems. Since displacement-based analysis procedures begin with the development of the nonlinear pushover curve, it is evident that the transformations equations [Equations (1) and (2)] are not strictly valid. To address this discrepancy, alternative nonlinear static analysis procedures have been developed. For example, the ATC-40 report (ATC 1996a) indicates that the transformation equations can be modified by updating the fundamental mode shape to reflect the effects of nonlinear response. Additional alternatives to the inelastic static analysis procedure described above include the application of a fundamental mode loading distribution that adapts to changes in the nonlinear response of the structure (e.g., see Bracci et al. 1997 and Elnashai 2001) and the consideration of response in multiple modes of vibration (ATC 1996a and Sasaki et al. 1996). According to the ATC-40 report (ATC 1996a), a general rule-of-thumb is that the higher modes of vibration of building structures should be considered for inelastic static analysis if the natural period of the fundamental mode is greater than about one second.

Capacity Spectrum Analysis

The Capacity Spectrum Analysis method, originally developed by Freeman et al. (1975), is a graphical procedure for evaluating the seismic performance of a structure. The method requires the development of a capacity curve (obtained from pseudo-static pushover analysis) and a series of demand curves. The demand curves are expressed in the form of elastic Acceleration-Displacement Response Spectra (ADRS) corresponding to various viscous damping ratios [see Figure 14(b)]. The acceleration in the demand curve is pseudo-acceleration, not actual acceleration. Once the capacity curve and demand curves (elastic ADRS spectra) have been obtained, the seismic performance of the structure is assessed by overlaying the curves. Typically, the yielding portion of the capacity curve will intersect the demand curves corresponding to several viscous damping ratios. Each point on the capacity curve can be associated with an equivalent viscous damping ratio and natural period. The point at which the capacity curve intersects a demand curve associated with the same viscous damping ratio is the performance point which defines the spectral displacement demand. The displacement and base shear demand for the MDOF structure are obtained by inverting the relations given by Equations (1) and (2).

The equivalent viscous damping ratio of the inelastic system (ξ_{eq}) is obtained by equating the energy dissipated in one cycle of motion for the inelastic system and the equivalent linear system. For bilinear systems with strain-hardening, the equivalent viscous damping ratio can be determined as follows (Chopra and Goel 1999):

$$\xi_{eq} = \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)} \tag{5}$$

where μ is the displacement ductility of the structure (i.e., the ratio of maximum displacement to yield displacement) and α is the ratio of post to pre-yielding stiffness. Note that Eq. (5), expressed as a percentage, is equivalent to Eq. (8-6) of the ATC-40 report (ATC 1996a). The total equivalent viscous damping ratio, ξ_{Total}, is equal to the inherent viscous damping [assumed to be 5% as per ATC-40 Eq. (8-5)] plus the equivalent viscous damping from Eq. (5). In addition, the equivalent viscous damping may be modified by a damping modification factor, κ, which accounts for the deviation of the actual hysteresis loops from the idealized parallelogram associated with bilinear hysteresis [see Eq. (8-8) of the ATC-40 report (ATC 1996a)]. In this study, the value of κ is taken as its largest possible value (i.e., unity).

The equivalent natural period, T_{eq}, of the bilinear system is based on the secant stiffness at a given displacement (ductility) level and is given by (Chopra and Goel 1999):

$$T_{eq} = T_n \sqrt{\frac{\mu}{1 + \alpha\mu - \alpha}} \tag{6}$$

where T_n is the natural period associated with elastic behavior.

Inelastic Demand Spectrum Analysis

The Inelastic Demand Spectrum Analysis method is a displacement-based procedure that is essentially the same as the improved Capacity-Demand-Diagram Method proposed by Chopra and Goel (1999). The method is similar to the Capacity Spectrum Analysis method in that it requires the development of a *capacity curve* (obtained from pseudo-static pushover analysis) and a series of *demand curves* except that the demand curves, rather than being represented by equivalent inelastic response spectra, are represented by actual inelastic response spectra [see Figure 14(b)]. An inelastic *design* response spectrum may be obtained by reducing the elastic design spectrum by appropriate ductility-dependent reduction factors, R. In this study, to maintain consistency with the force-based linear dynamic analyses in which the demand corresponded to three selected earthquake records, the seismic demand is not represented by design spectra but rather is represented by the inelastic response spectra corresponding to each of the three selected earthquakes. The inelastic response spectrum is commonly developed in the form of a pseudo-yield acceleration response spectrum with the assumption of elastoplastic behavior and a range of ductility levels. The yield acceleration spectra are subsequently converted to the Acceleration-Displacement Response Spectrum (ADRS) domain wherein the displacement represents the maximum displacement (rather than the yield displacement). Once the capacity curve and demand curves (inelastic ADRS spectra) have been obtained, the seismic performance of the bridge is assessed by overlaying the curves. Typically, the yielding portion of the capacity curve will intersect the demand curves corresponding to several ductility levels. The point at which the capacity curve intersects a demand curve associated with the same level of ductility is the performance point which defines the spectral displacement demand. The displacement and base shear demand for the MDOF structure are obtained by inverting the relations given by Equations (1) and (2).

NONLINEAR DYNAMIC ANALYSIS

Nonlinear dynamic analysis typically involves the development of a complex mathematical model of the bridge wherein an effort is made to model nonlinear forms of behavior in a highly localized (rather than global) manner. The model is then subjected to time-histories of earthquake ground acceleration that may be either historical records or design spectrum compatible records. In either case, an attempt is made to capture the full time-history of the nonlinear structural response. Nonlinear dynamic analysis is problematic for routine application to typical highway bridges. For example, there are difficulties in developing reasonable models for all the nonlinear components of the bridge, the results often exhibit strong sensitivity to the details of the model, and the extensive output can be difficult to interpret while masking important aspects of the seismic response.

APPLICATION AND RESULTS OF SEISMIC ANALYSIS PROCEDURES

LINEAR DYNAMIC ANALYSIS

As explained previously, linear dynamic analysis procedures include response spectrum analysis and time-history analysis. In this project, both of these procedures are employed to examine the transverse and longitudinal seismic response of the bridge for both the Basic and Spring Supported Models. The seismic excitation was represented by three different historical earthquake records that were applied non-concurrently in each direction. For the response spectrum and time-history analyses, the seismic loading was represented by 5%-damped elastic response spectra and ground acceleration time-histories, respectively (see Figure 13). In both cases, the damping in the bridge was characterized by an assumed 5% modal damping for each mode of vibration.

Prior to the seismic analysis, an eigenvalue analysis was performed (using SAP2000, GT-STRUDL, and ADINA) resulting in the longitudinal and transverse natural periods and associated modal participating mass ratios (i.e., effective modal mass to total mass ratios) shown in Table 4. The first four mode shapes for each bridge configuration are shown in Figure 15. The maximum elastic moments (either top or bottom), axial force, and displacements at the center column of the bridge bent due to combined dead and seismic loads are shown in Tables 5 through 7 for application of each earthquake motion in the transverse and longitudinal directions. The response spectrum analysis results include contributions from all modes of vibration (65 and 76 modes for the Basic and Spring Support Models, respectively). Note that results are presented for the center column, rather than the outboard columns, to be consistent with the displacement-based analyses wherein the results are provided for a control node associated with the center column. Also note that the results shown in Tables 5 through 7 were obtained using GT-STRUDL, SAP2000, and ADINA, with each program producing identical results.

Table 4 Summary of Fundamental Natural Periods and Associated Participating Mass Ratios for Longitudinal and Transverse Directions.

		Basic Support Model	Spring Support Model
Natural Period (sec)	Transverse	0.170	0.271
	Longitudinal	0.716	0.187
Modal Participating Mass Ratios (%)	Transverse	85.6	95.9
	Longitudinal	91.9	97.7

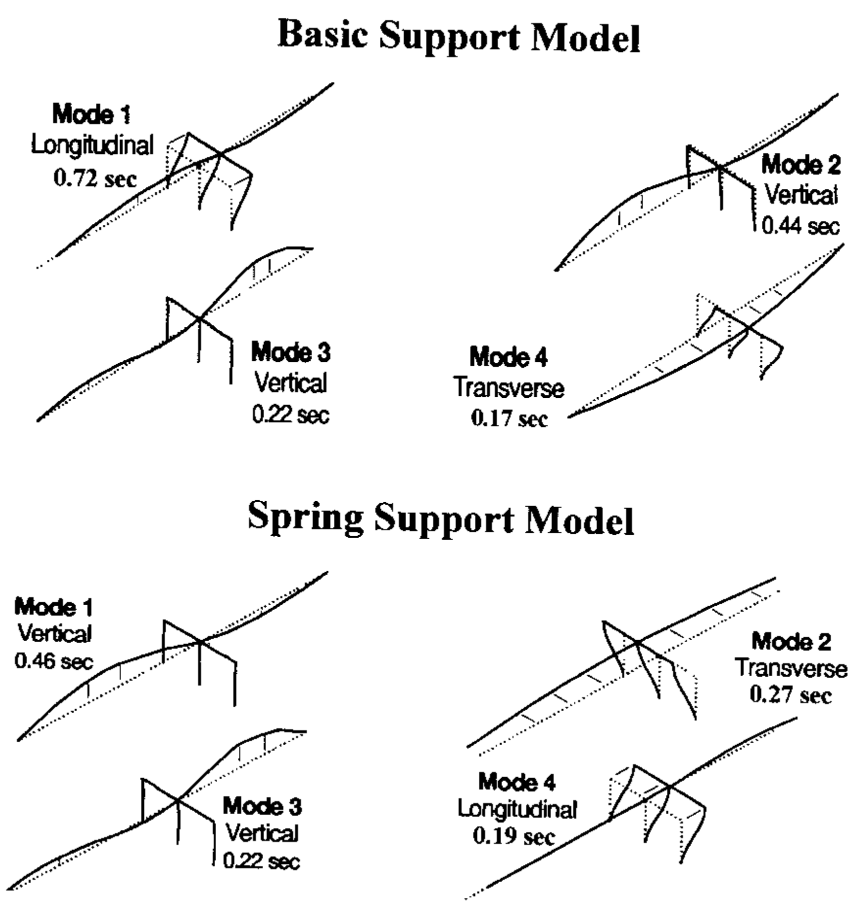


Figure 15 First Four Modes Shapes of Bridge for Basic and Spring Support Conditions as Obtained from Eigenvalue Analysis (adapted from Mast et al. 1996b).

Table 5 Elastic Seismic Response of Bridge Using Linear Dynamic Analysis for the 1940 El Centro Earthquake Record (includes dead load effect).

			Basic Support Model		Spring Support Model	
Analysis Method			RS[1]	TH[2]	RS[1]	TH[2]
Center Column	Moment (kip-ft)	TA[3]	1426	1330	1657	1692
	Axial Force (kip)		1098	1098	1096	1096
	Moment (kip-ft)	LA[4]	10220	10436	1010	1014
	Axial Force (kip)		1245	1277	1143	1144
	Displacement (in)	TA[3]	0.282	0.263	0.690	0.704
		LA[4]	2.577	2.572	0.307	0.306

[1]Response Spectrum Method; [2] Time-History Method; [3] Transverse Analysis; [4] Longitudinal Analysis.

Table 6 Elastic Seismic Response of Bridge Using Linear Dynamic Analysis for the 1949 Olympia Earthquake Record (includes dead load effect).

			Basic Support Model		Spring Support Model	
Analysis Method			RS[1]	TH[2]	RS[1]	TH[2]
Center Column	Moment (kip-ft)	TA[3]	968	991	1138	1250
	Axial Force (kip)		1098	1098	1096	1096
	Moment (kip-ft)	LA[4]	5179	5066	761	690
	Axial Force (kip)		1185	1198	1130	1124
	Displacement (in)	TA[3]	0.191	0.196	0.474	0.521
		LA[4]	1.405	1.386	0.204	0.173

[1] Response Spectrum Method; [2] Time-History Method; [3] Transverse Analysis; [4] Longitudinal Analysis

Table 7 Elastic Seismic Response of Bridge Using Linear Dynamic Analysis for the 1994 Sylmar Earthquake Record (includes dead load effect).

