MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

A National Center of Excellence in Advanced Technology Applications

ISSN 1520-295X

Evaluation of Bridge Damage Data from the Loma Prieta and Northridge, California Earthquakes

by

Nesrin Basöz and Anne S. Kiremidjian
The John A. Blume Earthquake Engineering Center
Department of Civil Engineering
Stanford University
Stanford, California 94305-4020

Technical Report MCEER-98-0004

June 2, 1998

This research was conducted at Stanford University and was supported by the Federal Highway Administration under contract number DTFH61-92-C-00106.

MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

A National Center of Excellence in Advanced Technology Applications

Evaluation of Bridge Damage Data from the Loma Prieta and Northridge, California Earthquakes

by

N. Basöz[1] and A.S. Kiremidjian[2]

Publication Date: June 2, 1998
Submittal Date: December 15, 1997

Technical Report MCEER-98-0004

Task Number 106-E-7.3.3

FHWA Contract Number DTFH61-92-C-00106

1 Senior Staff Engineer, K2 Technologies, Inc.; former Post-doctoral Researcher, The John A. Blume Earthquake Engineering Center, Department of Civil Engineering, Stanford University
2 Professor and Director, The John A. Blume Earthquake Engineering Center, Department of Civil Engineering, Stanford University

MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
State University of New York at Buffalo
Red Jacket Quadrangle, Buffalo, NY 14261

Preface

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the State University of New York at Buffalo, the Center was originally established by the National Science Foundation in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

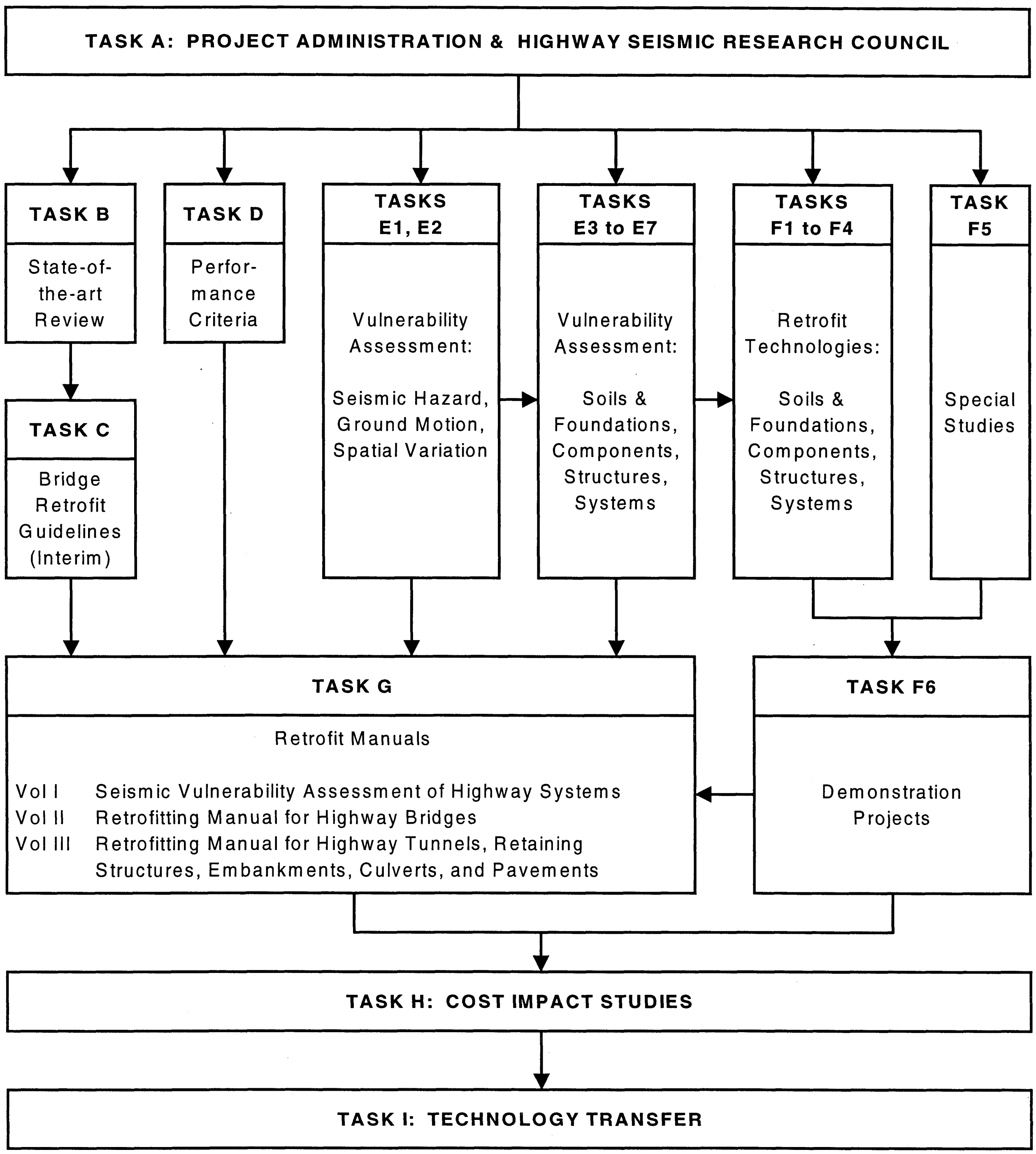
MCEER's research is conducted under the sponsorship of two major federal agencies, the National Science Foundation (NSF) and the Federal Highway Administration (FHWA), and the State of New York. Significant support is also derived from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

The Center's FHWA-sponsored Highway Project develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and improved seismic design criteria and procedures for bridges and other highway structures. Specifically, tasks are being conducted to:
- assess the vulnerability of highway systems, structures and components;
- develop concepts for retrofitting vulnerable highway structures and components;
- develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, which include consideration of soil-structure interaction mechanisms and their influence on structural response;
- review and recommend improved seismic design and performance criteria for new highway structures.

Highway Project research focuses on two distinct areas: the development of improved design criteria and philosophies for new or future highway construction, and the development of improved analysis and retrofitting methodologies for existing highway systems and structures. The research discussed in this report is a result of work conducted under the existing highway structures project, and was performed within Task 106-E-7.3.3, "Evaluation of Bridge Damage Data from Recent Earthquakes" of that project as shown in the flowchart on the following page.

The overall objective of this task was to correlate observed bridge damage resulting from the 1989 Loma Prieta and 1994 Northridge earthquakes to the local ground motions, bridge structural characteristics, and repair costs and time. Damage states reported after the earthquakes were investigated and new damage state definitions for concrete bridges were proposed. Bridges were grouped by their structural characteristics and correlation studies were performed to obtain ground motion-damage relationships and ground motion-repair cost ratio relationships. Logistic *regression analysis was used to obtain empirical fragility curves. Currently available fragility curves and damage probability matrices were compared to observed damage data and the empirical relationships developed in this study.*

SEISMIC VULNERABILITY OF EXISTING HIGHWAY CONSTRUCTION
FHWA Contract DTFH61-92-C-00106



ABSTRACT

This report presents the significant findings from a study of damage to bridges during the 1989 Loma Prieta and the 1994 Northridge, CA earthquakes. In both earthquakes, less than five percent of the bridges that were exposed to ground shaking were damaged. As experienced in the past earthquakes, bridges with non-monolithic abutment types, discontinuous spans and single column bents performed poorly. High skew contributed to high damage levels. Performance of bridges designed and built before 1971 was poorer than those designed according to more recent standards. The total estimated repair cost of $150 million for the bridges damaged in the Northridge earthquake was about two thirds of the repair cost estimated from the Loma Prieta earthquake. Column damage was the most damaged component in both earthquakes.

Data on bridge damage were compiled, reviewed and analyzed to correlate observed bridge damage to structural characteristics of a bridge, ground motion levels and estimated repair costs. Damage states reported after the earthquakes were investigated and new damage state definitions for concrete bridges were proposed. Bridges were grouped by their structural characteristics and correlation studies were performed to obtain ground motion-damage relationships and ground motion-repair cost ratio relationships. Logistic regression analysis was used to obtain empirical fragility curves. Currently available fragility curves and damage probability matrices were compared to observed damage data and the empirical relationships developed in this study.

ACKNOWLEDGMENTS

The authors would like to thank Caltrans bridge engineers from the Structure Maintenance and Investigation Division for their help in providing the bridge damage and inventory data. In particular, the assistance of James Hare, Mike Johnson, Evan Evans and Tom Sandoval are greatfully acknowledged. Initial discussions with Dr. Ian G. Buckle helped in defining the scope of this study and we appreciate his interest and support. This study (Task No. 106-7.3.3) was sponsored by the National Center for Earthquake Engineering Research under the Highway Project entitled "Seismic Vulnerability of Highway Construction", with funding from the Federal Highway Administration (FHWA).

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd)

LIST OF ILLUSTRATIONS

LIST OF ILLUSTRATIONS (Cont'd)

LIST OF ILLUSTRATIONS (Cont'd)

LIST OF ILLUSTRATIONS (Cont'd)

LIST OF ILLUSTRATIONS (Cont'd)

LIST OF ILLUSTRATIONS (Cont'd)

LIST OF ILLUSTRATIONS (Cont'd)

LIST OF ILLUSTRATIONS (Cont'd)

LIST OF ILLUSTRATIONS (Cont'd)

LIST OF ILLUSTRATIONS (Cont'd)

LIST OF TABLES

LIST OF TABLES (Cont'd)