			Basic Support Model		Spring Support Model	
Analysis Method			RS[1]	TH[2]	RS[1]	TH[2]
Center Column	Moment (kip-ft)	TA[3]	1385	1279	1714	1741
	Axial Force (kip)		1098	1098	1096	1096
	Moment (kip-ft)	LA[4]	15862	14816	1004	1137
	Axial Force (kip)		1318	1371	1150	1144
	Displacement (in)	TA[3]	0.274	0.257	0.715	0.726
		LA[4]	3.894	3.575	0.306	0.336

[1] Response Spectrum Method; [2] Time-History Method; [3] Transverse Analysis; [4] Longitudinal Analysis

As shown in Tables 5 through 7, the elastic seismic response is smallest for the Olympia Record and tends to be largest for the Sylmar Record. This result is consistent with the peak ground acceleration associated with the three earthquake records (see Table 2). Also note that the response for the Basic and Spring Support Conditions is quite different. This is expected due to the significant difference in the natural periods of the two cases, particularly in the longitudinal direction (see Table 4). A comparison of the response spectrum and time-history analyses results reveals that the elastic seismic response of the bridge is similar in both cases with maximum errors being 11.7%, 3.9%, and 17.9% for moment, axial force, and displacement, respectively, where the results from the time-history analysis are regarded as "correct" results.

The column axial forces in the transverse direction are due to the dead load only since, for seismic loading in the transverse direction, the superstructure deforms in its own plane as a deep beam (see Figure 7) and thus does not induce any axial load in the columns. For the longitudinal direction, the seismic loading increases the axial force in the columns due to the out-of-plane flexural deformations of the superstructure (see Figure 7). Note that, as per the *Bridge Design Example* (Mast et al. 1996a), the dead load axial forces are due only to the self-weight of the bridge (i.e., the secondary effects due to post-tensioning are not included). The bridge model used in this study employs a spine-type configuration for the superstructure (see Figure 9) and, as a result, the self-weight is not realistically distributed to each of the bent columns. Based on the actual geometry of the superstructure and column bent (see Figure 2), the column axial forces were estimated by summing the vertical reactions at the base of the columns and equally dividing the sum among the three columns. The final result is that, for the Basic and Spring Support Models, the center column carries a dead load of 1098 and 1096 kips, respectively.

Upon review of the magnitudes of the moments and axial forces, it is evident that inelastic behavior of the columns is most likely to occur for earthquake loading (El Centro and Sylmar Records) in the longitudinal direction for the Basic Support Model. Consequently, nonlinear static analyses will only be performed for the aforementioned conditions. Furthermore, only the fundamental mode of vibration will be considered in the nonlinear static analyses since the natural period of the fundamental mode (0.72 sec) is less than one second (ATC 1996a) and the modal participating mass ratio for the fundamental mode was 91.9% which is greater than the typical minimum of 90%.

NONLINEAR STATIC ANALYSIS

Pushover Curve

The pushover curve was obtained via nonlinear pseudo-static pushover analysis using SC-Push3D, SAP2000, and GT-STRUDL. Each of the aforementioned programs has specific utilities for performing pushover analysis. ADINA was not used to generate the pushover curve since it does not contain a specific utility for performing pushover analysis.

The significantly larger column forces that are expected to develop for longitudinal excitation as compared to transverse excitation (since the abutments resist most of the force for transverse excitation) is evident by inspection of the elastic moments in Tables 5 and 7 for the El Centro and Sylmar records, respectively. Thus, as explained above, for purposes of illustrating the nonlinear static analysis procedures, the nonlinear pushover analyses were performed only in the longitudinal direction of the bridge.

SC-Push3D

As explained previously, SC-Push3D is a three-dimensional finite element program that is capable of performing linear and nonlinear psuedo-static pushover analysis of frame-type structures. Four element types are available to model the structural members of a bridge: a linear beam element, a nonlinear beam element, a linear pile element, and a linear spring element.

The pushover curve of a structure is determined in two steps: First, a linear analysis is performed under the effect of static loads (e.g., gravity loads). Second, a nonlinear analysis is performed under the effect of pseudo-static lateral seismic forces. In the latter step, yielding is assumed to take place in concentrated plastic hinges located at the ends of each element. Thus, inelastic behavior is assumed to occur over the entire cross-section rather than at individual fibers. For the bridge model used in this study, the potential plastic hinges were restricted to the columns, while the elements representing the superstructure were assumed to remain elastic.

The behavior of the plastic hinges is characterized by a yield surface and a moment-rotation relation. The yield surface defines the interaction between axial force, weak and strong bending moment, and torque. To allow for the behavior of different materials, one user-defined and three different standard yield surfaces are available. One of the yield surfaces (Type 2) is particularly well suited to reinforced concrete column sections. The moment-rotation relation is defined to be rigid-elastic/plastic wherein elastic post-yielding bending stiffness may correspond to either strain hardening or softening. The plastic hinge length is defined as an equivalent length over which the curvature is considered to be constant and equal to the curvature at the critical (end) section of the member. The curvature is assumed to produce the same plastic rotation as occurs in the real member, where plastic curvature reduces with distance from the critical section. The moment-curvature relation for the columns of the bridge was obtained using the program SEQMC (SEQMC 1998) and is shown in Figure 16. The moment-curvature analysis is based on Mander's confined concrete model (Mander et al. 1988). The idealized bi-linear moment-curvature relation shown in Figure 16 was obtained using the following anchor points: 1) the theoretical first yield in the reinforcement; 2) the ideal section moment capacity (i.e., the idealized yield point); and 3) the ultimate limit state (i.e., the point at which the confined compression strain is equal to 2.2%). From Figure 16, it is apparent that the post-yielding stiffness of the column can be reasonably approximated to be linear with zero slope. A total of 16 parameters were specified to define the plastic hinge within SC-Push3D (see Table 8). Note that, as per Priestley et al. (1996), the flexural strength of the plastic hinges is based on material properties expected in the field at the so-called characteristic age of the material at the time of the seismic event. Specifically, the nominal concrete compressive strength (4 ksi) and the reinforcement yield strength (60 ksi) are increased by 30% and 10%, respectively, and the unconfined ultimate compression strain and the confined spalling strain are taken as 0.3% and 0.5%, respectively.

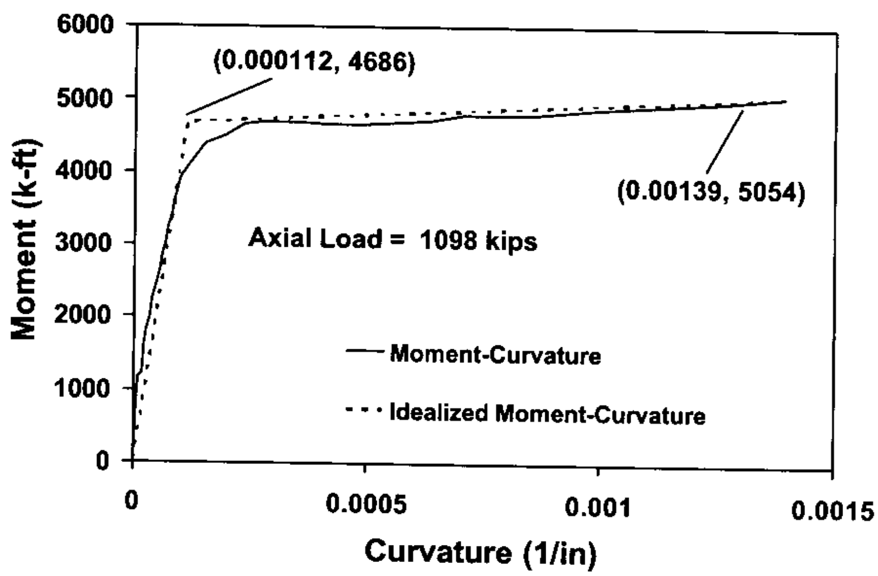


Figure 16 Moment-Curvature Relationship for the Columns of the Bridge Model.

The next step in the pushover analysis is to define the pushover load pattern. Two options are available to define the loading pattern in SC-Push3D: displacement-control using a displacement pattern or force-control using a force pattern consisting of concentrated or distributed forces. The displacement pattern is typically selected as that corresponding to the fundamental mode shape in the direction under consideration while the force pattern typically corresponds to a code-based static force distribution wherein the code-based static force distribution is intended to produce deformations that are approximately proportional to the fundamental mode shape. Since both load patterns are typically related to the fundamental mode shape, it is generally not clear which load pattern is more appropriate. However, the displacement-control method is regarded as most suitable when a predominant displacement pattern can be readily identified as is typically the case for bridges. Furthermore, for seismic loading the lateral distribution of forces will depend on the displaced shape, which will change as inelasticity develops in the structure (SC Solutions 1998). The use of displacement-controlled analysis does not require that the displacement be defined at all nodes; rather, the displacement may be applied at only a few nodes that dominate the response (e.g., nodes that dominate the fundamental mode response). In this study, the displacement-control method was used to obtain the pushover curve for the bridge by applying a prescribed maximum displacement of 12.0 in. in the longitudinal direction at the control node located at the intersection of the top of the center column and the superstructure (see Figure 9).

Having defined the pushover displacement pattern, the displacement pattern is applied to the bridge in a monotonically increasing fashion until the specified level of displacement is achieved. During application of the displacement pattern, the bridge deforms laterally and inelastic behavior at potential column hinge locations is monitored. The resulting base shear versus control node displacement represents the pushover curve.

As plastic hinges develop, the lateral stiffness of the bridge reduces and the pushover curve becomes nonlinear.

Table 8 Values of Parameters used to Define Plastic Hinge for SC-Push3D.

Description of Parameter	**Value**
Reinforcement bar yield strength (ksi)	66
Diameter of longitudinal reinforcement (in)	1.41
Tensile yield strength of cross-section (k)	2265
Compressive yield strength of cross-section (k)	10112
Compression force on the yield surface maximizing moments about the y- and z-axis of the cross-section (k)	3222
Maximum yield moment about y-axis on the yield surface (k-ft)	5208
Maximum yield moment about z-axis on the yield surface (k-ft)	5208
Maximum yield torque about z-axis on the yield surface (k-ft) [Value is zero since torque is not considered.]	0.0
Tensile ultimate strength of the cross-section (k)	3398
Compressive ultimate strength of the cross-section (k)	11244
Ultimate yield moment about y-axis (k-ft)	5376
Ultimate yield moment about z-axis (k-ft)	5376
Ultimate yield torque with zero axial force (k-ft) [Value is zero since torque is not considered.]	0.0
Post-yielding bending stiffness about y-axis (k-ft^2) [Value is approximated as zero.]	0.0
Post-yielding bending stiffness about z-axis (k-ft^2) [Value is approximated as zero.]	0.0
Type of yield surface	2

During application of the control node displacement, either local or global collapse may occur. A local collapse is associated with reaching the curvature capacity at a hinge resulting in a hinge that can no longer carry gravity load. Note that the bridge may be globally stable under the local collapse and, in this case, the failed element must be manually removed from the bridge model before proceeding (i.e., SC-Push3D will provide notification that the ultimate flexural strength at a hinge is reached but will not automatically remove the element). A global collapse is associated with forming sufficient plastic hinges which result in a mechanism. Note that SC-Push3D does not monitor the occurrence of global collapse mechanisms; rather the user must check for this possibility.

The pushover curve obtained from the pushover analysis using SC-Push3D is shown in Figure 17. As explained previously, the pushover analysis is preceded by a static analysis in which only the dead loads are applied. The application of the dead loads (self-weight) resulted in a control node displacement of 0.28 in. in the negative direction with zero base shear force. After the dead loads were applied, the first step of the nonlinear pushover analysis was performed with linear response occurring up to a control node displacement of 0.76 in. and a base shear force of 956.0 kips. In the second step, the displacement at the control node was increased until plastic hinges formed at the top of the outboard columns at a displacement of 0.86 in. and a base shear force of 1011.8 kips. In the third step, the displacement at the control node was increased until a plastic hinge formed at the top of the center column at a displacement of 0.96 in. and a base shear force of 1051.1 kips. In the fifth step, the displacement at the control node was increased until plastic hinges formed at the bottom of both the outboard and center columns at a displacement of 1.15 in. and a base shear force of 1089.4 kips. The analysis was continued until a local collapse occurred. In this case, the curvature at the top of the center column exceeded the ultimate curvature at a displacement of 2.34 in. and a base shear force of 1095.2 kips.

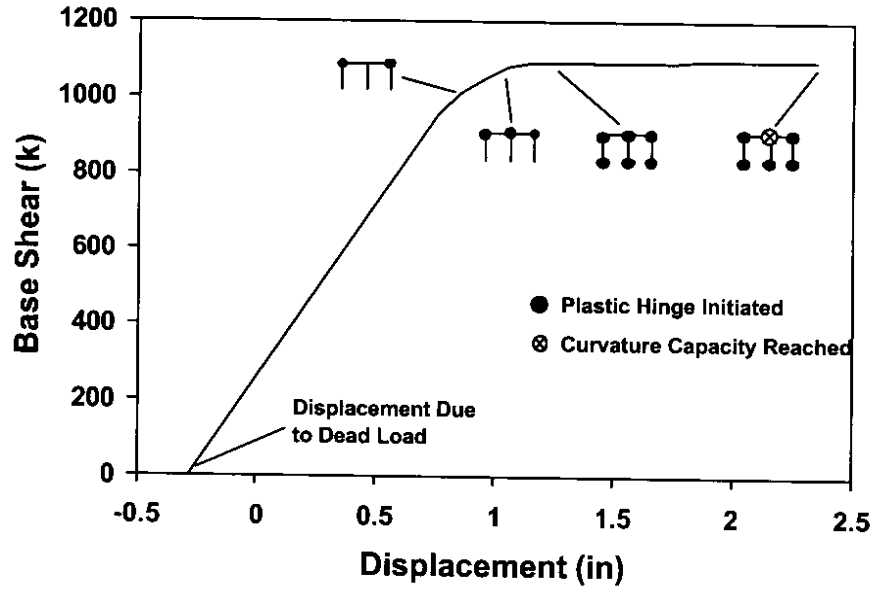


Figure 17 Pushover Curve from Analysis in the Longitudinal Direction for Basic Support Conditions using SC-Push3D.

SAP2000

As explained previously, SAP2000 is a general purpose finite element analysis program for static and dynamic analysis of two- and three-dimensional linear and nonlinear structures with a particular emphasis on dynamic loading and earthquake loading. The particular program used for this study, SAP2000 Nonlinear, is capable of performing pseudo-static nonlinear pushover analysis and nonlinear time-history analysis.

Prior to performing the pushover analysis, SAP2000 requires that a static analysis of the bridge be performed. The intent of requiring the static analysis is to encourage an examination of the bridge behavior under static loads to identify possible problems with the computer model.

For pushover analysis, nonlinear behavior is assumed to occur within frame elements at concentrated plastic hinges with default or user-defined hinge properties being assigned to each hinge. The default hinge types include an uncoupled moment hinge, an uncoupled axial hinge, an uncoupled shear hinge, and a coupled axial force and biaxial bending moment hinge. The latter is a hinge which yields based on the interaction of axial force and bending moments at the hinge location. In addition to the default and user-defined hinge properties, there are generated hinge properties. The default or user-defined hinge properties are assigned to frame elements and SAP2000 then creates, for each hinge, a different generated hinge property which is used in the pushover analysis. The generated hinge properties make use of the frame element section information and the default or user-defined hinge properties to fully define the plastic hinge properties. The main advantage of using default or user-defined hinge properties combined with generated hinge properties is that the process of defining a large number of hinge properties is simplified.

In this study, the coupled axial force and biaxial bending moment hinge (P-M2-M3 hinge) was assigned to the ends of the columns of the bridge model. The default hinge properties are section-dependent and are typically based on the nonlinear modeling parameters given in Table 9-6 of the ATC-40 document [Note that, although the ATC-40 nonlinear modeling parameters are characterized by performance levels associated with building behavior (i.e., Immediate Occupancy, Life Safety, and Collapse Prevention), the performance levels are useful for interpreting the pushover response of bridge structures]. The hinge properties are defined through the definition of the moment-rotation relation, the interaction surface, and acceptance criteria. The moment-rotation relation, which can either be user-defined or determined by SAP2000 based on section properties, characterizes the hinge deformation curve in the moment-rotation plane where the moment and rotation are normalized by the yield moment and yield rotation, respectively. In the case where the moment-rotation relation is determined by SAP2000, the yield rotation is automatically determined based on the defined material properties and the post-yielding stiffness is taken as 5% of the elastic stiffness. The interaction surface can be defined using the methodology presented in ACI 318-95 (ACI 1995) or by developing a user-defined interaction surface which is defined by a number of equally spaced axial force-moment curves between 0 and 360 degrees. The acceptance criteria are deformations that have been normalized by the yield deformation for immediate occupancy, life safety, and collapse prevention performance levels. For the P-M2-M3 hinge, the acceptance criteria deformation is the plastic rotation normalized by the yield rotation, which can either be input by the user or calculated by SAP2000 based on section properties.

Once the plastic hinge properties have been defined, the next step is to define the static pushover cases and the type of pushover analysis to be performed. In general, a pushover analysis consists of more than one pushover case. Typically, the analysis might consist of two load cases. The first load case would apply gravity load to the bridge and the second load case would apply a specific lateral load pattern to the bridge. In SAP2000, there are four different types of lateral load patterns to describe the distribution of loads on the structure for a pushover load case. The load patterns are: 1) uniform acceleration (i.e., an equal acceleration imposed simultaneously on each lumped mass in the direction of one of the global axes); 2) a force that is proportional to the product of a specified mode shape multiplied by its circular frequency squared multiplied by the mass tributary to a node; 3) an arbitrary static load pattern; and 4) a combination of the aforementioned patterns. Regarding the type of pushover analysis, SAP2000 can perform both force-controlled analysis where the pushover proceeds to the full load value defined by the load pattern or displacement-controlled analysis where the pushover proceeds to the specified displacement in the specified direction at the specified node. In this study, the pushover curve was obtained using SAP2000 by first analyzing the bridge under the effect of dead load and then pushing the bridge longitudinally using the uniform acceleration load pattern until either the prescribed maximum displacement of 6.0 in. (displacement-controlled analysis) was achieved at the control node (located at the intersection of the top of the center column and the superstructure; see Figure 9) or the bridge failed. Note that SAP2000 automatically stops the analysis when a plastic hinge reaches its curvature capacity (by default, defined to be a plastic hinge rotation equal to six times the yield rotation), requiring the user to check if a sufficient number of plastic hinges have developed to render the structure globally unstable.

The results from pushover analysis can be displayed in a variety of formats including the pushover-deformed shape, the sequence of pushover hinge formation, frame element forces at each step of the pushover, and the capacity curve (i.e., the pushover curve converted to ADRS format). The pushover curve is converted by SAP2000 to ADRS format based on Equations (8-1) through (8-4) in the ATC-40 document which are equivalent to Equations (1) through (4) in this report.

The pushover curve obtained from the pushover analysis using SAP2000 is shown in Figure 18. Major events, which include the initial formation of plastic hinges and the achievement of pre-defined performance levels, are indicated on the pushover curve. As explained previously, the pushover analysis is preceded by a static analysis in which only the dead loads are applied. The application of the dead loads (self-weight) resulted in a control node displacement of 0.27 in. in the negative direction with zero base shear force. The deformed shape of the bridge under the dead load, as presented by SAP2000, is shown in Figure 19(a). After the dead loads were applied, the nonlinear pushover analysis was performed with plastic hinges first developing at the top of the columns at a displacement of 0.80 in. and a base shear force of 999.9 kips. In the next step, the plastic hinges formed at the bottom of columns at a displacement of 0.91 in. and a base shear force of 1032.9 kips. The bridge was loaded further until the Immediate Occupancy (IO) performance level was achieved at the top of the columns at a displacement of 1.15 in. and a base shear force of 1035.8 kips. Upon further loading, the Immediate Occupancy (IO) performance level was achieved at the bottom of the columns followed by the Life Safety (LS) performance level at the top of the columns at a displacement of 2.47 in. and a base shear force of 1052.1 kips. Finally, loading was applied until the Life Safety (LS) performance level was achieved at the bottom of the columns followed shortly thereafter by reaching the Collapse Prevention performance level at the top of the columns at a displacement of 4.31 in. and a base shear force of 1074.8 kips. At this point, the hinges at the top of the columns are regarded as having reached their curvature capacity and the analysis is stopped (actually, SAP2000 takes one additional step wherein the strength reduces significantly). The deformed shape of the bridge at the collapse limit state, as presented by SAP2000, is shown in Figure 19(b). Note that the condition of each plastic hinge in Figure 19(b) is indicated by its color as per a color code that is provided on the bottom of the window. Furthermore, the use of color-codes to define hinge behavior allows the user to observe the evolution of damage in the structure.

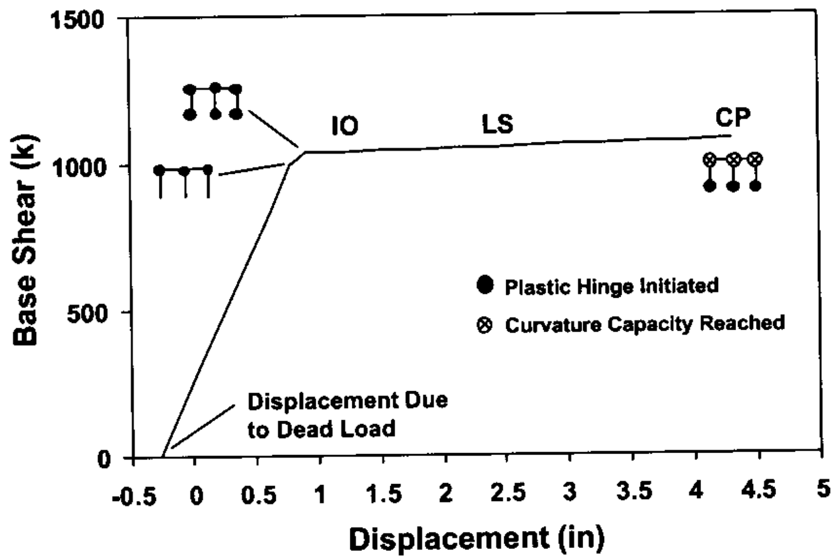
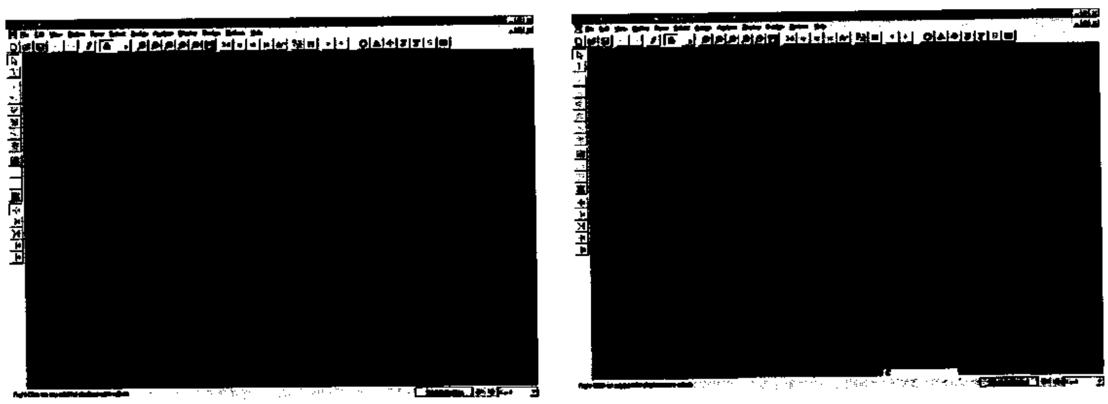


Figure 18 Pushover Curve from Analysis in the Longitudinal Direction for Basic Support Conditions using SAP2000.



(a) Dead Load Only **(b) Dead + Lateral Load**

Figure 19 Deformed Shape of Bridge as Presented by SAP2000.

GT-STRUDL

GT-STRUDL is a general-purpose finite element analysis program for static and dynamic analysis of linear and nonlinear structures. The version of this software used in this study (Version 25) includes a special utility for performing pushover analysis.

Prior to performing the pushover analysis, GT-STRUDL recommends that a static analysis of the bridge be performed. The intent of requiring the static analysis is to encourage an examination of the bridge behavior under static loads to identify possible problems with the computer model.

For pushover analysis, nonlinear behavior is assumed to occur at plastic hinges located at the ends of members. A distributed plasticity model is used to characterize the plastic hinge behavior. In this type of plastic hinge model, the hinge region is subdivided into concrete and steel fibers. The hinge behavior is defined by specifying the stress-strain relation of the fibers, integrating over the cross-section to obtain the moment-curvature relation, and integrating over the hinge length to obtain the moment-rotation relation. For reinforced concrete sections, the plastic hinge properties are defined by START/END specifications, fiber specifications, and reinforced concrete specifications. The START/END specifications define the member ends at which plastic hinges are permitted to form while the fiber specifications define the geometric characteristics of the plastic hinge. For a circular cross section, the following parameters define the geometric characteristics of the plastic hinge: number of fiber divisions through the radius of the cross-section, number of fiber divisions around the circumference of the cross section, and the plastic hinge length. To accurately model the behavior of the plastic hinge, a larger number of fiber divisions should be used. The reinforced concrete specifications define properties such as cross section shape (limited to rectangular or circular shapes), cross-section dimensions, principal concrete material properties, and reinforcing steel locations. The principal reinforced concrete material properties are the unconfined compressive strength, the ultimate unconfined compressive strain, and the tensile yield strength of the transverse reinforcement. The compression stress-strain behavior of the concrete is based on the confined concrete compression stress-strain model presented by Mander et al. (1988). The data defining the location of reinforcing steel includes: bar specification; arrangement of bars; transverse reinforcing tensile yield strength, bar size, and spacing; and clear cover.

The next step in the pushover analysis is to define the static pushover load cases. Two types of loads need to be defined in this step, a constant load for static analysis and an incremental load for pushover analysis. The pushover analysis is then performed through a series of steps in which, at each step, a new total applied load is computed by adding a load increment to the previous applied total load. For the bridge model in this study, the constant load represents the self-weight of the structure while the incremental load was a uniform traction load applied along the longitudinal axis of the superstructure (see Figure 9). Note that, in GT-STRUDL, there is no convenient tool for applying a displacement pattern in lieu of a load pattern.

The results of the pushover analysis are available in a variety of formats. The nonlinear analysis output commands in GT-STRUDL include *print nonlinear effects, list plastic hinge displacements, list plastic hinge status*, and *list pushover analysis ductility ratio*. The *print nonlinear effects* command provides a list of members which experience nonlinear behavior while the *list plastic hinge displacements* command provides a list of displacements at the start and end of members which experience nonlinear behavior. The *list plastic hinge status* provides a list containing the percentage of plastic hinge formation at the start and end of members which contain plastic hinges. The *list pushover analysis ductility ratio* provides the global ductility ratio for a particular degree of freedom at a selected node. The ductility ratio is a function of the yield deformation, which is determined by GT-STRUDL based on the load at which the initiation of plastic hinge formation was first detected throughout the structure. Consequently, the ductility ratio is sensitive to the fiber grid geometry defined for the plastic hinge and the magnitude of the load increment.

The pushover curve obtained from the pushover analysis using GT-STRUDL is shown in Figure 20. As explained previously, the pushover analysis is preceded by a static analysis in which only the dead loads are applied. The application of the dead loads (self-weight) resulted in a control node (located at the intersection of the top of the center column and the superstructure; see Figure 9) displacement of 0.28 in. in the negative direction with zero base shear force. After the dead loads were applied, the nonlinear pushover analysis was initiated with plastic hinges forming at the top of the outboard columns at a displacement of 0.06 in and a base shear of 306.1 kips. The percentage of plastic hinge formation at this stage of loading was 27% at the top of the outboard columns (i.e., 27% of the total number of fibers have yielded). The lateral load was then increased until the extreme fibers of the plastic hinges at the top and bottom of the outboard columns as well as the top of the center column exceeded the ultimate compressive strain. The percentage of plastic hinge formation at this stage of loading was 67% and 29% for the hinges at the top and bottom of the outboard columns, respectively, and 10% at the top of the center column. The pushover analysis was automatically terminated by GT-STRUDL when global instability was detected at which time the percentage of plastic hinge formation was 84% at the top and bottom of the outboard columns and 73% at the top and bottom of the center column. The displacement and base shear at the point of global instability was 1.14 in and 1145.7 kips.

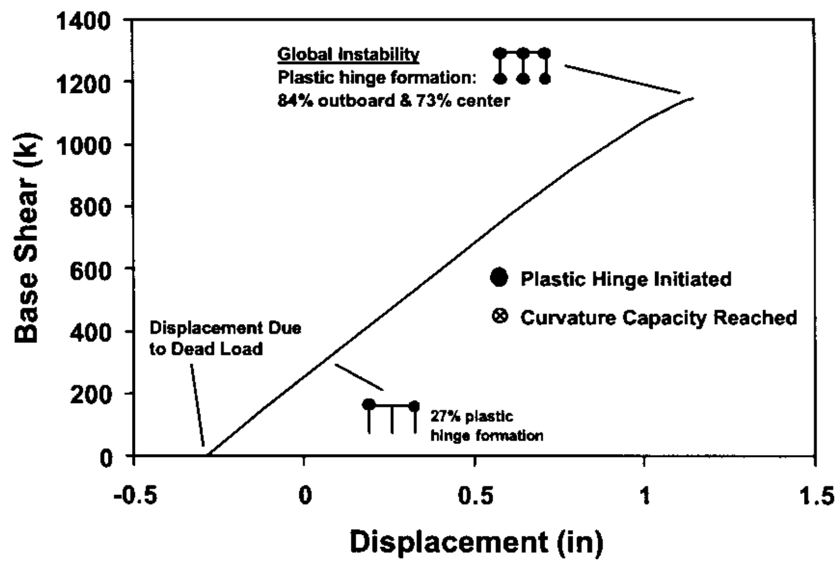


Figure 20 Pushover Curve from Analysis in the Longitudinal Direction for Basic Support Conditions using GT-STRUDL.

Comparison of Pushover Curves

The pushover curves as obtained from SC-Push3D, SAP2000, and GT-STRUDL are compared in Figure 21. It is evident that the three software programs produce essentially identical results for the initial linear dead load analysis and the elastic portion of the pushover analysis. However, the results are significantly different for the nonlinear portion of the pushover analysis, particularly with respect to the displacement corresponding to failure. The difference in failure displacements is due to the difference in how failure is defined by each program. For SC-Push3D, no definition of failure is provided and thus the user must monitor the structural response and decide when failure has occurred. For this study, the SC-Push3D analysis was stopped at the development of a local collapse mechanism (i.e., the curvature at the top of the center column exceeded the ultimate curvature). For SAP2000, failure is defined as occurring at the first step at which the Collapse Prevention performance level is achieved at any plastic hinge (i.e., the curvature capacity is reached at one of the hinges). For GT-STRUDL, failure is defined as the occurrence of a global collapse mechanism. For the bridge under study, all three software programs indicate that a global collapse mechanism occurs at a control node displacement of approximately one inch. However, the bridge is capable of continuing to carry lateral load, with SAP2000 indicating the displacement capacity is approximately 4.3 inches, at which time the plastic hinges at the top of all three columns reach their curvature capacity. Interestingly, a re-run of the SC-Push analysis wherein the definition of failure was revised to be consistent with the default definition used by SAP2000 (i.e., an ultimate rotation of six times the yield rotation) results in a failure displacement that is approximately equal to that obtained with SAP2000. Note that, for displacements between about 0.25 and 1 inch, the pushover curve obtained from GT-STRUDL is distinctly different from the curves obtained from SC-Push3D and SAP2000. The primary reason for this difference is the model used for the plastic hinges. In SC-Push3D and SAP2000, a concentrated plasticity model is used for the plastic hinges whereas in GT-STRUDL a distributed plasticity model is used.

It should be recognized that the base shear corresponding to a global collapse mechanism can be readily estimated via simple plastic analysis of the bridge bent. For the longitudinal direction, a global collapse mechanism is achieved when six plastic hinges develop, three at the top and three at the bottom of the bent. Equating the external work due to the applied lateral force to the internal work due to the development of plastic hinges leads to a lateral force (or base shear) of six times the plastic moment strength divided by the height of the bent. Taking the plastic moment strength as that corresponding to the idealized yield point in the moment-curvature relation shown in Figure 16, the resulting base shear is 998.1 kips which is only 3.4% smaller than the global collapse mechanism base shear (1032.9 kips) obtained from SAP2000.

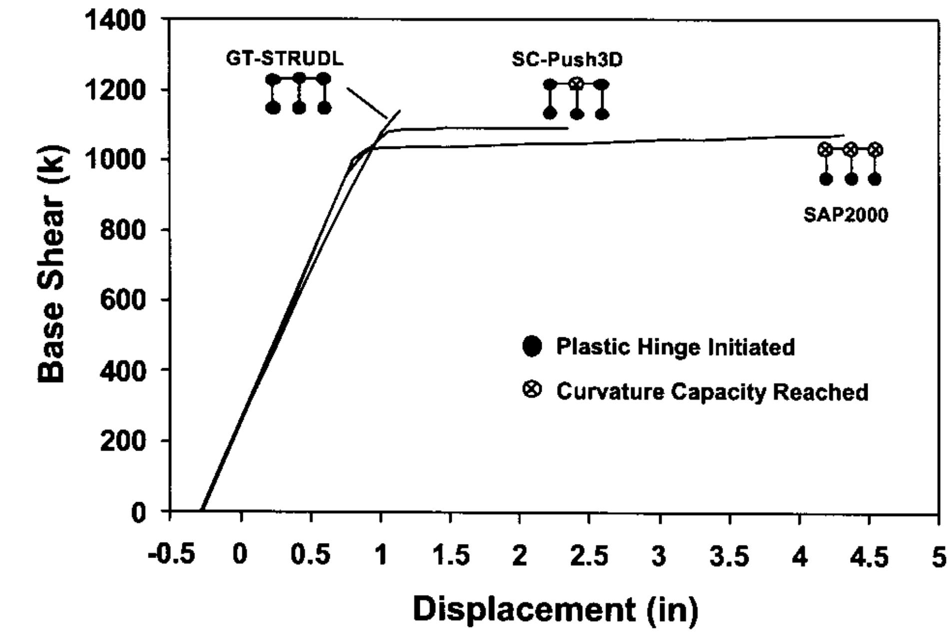


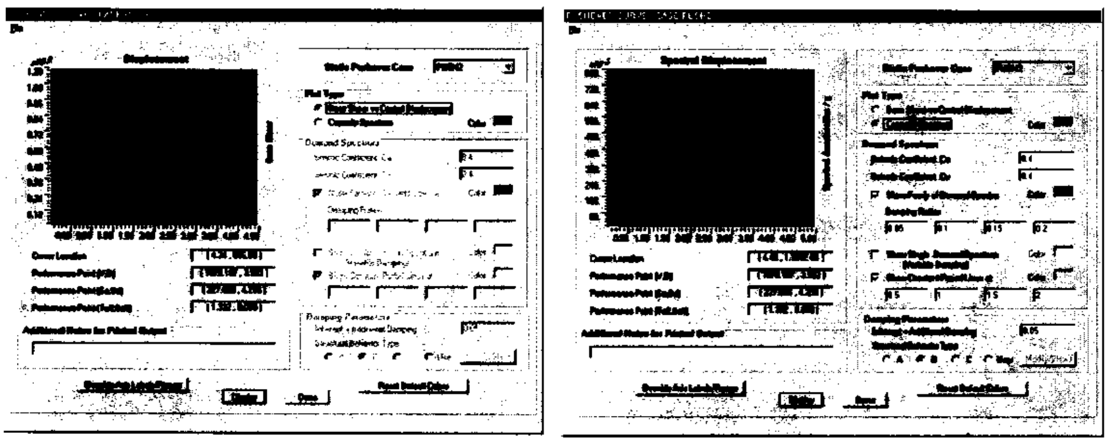
Figure 21 Comparison of Pushover Curves from Analysis in the Longitudinal Direction for Basic Support Conditions.

Capacity Curve

As explained previously, the pushover curve obtained from the pushover analysis is in the base shear - control node displacement domain for the MDOF system and must be converted to a capacity curve in the fundamental mode modal acceleration - modal displacement domain for the associated SDOF system [see Eq. (1) and (2)]. The modal participation factor, Γ_1, and effective modal mass, M_1^*, for the fundamental mode are determined using Eq. (3) and (4). The resulting values are 11.52 and 139.11 $k\text{-}s^2/ft$, respectively. The modal amplitude of the control node in the fundamental mode of vibration, ϕ_{N1}, is 0.0795. Note that the values of Γ_1 and ϕ_{N1} depend on how the mode shapes are normalized but the product, which appears in Eq. (2), is independent of how the mode shapes are normalized. Also, as mentioned previously, changes in the fundamental mode shape due to nonlinear response can be accounted for in applying the transformation expressed by Eq. (1) and (2). In this study, such changes in the fundamental mode shape were not considered. Focusing on the pushover curve obtained from SAP2000 (since SAP2000 has a utility for generating the capacity curve), the pushover curve and capacity curve, as presented by SAP2000, are shown in Figure 22. Note that the pushover curve shown in Figure 22(a) includes one additional step beyond that shown in Figure 18; this additional step being associated with the loss of strength of the plastic hinges at the top of the columns. In addition to obtaining the capacity curve using SAP2000, the capacity curve was obtained directly from the pushover curve shown in Figure 18 using Eq. (1) and (2) (see Figure 23). Interestingly, the capacity curve obtained directly from SAP2000 [Figure 22(b)] is not the same as that obtained by the authors [Figure 23(a)]. The difference between the two curves is due to the method by which the initial dead load displacement is handled. In SAP2000, the magnitude of the initial negative displacement is used to determine the initial point on the capacity curve. In contrast, and as explained in more detail below, the authors believe that a more appropriate approach is to retain the sign of the initial negative displacement but then remove the dead load effect prior to performing the capacity spectrum analysis.

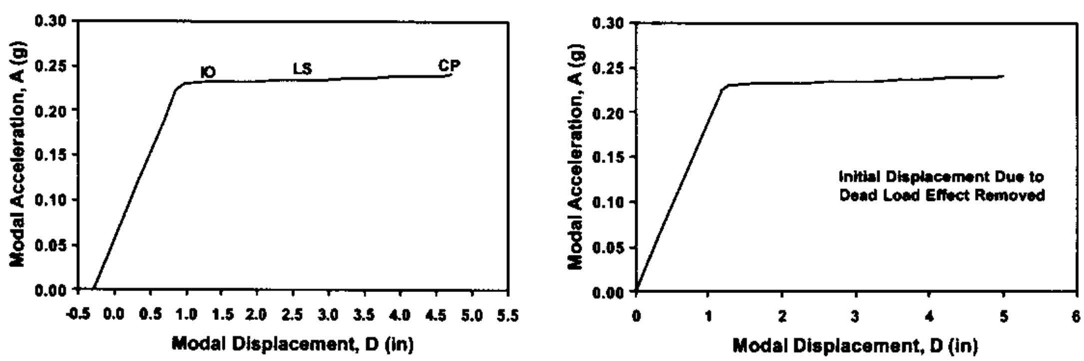
The capacity curve obtained by the authors through direct application of Eq. (1) and (2) is shown in Figure 23. The seismic performance evaluation of the bridge is based on comparing the capacity curve with a demand curve where, in the capacity spectrum method, the demand curve is in the form of an ADRS curve in which lines of constant natural period radiate out from the origin. In order to compare the capacity curve with the ADRS demand curve, the displacement due to dead load must be removed from the capacity curve so that the lines of constant effective natural period on the capacity curve coincide with the associated lines on the ADRS demand curve. Thus, for purposes of seismic performance evaluation, the lateral displacement due to dead load is temporarily removed [see Figure 23(b)]. The dead load displacement will be accounted for subsequent to the seismic performance evaluation.

For displacement-based performance evaluation, the capacity curve is overlaid with either an elastic or inelastic ADRS demand curve and, as a result, it is necessary to determine the ductility and total viscous damping at various points along the capacity curve. These properties are determined by replacing the capacity curve with a simplified bilinear curve; the simplified curve being generated based on various principles (e.g., equal area under the two curves). In this study, since the capacity curve is nearly bilinear, the bilinear curve was generated by simply extending the pre- and post-yielding regions of the capacity curve to find their common intersection point which was then defined as the yield point for the bilinear capacity curve. From the bilinear capacity curve, the post- to pre-yielding stiffness ratio and the ductility level at selected displacements is determined. Finally, the total viscous damping ratio at the selected displacements is obtained by adding the 5% inherent damping to the equivalent viscous damping ratio defined by Eq. (5). The results are shown graphically in Figure 24 where lines of constant effective natural period, determined using Eq. (6), radiate out from the origin. Note that the ductility levels and damping ratios shown in Figure 24 are associated with the points on the capacity curve that intersect the radial lines. Also, note that the total viscous damping ratios are convenient integer values (except for the failure point). This is because the displacements were purposefully selected such that the damping ratios would have these integer values.



(a) Pushover Curve (b) Capacity and Demand Curve

Figure 22 Results of Pushover Analysis as Presented by SAP2000.



(a) Dead Load Displacement Included **(b) Dead Load Displacement Removed**

Figure 23 Capacity Curve Generated from Pushover Curve Shown in Figure 18.

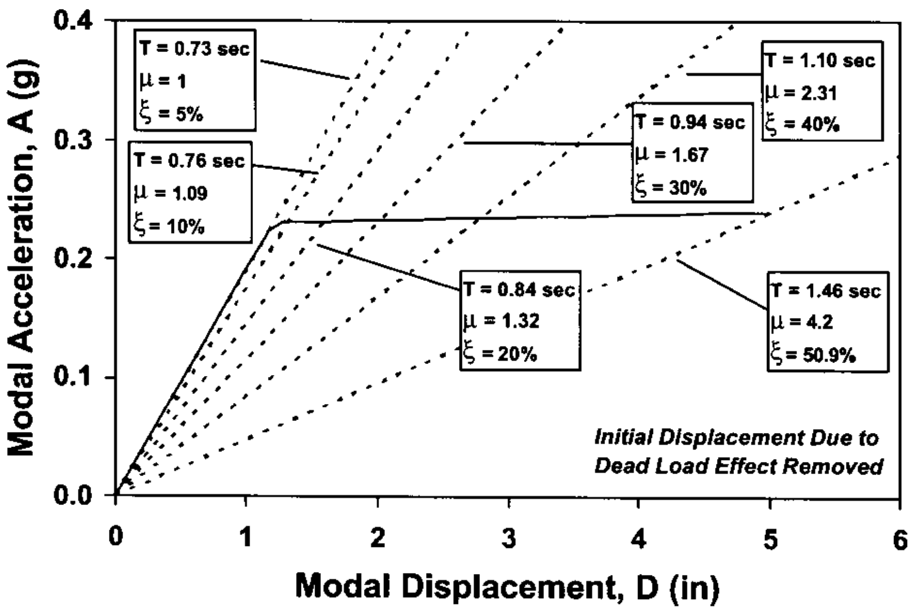


Figure 24 Capacity Curve with Effective Period, Ductility, and Total Damping Ratio Identified at Selected Displacements.

Demand Curve

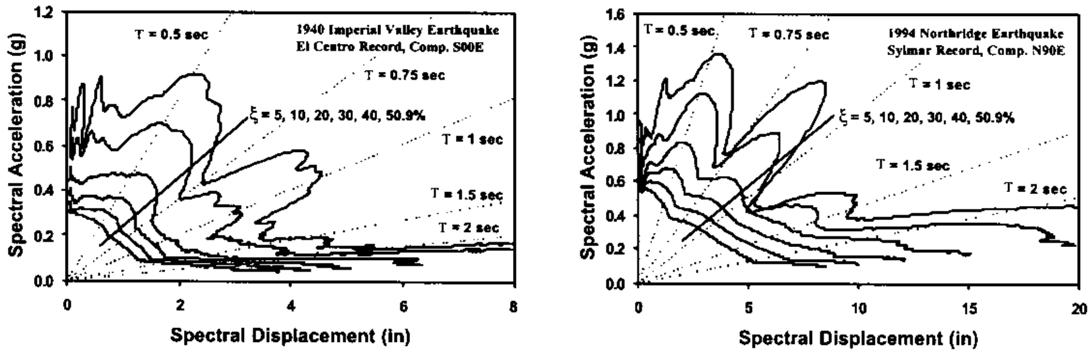
Equivalent Inelastic Response Spectra

As explained previously, in the Capacity Spectrum Analysis method, the seismic demand is represented by a series of Acceleration-Displacement Response Spectra (ADRS) for elastic SDOF systems. The 5%-damped elastic ADRS curve is modified to account for inelastic behavior by using higher levels of viscous damping, the resulting spectrums being regarded as equivalent inelastic response spectrums. As is evident in Figure 22(b), SAP2000 is capable of generating smoothed *design* ADRS curves that are based on the 1997 NEHRP Rehabilitation Guidelines (FEMA 1997a). The ADRS curves can be generated for four different viscous damping ratios with four different constant period lines being available for display. As explained previously, the seismic excitation for evaluation of the bridge is based on three historical earthquake records rather than design response spectra. Thus, the SAP2000 ADRS design curves were not utilized. In addition, it should be noted that SC-Push3D and GT-STRUDL do not have the capability of generating ADRS demand curves.

The ADRS demand curves, as generated by the authors, for damping ratios ranging from 5% to 50.9% are shown in Figure 25. The lowest damping level, 5%, corresponds to inherent damping associated with elastic behavior whereas the maximum damping ratio, 50.9%, corresponds to combined elastic and inelastic behavior at the Collapse Prevention performance level (see Figure 24). Recall that the spectral acceleration in the ADRS demand curves represents pseudo-acceleration, not actual acceleration.

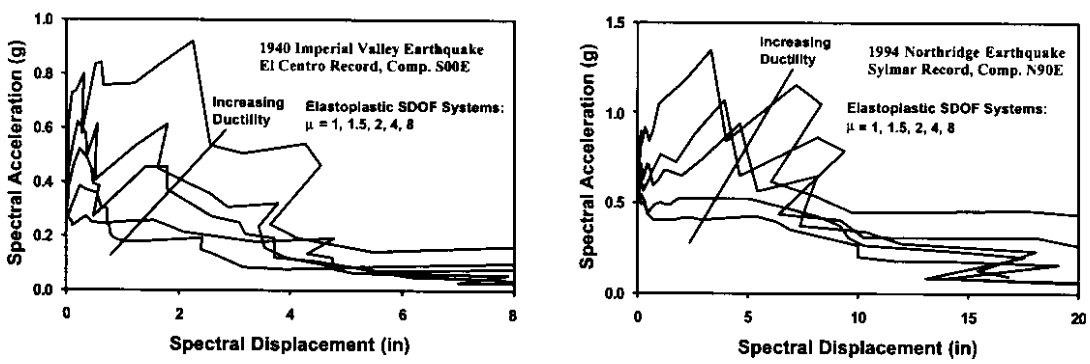
Inelastic Response Spectra

As explained previously, the Inelastic Demand Spectrum method represents the seismic demand using an inelastic ADRS spectrum for a SDOF system. In this study, the inelastic response spectra were generated assuming elastoplastic behavior and using standard procedures as described by, for example, Chopra (2001). The ADRS spectra for 5%-damped elastoplastic systems having a ductility ranging from 1 to 8 and subjected to the El Centro and Sylmar earthquake records are shown in Figure 26 for a natural period range of 0.02 to 3 seconds. Note that, due to the time-consuming process of generating inelastic response spectra, the natural period increment is not constant and may be somewhat large (up to 0.5 sec), giving rise to the somewhat jagged appearance of the spectrums. As mentioned previously, the spectral acceleration and spectral displacement in the inelastic ADRS spectrums represent pseudo-acceleration and maximum (not yield) displacement, respectively.



(a) El Centro Record **(b) Sylmar Record**

Figure 25 ADRS Demand Curves for a Range of Total Viscous Damping Ratios.



(a) El Centro Record (b) Sylmar Record

Figure 26 ADRS Curves for Elastoplastic SDOF Systems Subjected to El Centro and Sylmar Earthquake Records.

Capacity Spectrum Analysis

As explained previously, according to the Capacity Spectrum Analysis method, the seismic performance of the bridge can be evaluated by overlaying the capacity curve obtained from pushover analysis [see Figure 23(b)] with the elastic ADRS demand curves (see Figure 25). The overlaid curves are shown in Figure 27 and 28 for the El Centro and Sylmar records, respectively. Three procedures are presented in ATC-40 for identifying the performance point. The procedure utilized in this study is a graphical procedure that is essentially the same as that described as Procedure C in ATC-40. The procedure may be described as follows: For each viscous damping ratio, identify the point of intersection between the ADRS demand curve for that damping ratio and the radial line of constant period on the capacity curve associated with that damping ratio. These points have been indicated by solid circles in Figure 27 and 28. The performance point is that point which coincides with the capacity curve. In general, no points will exactly intersect with capacity curve. In this case, the performance point can simply be estimated by drawing a line through all of the points that have been identified and the location where the line intersects the capacity curve is the performance point. Alternatively, an intermediate trial damping ratio may be selected and the ADRS curve and associated constant period line constructed to determine if the trial damping ratio is associated with the performance point. As shown in Figure 27, for the El Centro record, the performance point is associated with a total viscous damping ratio of 20%. For the Sylmar record (see Figure 28), the performance point did not lie on one of the radial lines associated with a pre-selected damping ratios and thus trial damping ratios were employed until the performance point coincided with the radial line associated with a total viscous damping ratio of 48%.

The performance of the bridge should be evaluated based on its response as a MDOF structural system. The performance points identified in Figure 27 and 28 are associated with a SDOF fundamental mode response of the bridge. The associated MDOF response is determined by inverting Equations (1) and (2). The final results are given in Table 9 with due consideration given to the initial dead load displacement of negative 0.27 in. A comparison of the control node displacements shown in Table 9 with the pushover curve shown in Figure 18 indicates that, for the El Centro record, the bridge is predicted to just achieve the Immediate Occupancy (IO) performance level while, for the Sylmar record, the bridge performance is predicted to be approximately midway between the Life Safety (LS) and Collapse Prevention (CP) performance levels. The qualitative description of the bridge performance is summarized in Table 9.

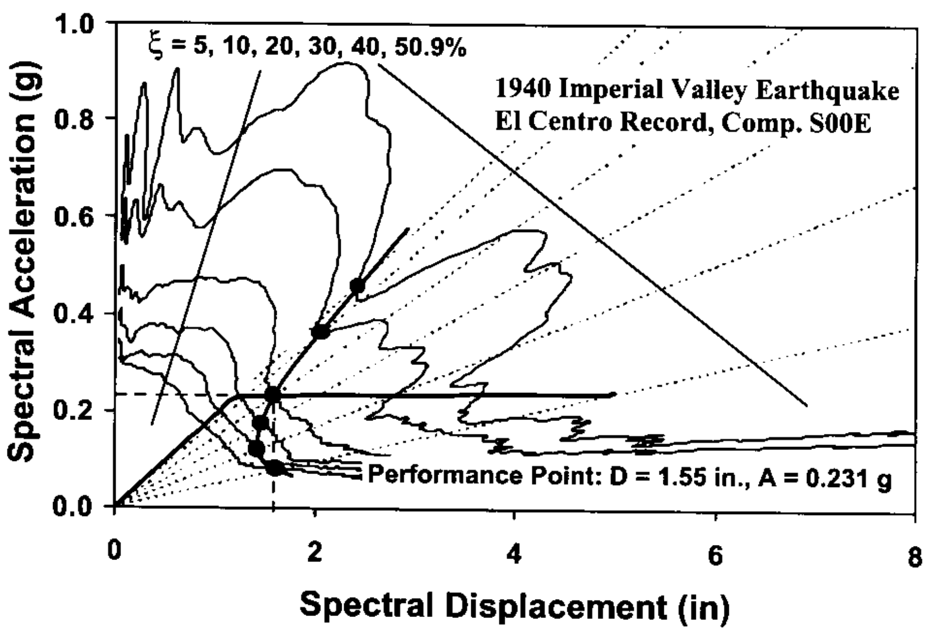


Figure 27 Graphical Performance Evaluation of Bridge Subjected to El Centro Record using Capacity Spectrum Analysis.

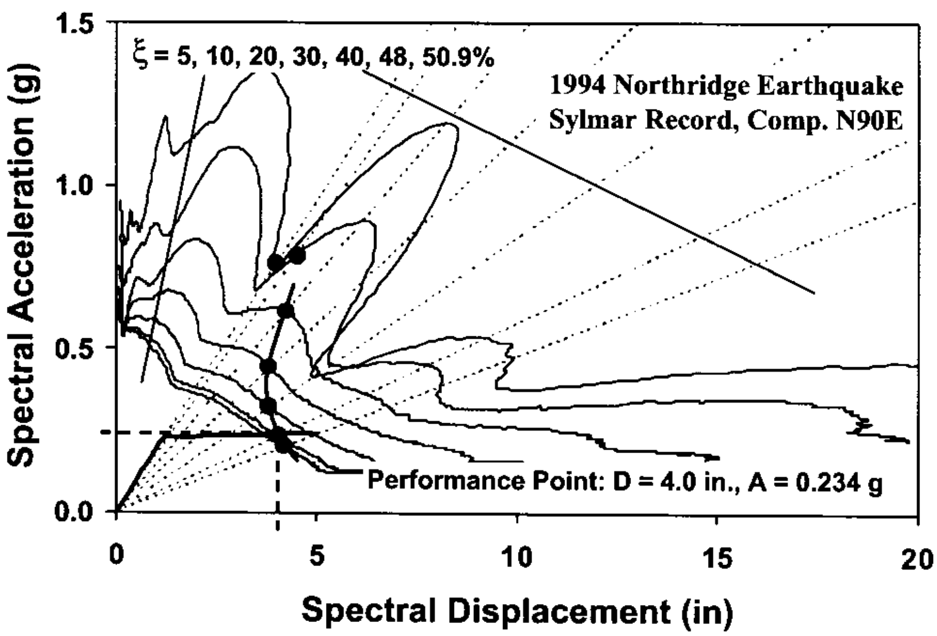


Figure 28 Graphical Performance Evaluation of Bridge Subjected to Sylmar Record using Capacity Spectrum Analysis.

Table 9 Summary of Bridge Response from Displacement-Based Seismic Analyses.

	Capacity Spectrum Analysis			Inelastic Demand Spectrum Analysis		
	u_N (in)	V_b (kips)	Performance	u_N (in)	V_b (kips)	Performance
El Centro	1.15	1034.7	IO	2.25	1034.7	LS
Sylmar	3.39	1048.2	LS/CP	---	---	Collapse

Inelastic Demand Spectrum Analysis

As explained previously, according to the Inelastic Demand Spectrum Analysis method, the seismic performance of the bridge can be evaluated by overlaying the capacity curve obtained from pushover analysis [see Figure 23(b)] with the inelastic ADRS demand curves (see Figure 26). The overlaid curves are shown in Figure 29 and 30 for the El Centro and Sylmar records, respectively. Recall that the Inelastic Demand

Spectrum Analysis is similar to the *Capacity-Demand-Diagram Method* described by Chopra and Goel (1999) in which an inelastic spectrum is utilized to characterize the demand while retaining the graphical appeal of the Capacity Spectrum Analysis method. The procedure utilized to identify the performance point parallels that described previously for the Capacity Spectrum method and may be described as follows: For each ductility level, identify the point of intersection between the inelastic ADRS demand curve for that ductility level and the radial line passing through the capacity curve and associated with that same ductility level. These points have been indicated as solid circles in Figure 29 and 30. The performance point is that point which coincides with the capacity curve. In general, no points will exactly intersect with capacity curve. In this case, the performance point can simply be estimated by drawing a line through all of the points that have been identified and the location where the line intersects the capacity curve is the performance point. Alternatively, for a more refined analysis, an intermediate trial ductility level may be selected and the ADRS curve and associated constant ductility line constructed to determine if the trial ductility ratio is associated with the performance point. As shown in Figure 29 and 30, none of the potential performance points lie on the capacity curve. However, due to the time consuming process of generating inelastic demand response spectra, a more refined analysis was not performed. The final performance points are indicated in Figure 29 and 30. It should be noted that any ductility levels can be selected for identifying potential performance points. For example, in this study, the ductility levels used in Figure 29 and 30 are not the same as the ductility levels shown on the capacity curve of Figure 24. As explained previously, the ductility levels shown in Figure 24 resulted from displacements that were selected to correspond to convenient values of the total viscous damping ratio.

The performance of the bridge should be evaluated based on its response as a MDOF structural system. The performance points identified in Figure 29 and 30 are associated with a SDOF fundamental mode response of the bridge. The associated MDOF response is determined by inverting Equations (1) and (2). The final results are given in Table 9, with due consideration given to the initial dead load displacement of negative 0.27 in. A comparison of the control node displacements shown in Table 9 with the pushover curve shown in Figure 23(a) indicates that, for the El Centro record, the bridge is predicted to just achieve the Life Safety (LS) performance level while, for the Sylmar record, the bridge is predicted to collapse.

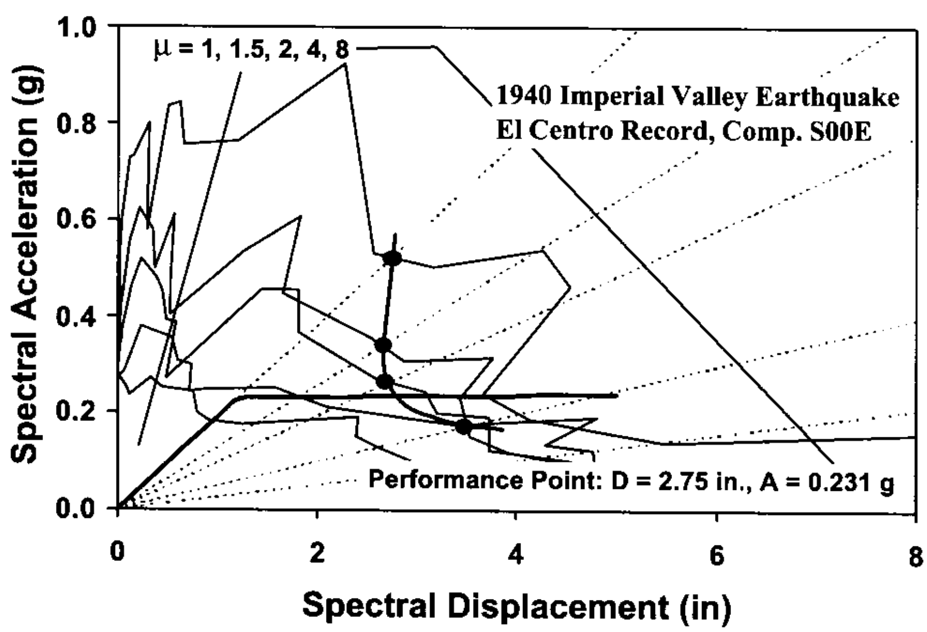


Figure 29 Graphical Performance Evaluation of Bridge Subjected to El Centro Record using Inelastic Demand Spectrum Analysis.

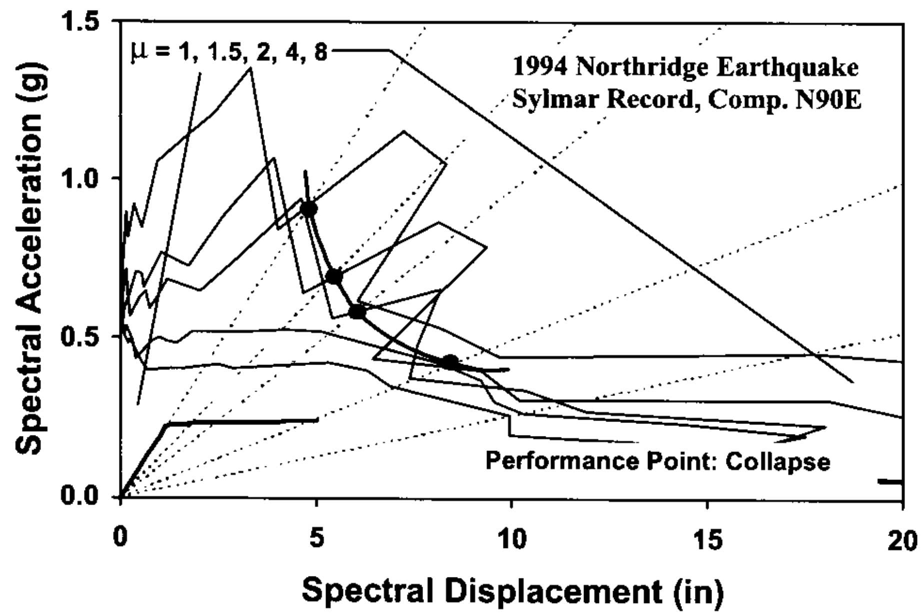


Figure 30 Graphical Performance Evaluation of Bridge Subjected to Sylmar Record using Inelastic Demand Spectrum Analysis.

As an alternative to generating inelastic ADRS curves, the so-called *Inelastic Static Analysis* method may be employed in which only the yield acceleration response spectrum is utilized (Yu et al. 1999). In this case, an assessment of performance is made by using the yield acceleration response spectrum to determine the yield acceleration demand associated with the fundamental natural period and the ductility capacity of the bridge; the ductility capacity being taken from an idealized bilinear representation of the capacity curve. The maximum displacement demand is then obtained by relating the yield acceleration to the yield displacement. A comparison of the yield acceleration and displacement demands to the corresponding capacities from the capacity curve (at the point of maximum ductility) reveals whether the bridge will fail or not. For the bridge discussed in this study, this method indicated that the bridge would survive the El Centro record (yield acceleration demand = 0.18 g < yield acceleration capacity = 0.24 g;

displacement demand = 3.89 in. < displacement capacity = 5.00 in.) but fail under the Sylmar record (yield acceleration demand = 0.4 g > yield acceleration capacity = 0.24 g; displacement demand = 8.66 in. > displacement capacity = 5.00 in.). As can be seen by comparing the above results with those provided in Table 9, this method leads to the same conclusion regarding the survivability of the bridge as the Inelastic Demand Spectrum Analysis method. Note that, if the bridge is predicted to survive, this method provides no information on the performance point. An iterative analysis could be employed to identify the performance point by using the calculated displacement demand as the starting point in the next iteration. For the El Centro record, the iterative analysis identified the performance point as corresponding to an acceleration and displacement of 0.22g and 3.41 in., respectively. Recall that the performance point is associated with a SDOF fundamental mode response of the bridge. The associated MDOF response, obtained by inverting Equations (1) and (2) and giving due consideration to the initial dead load displacement of negative 0.27 in., is: Base Shear = 1003.4 kips and Control Node Displacement = 2.85 in. These results are approximately 3% less and 27% larger, respectively, than the results obtained from the Inelastic Demand Spectrum Analysis (see Table 9).

DISCUSSION

The major points of this study are summarized and briefly discussed below.

Seismic Analysis and Software Survey: The seismic analysis and software survey indicated that, among practicing bridge engineers, SEISAB is most commonly used for AASHTO single mode methods while SAP2000 is most commonly used for elastic modal response spectrum/time-history analysis and inelastic static analyses. For capacity spectrum analysis and inelastic time-history analysis, WFRAME and ADINA, respectively, are most commonly used. The survey also showed that modal response spectrum analysis is the method that is most often used for seismic analysis while the capacity spectrum analysis method and the ATC-32 equivalent static analysis method have the least number of users. In general, the survey revealed that few practicing engineers are routinely performing nonlinear static analysis.

Distinction Between Force- and Displacement-Based Seismic Analysis Methods: Due to the confusion that often exists regarding the meaning of the terms force-based and displacement-based seismic analysis, a simple qualitative description of these terms was sought that would simultaneously distinguish one from the other. The descriptions are as follows: Force-based methods of seismic analysis can be defined as methods in which forces play a dominant role in the analysis and displacements are checked on a secondary level. Furthermore, force-based methods are methods in which the analysis is performed with the stiffness of the structure remaining equal to the elastic stiffness and the evaluation criteria are primarily in terms of strength. Displacement-based methods of seismic analysis can be defined as methods in which displacements play a dominant role in the analysis and forces are checked on a secondary level. Furthermore, displacement-based methods are methods in which the analysis is performed with the stiffness of the structure being displacement-dependent and the evaluation criteria are primarily in terms of displacements.

Influence of Support Conditions: As is evident from the eigenvalue analysis and the force-based analysis performed herein, the dynamic behavior of a bridge can be significantly influenced by the support conditions. For the bridge analyzed in this study, the influence of support conditions was particularly strong for seismic excitation in the longitudinal direction where the different abutment types resulted in significant differences in lateral resistance. As explained below, the distinct difference in the force-based response of the bridge with the Basic Support Model (fixed column footings and seat-type abutments) and the Spring Support Model (flexible column footings and stub wall abutments) resulted in different conclusions regarding the need for performing displacement-based seismic analysis.

Force-Based Analysis as a Precursor to Displacement-Based Analysis: The results from force-based seismic analyses (linear dynamic analysis) can be used to prioritize cases (i.e., bridge configuration and/or seismic loading) under which nonlinear displacement-based analysis should be performed. In this study, the results from the force-based analysis suggested that the case under which failure was most probable was the bridge with Basic Support Conditions and subjected to the El Centro and Sylmar records. Thus, displacement-based seismic analysis was performed only for this case.

Consideration of Higher Modes for Displacement-Based Analysis: It is generally accepted that a sufficient number of modes have been considered for modal analysis when at least 90% of the mass is participating in those modes or when the fundamental period of vibration is greater than about one second. For the displacement-based methods of analysis considered herein, the MDOF bridge model (Basic Support Conditions) was converted to a SDOF model associated with the fundamental mode of vibration in the longitudinal direction. The modal participating mass ratio and natural period for this mode was 91.9% and 0.72 sec, respectively, and thus higher mode effects were not considered.

Methods of Performing Displacement-Based Seismic Analysis: Two methods for performing displacement-based seismic analysis are emphasized herein; the Capacity Spectrum Analysis and Inelastic Demand Spectrum Analysis. The Capacity Spectrum Analysis method is currently being utilized to a limited degree within the practicing earthquake engineering community while the Inelastic Demand Spectrum Analysis method has been primarily limited to use within research studies.

Generation of Pushover and Capacity Curve: This study includes a detailed explanation of the procedure for performing pushover analysis to generate a pushover curve along with an explanation of how to convert the pushover curve in the MDOF domain to the capacity curve in the SDOF domain. In addition, the features of each software program that are unique to generating a pushover and capacity curve are described.

Pushover Analysis Using SC-Push3D: SC-Push3D, being a DOS-based program, does not permit the user to review the computer model graphically. Yielding is assumed to take place in concentrated plastic hinges located at the ends of the each element and thus refinement of the mesh representing the model is dependent on the user's judgment. The user defines the yield surface and the moment-rotation behavior of the plastic hinge. Since the program will not automatically stop the pushover analysis, the user needs to monitor the curvature of each plastic hinge to check for local instability and the distribution of plastic hinges to check for global instability. The pushover curve can not be graphically displayed by SC-Push3D.

Pushover Analysis Using SAP2000: SAP2000 has the ability to assign concentrated plastic hinges at any location along an element (resulting in the possibility of needing fewer elements than are needed in SC-Push3D) and has the ability to assign more than one plastic hinge type to an element (which is not possible within SC-Push3D). In addition, P-Delta effects can be considered while performing pushover analysis which is not possible with either SC-Push3D or GT-STRUDL. SAP2000 automatically stops when a plastic hinge reaches its curvature capacity, requiring the user to check if a sufficient number of plastic hinges have developed to render the structure globally unstable. Finally, the pushover curve can be graphically displayed along with the sequence of hinge formation.

Pushover Analysis Using GT-STRUDL: The program GT-STRUDL utilizes a distributed plasticity model for the plastic hinges. In contrast to a concentrated plasticity model, a distributed plasticity model can provide deeper insight into the behavior of the plastic hinge. The user can define the confinement of the cross section which results in the ability to more accurately capture the "exact" behavior of the hinge. GT-STRUDL automatically stops when it detects global instability of the structure due to the development of sufficient plastic hinges. One disadvantage of GT-STRUDL is found in the *list pushover analysis ductility ratio* command whose results are strongly sensitive to the fiber grid geometry defined by the user. The pushover curve can not be graphically displayed by GT-STRUDL.

Pushover Analysis Using ADINA: While ADINA is a powerful program for performing static and dynamic analysis of structural systems, it does not have a specific utility for performing pushover analysis and thus is not regarded as very useful for such an analysis. Thus, in spite of having been selected for evaluation, ADINA was not utilized for pushover analysis in this study.

Capacity Curve Obtained from Software: Among the four programs evaluated, only SAP2000 was capable of generating the capacity curve and providing the curve in graphical form. Interestingly, for the bridge studied herein, the capacity curve generated by SAP2000 did not appear to correctly account for the initial negative displacement produced by the gravity loads. Further investigation of this issue is warranted.

Generation of Demand Spectrums for Displacement-Based Analysis: In the Capacity Spectrum Analysis method, the demand is represented by elastic response spectra for a range of viscous damping ratios while, for the Inelastic Demand Spectrum method, the demand is represented by inelastic response spectra for a range of ductility levels. In the case of *design* demand spectra, the effort required to generate the elastic and inelastic spectra is essentially the same. However, in the case of demand spectra for specific earthquake records, the effort required to generate inelastic demand spectra is quite significant (in terms of human effort and computation time) and is regarded as a significant drawback to the Inelastic Demand Spectrum method.

Evaluation of Performance Using Displacement-Based Analysis: Both the Capacity Spectrum Analysis method and the Inelastic Demand Spectrum analysis method were used to evaluate the performance of the bridge. The evaluatio n involved overlaying the capacity curve with the demand spectrums to identify the performance point (spectral acceleration and displacement). The performance point was then converted to the base shear and control node displacement for the MDOF bridge structure. The results from the two methods were different, with the Inelastic Demand Spectrum Analysis predicting larger displacement demands on the bridge. Note that the neither method of analysis can be regarded as more accurate than the other since a number of important issues are not properly considered in displacement-based nonlinear static analysis (e.g., duration and cyclic nature of loading). The major appeal of displacement-based methods of analysis is their emphasis on displacements (i.e., damage-related quantities) and the graphical nature of some of the methods. The loss of accuracy is accepted since the methods are regarded as providing reasonable estimates of the inelastic demands on the structure.

Software Evaluation: Based on a thorough evaluation of the selected software programs and experience in utilizing the programs for both force-based and displacement-based seismic analysis, the qualitative software evaluation matrix shown in Table 10 was developed. Based on the discussion provided above, the results shown in the evaluation matrix, and the personal opinions of the authors, SAP2000 is recommended for performing practical displacement-based seismic analysis of simple highway bridge structures.

Table 10 Software Evaluation Matrix.

Evaluation Criteria	SC-Push3D	SAP2000	GT-STRUDL	ADINA
DOS platform	✓			
Windows platform		✓	✓	✓
Equivalent static analysis		✓	✓	
Modal response spectrum analysis		✓	✓	✓
Elastic time-history analysis		✓	✓	✓
Pushover analysis utility	✓	✓	✓	
Capacity spectrum analysis		✓		
Inelastic demand spectrum analysis				
Nonlinear time-history analysis			✓	✓
Simple graphical input		✓	✓	✓
Simple graphical output		✓	✓	✓
Foundation flexibility modeling	✓	✓	✓	✓
Circular and Rectangular columns	✓	✓	✓	✓
Bi-axial bending of circular and Rectangular columns	✓	✓	✓	✓
Bi-axial yield surface interpolated		✓		✓
Elements specific to bridge structures		✓		✓
Concentrated plasticity	✓	✓		✓
Distributed plasticity			✓	✓
P-delta effects with pushover analysis		✓		✓

CONCLUSIONS

The following conclusions may be drawn from this study:

- The majority of practicing bridge engineers are not currently utilizing displacement-based analysis methods for assessing the seismic performance of bridges. Of those who are, the software programs WFRAME and SAP2000 are being utilized.
- The need for performing nonlinear displacement-based seismic analysis is influenced by both the bridge configuration and seismic loading. Linear force-based analysis methods may be conveniently used to prioritize cases under which nonlinear displacement-based analysis should be conducted.
- Pushover analysis can be performed using a variety of software programs, some programs being particularly well-suited to such analysis. However, of those programs evaluated herein, there was only one program (SAP2000) that completes the displacement-based seismic analysis by generating the capacity curve and demand curves, identifying the SDOF performance point, and converting back to the MDOF domain. Furthermore, SAP2000 is largely graphical in nature, allowing for straightforward data input and interpretation of results. Thus, SAP2000 is regarded as the most effective software for performing practical displacement-based seismic analysis.
- For detailed evaluation of plastic hinge behavior, it is necessary to use software that incorporates distributed plasticity models for the plastic hinges (e.g., GT-STRUDL).
- The SC-Push3D software, while specifically developed for nonlinear pseudo-

static pushover analysis, was determined to be impractical due to its non-graphical DOS-based operating environment which results in difficulty in identifying errors in the computer model and difficulty in interpreting the output. In addition, the ADINA software is regarded as impractical since it does not have a specific utility for performing pushover analysis.

- Regarding the displacement-based methods of seismic analysis used in this study, the Capacity Spectrum Analysis method is well-known and the seismic demand (represented by elastic response spectra) can be easily determined. For the Inelastic Demand Spectrum method, the seismic demand is much more difficult to define unless a simplified inelastic design spectrum is utilized.
- The Capacity Spectrum Analysis method and the Inelastic Demand Spectrum Analysis method produce different structural demands. Neither method is regarded as producing accurate results due to a number of simplifications inherent in the methods. However, the methods appear to be attractive to practicing engineers due to their emphasis on a graphical evaluation of seismic performance.

RECOMMENDATIONS

Based on the results and conclusions of this study, the following recommendations are made:

- Elastic force-based seismic analysis may be employed to identify the need for performing inelastic displacement-based seismic analysis.
- Due to its emphasis on graphical input/output and its comprehensive tools for displacement-based analysis, the SAP2000 software program is recommended for

efficient and practical displacement-based seismic analysis of simple highway bridges.

- Due to its use of a distributed plasticity plastic hinge model, the GT-STRUDL software program is recommended for displacement-based seismic analysis when a more detailed evaluation of plastic hinge behavior is required.
- For future research, the approach to converting the pushover curve to the capacity curve when gravity loads produce initial displacements needs to be investigated. In addition, the theoretical validity of both the Capacity Spectrum Analysis method and the Inelastic Demand Spectrum method needs to be investigated.
- Current national research on displacement-based seismic analysis (e.g., the ATC-55 Project entitled "Evaluation and Improvement of Inelastic Seismic Analysis Procedures") should be monitored to keep abreast of the most recent developments on this evolving topic.

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APPENDIX A

SOFTWARE SURVEY

The following two pages show the questionnaire that was sent to State Departments of Transportation and consulting engineering firms.

Survey on Seismic Analysis Methods and Associated Software for Highway Bridges

ESTIMATED TIME TO COMPLETE SURVEY: 5 MINUTES

Company / DOT Name:____

Name of engineer who filled out survey: _____________________________

Email address: ____

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1. Do you use the AASHTO single mode methods? ❑ Yes ❑ No

If yes, what software do you use for performing this type of analysis?

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2. Do you use the ATC-32 equivalent static method? ❑ Yes ❑ No

If yes, what software do you use for performing this type of analysis?

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3. Do you perform modal response spectrum analysis? ❑ Yes ❑ No

If yes, what software do you use for performing this type of analysis?

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4. Do you perform elastic time history analysis? ❑ Yes ❑ No

If yes, what software do you use for performing this type of analysis?

5. Do you perform inelastic static analysis (i.e., pushover analysis for capacity and inelastic response spectrum for demand)? ☐ Yes ☐ No

If yes, what software do you use for performing this type of analysis?

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6. Do you perform capacity spectrum analysis (i.e., pushover analysis for capacity and modified elastic response spectrum for demand)? ☐ Yes ☐ No

If yes, what software do you use for performing this type of analysis?

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7. Do you perform inelastic time history analysis? ☐ Yes ☐ No

If yes, what software do you use for performing this type of analysis?

Additional Comments

Table A-1 No. of State DOT's and consulting firms contacted and replied to survey

No. of State DOT's and consulting firms contacted	53
No. of State DOT's and consulting firms replied	36

Table A-2 Distribution of contacts and replies to the survey

No. of State DOT's contacted	22
No. of State DOT's replied	19
No. of consulting firms contacted	31
No. of consulting firms replied	17

Table A-3 Response to question No. 1 "Do you use the AASHTO single mode methods? If yes what software do you use for performing this type of analysis?"

State DOT's replied YES	10
State DOT's replied NO	9
Consulting firms replied YES	4
Consulting firms replied NO	13

Table A-4 Software used for performing AASHTO single mode methods versus number of users

Software	No. of Users
SEISAB	6
Hand Calculations	2
M-STRUDL	1
STAAD3	1
Visual Analysis	1
SEISAB and SAP2000	1
GT-STRUDL and STAAD3	1
Hand Calculations and RISA3D	1

Table A-5 Response to question No. 2 "Do you use the ATC-32 equivalent static method? If yes what software do you use for performing this type of analysis?"

State DOT's replied YES	4
State DOT's replied NO	15
Consulting firms replied YES	4
Consulting firms replied NO	13

Table A-6 Software used for performing ATC-32 equivalent static method versus number of users

Software	No. of Users
Hand Calculations	3
SEISAB	1
M-STRUDL	1
STAAD3	1
SAP2000	1
Hand Calculations or SAP2000	1

Table A-7 Response to question No. 3 "Do you perform modal response spectrum analysis? If yes what software do you use for performing this type of analysis?"

State DOT's replied YES	16
State DOT's replied NO	3
Consulting firms replied YES	14
Consulting firms replied NO	3

Table A-8 Software used for performing modal response spectrum analysis versus number of users

Software	No. of Users
SAP2000 and SEISAB	9
SEISAB	7
SAP2000	4
STAAD3	2
GT-STRUDL	2
STAAD3 and GT-STRUDL	1
GT-STRUDL and STAAD3	1
M-STRUDL and GT-STRUDL	1
LARSA and ALGOR	1
ADINA and RM-Space Frame	1
SEISAB and LARSA	1

Table A-9 Response to question No. 4 "Do you perform elastic time history analysis? If yes what software do you use for performing this type of analysis?"

State DOT's replied YES	5
State DOT's replied NO	14
Consulting firms replied YES	6
Consulting firms replied NO	11

Table A-10 Software used for performing elastic time history analysis versus number of users

Software	No. of Users
SAP 2000	4
ADINA	3
ANSYS	1
DRAIN 2DX	1
NASTRAN	1
ADINA and RM-Space Frame	1

Table A-11 Response to question No. 5 "Do you perform inelastic static analysis? If yes what software do you use for performing this type of analysis?"

State DOT's replied YES	4
State DOT's replied NO	15
Consulting firms replied YES	7
Consulting firms replied NO	10

Table A-12 Software used for performing inelastic static analysis versus number of users

Software	No. of Users
SAP 2000	4
SC-Push3D	1
NASTRAN	1
ANSYS	1
Frame 407	1
ANACAP-U and ABAQUS	1
WFRAME and X-SECTION	1
M-STRUDL and GT-STRUDL	1
In-House developed software	1

Table A-13 Response to question No. 6 "Do you perform capacity spectrum analysis? If yes what software do you use for performing this type of analysis?"

State DOT's replied YES	2
State DOT's replied NO	17
Consulting firms replied YES	6
Consulting firms replied NO	11

Table A-14 Software used for performing capacity spectrum analysis versus number of users

Software	No. of Users
SAP2000 and WFRAME	2
SAP2000, WFRAME, COLX, and BeamX	1
In-House developed software	1
XSECTION, WFRAME, and ADINA	1
XSECTION, WFRAME, LARSA, and BIAX	1
SAP2000 and DRAIN	1
WFRAME	1
GT-STRUDL, WFRAME, and SEISAB	1

Table A-15 Response to question No. 7 "Do you perform inelastic time history analysis? If yes what software do you use for performing this type of analysis?"

State DOT's replied YES	4
State DOT's replied NO	15
Consulting firms replied YES	7
Consulting firms replied NO	10

Table A-16 Software used for performing capacity spectrum analysis versus number of users

Software	No. of Users
ADINA	5
SAP2000	2
SADSAP	1
NASTRAN	1
DRAIN2D	1
ADINA, ANCAP, and ABAQUS	1