SECTION 1

INTRODUCTION

1.1 MOTIVATION

Damage to highway systems from recent earthquakes has emphasized the need for risk assessment of the existing highway transportation systems. Risk assessment efforts were initiated in California by the early seventies and within the last few years have been followed by efforts in other states. The 1971 San Fernando earthquake caused substantial damage to then recent bridge construction and exposed a number of deficiencies in bridge design specifications in effect at that time. Following the earthquake, bridge design specifications in California were modified. During the 1987 Whittier Narrows, and the 1989 Loma Prieta earthquakes, bridges designed to pre-1971 force levels specified by Caltrans or AASHTO were damaged extensively. In contrast, the majority of bridges performed well in the most recent 1994 Northridge earthquake demonstrating the benefits of improved seismic bridge design and retrofitting schemes in the United States within the previous two decades. Bridge damage data from these earthquakes, however, have not been systematically studied with the objective to evaluate damage characteristics and to correlate these to observed or estimated ground motion levels.

This report presents the significant findings from a study of damage to bridges during the 1989 Loma Prieta and the 1994 Northridge, CA earthquakes. These two earthquakes caused the most significant bridge damage since the San Fernando earthquake. This study (Task No. 106-7.3.3) was part of the seismic research project conducted by the National Center on Earthquake Engineering Research (NCEER) entitled "Seismic Vulnerability of Existing Highway Construction" with funding by the Federal Highway Administration. The project aims "to produce state-of-the-art guidance on the seismic assessment, screening, evaluation and retrofitting of not only the structural components of highway systems (bridges, tunnels, retaining walls and alike) but also the system itself as a network of transportation corridors" [Buckle and Friedland, 1996].

The performance of the system is highly dependent on the performance of individual components as well as the connectivity of these components. Assessment of potential damage to bridges is thus essential to estimate the performance of the transportation system. The vulnerability assessment of bridges is useful for seismic retrofitting decisions, disaster response planning, estimation of direct monetary loss, and evaluation of loss of functionality of the highway system due to damage to bridges from earthquakes. During the post-earthquake recovery period, the structural vulnerability assessment can assist in decisions, such as whether a bridge should be open to traffic immediately after an earthquake, it should be demolished or repaired, and if there are several bridges to be repaired in what order should they be repaired or replaced. A detailed methodology for risk assessment of highway transportation systems is presented in Basöz and Kiremidjian [1996]. A similar study is under progress as part of the NCEER seismic research project [Werner et al., 1996]. A key component of the methodologies in these studies is the quantification of bridge vulnerability, i.e., the relationship between bridge damage and ground motion as well as the relationship between bridge damage and repair cost/time, therefore the bridge functionality.

1.2 Objective and Scope

The objectives of this study were:

- to compile, review and analyze bridge damage data from the 1989 Loma Prieta and 1994 Northridge earthquakes, and
- to correlate observed bridge damage to its structural characteristics as well as the ground motion and repair cost.

A major effort was undertaken in the compilation of a comprehensive database on bridge damage and repair cost for the two most recent California earthquakes that struck an urban area. This database can be used for comparative studies on bridge damage and to identify the needs for better data collection and post-earthquake investigation.

In both the Loma Prieta and the Northridge earthquakes, local bridges were reported to have sustained much less damage than the state bridges. Therefore, damage only to state bridges was analyzed in this study. Statistics were obtained for the bridges compiled in this database. Correlation studies were performed to obtain ground motion-damage relationships and ground motion-repair cost ratio relationships.

A geographic information system (GIS) was used to obtain the ground shaking levels at each bridge site. A relational database management system (RDBMS) was used to compile the database on bridge damage and structural characteristics of bridges.

The results of this study are specific to bridges designed and constructed according to California standards. These results, however, can be utilized to develop an improved understanding of the seismic vulnerability of bridges in other parts of the country.

1.3 ORGANIZATION OF THE REPORT

Section 2 of this report outlines the tasks of the study and describes the approach used to analyze data on bridge damage. This section includes the structure of the database and its attributes, and the descriptions for different bridge classifications used in the correlation studies. A review of damage state definitions used in post-earthquake investigations is presented. The methods used to obtain relationships between ground motion and bridge damage and repair cost are explained in Section 2.3.

The application of the methods described in Section 2 to the 1989 Loma Prieta and the 1994 Northridge earthquakes and the results are presented respectively in Sections 3 and 4. A discussion of the results is provided at the end of each section. The results from the two earthquakes are compared in Section 5. Lastly, major findings of this study are presented in Section 6.

SECTION 2

ANALYSIS METHOD FOR DAMAGE DATA

In order to achieve the objectives stated in Section 1, the following tasks were defined:

(1) *Data compilation:*

(1.a) Compile a database on structural characteristics of damaged and undamaged bridges in the San Francisco Bay area and the Greater Los Angeles area,

(1.b) Collect bridge damage data observed in the 1989 Loma Prieta and the 1994 Northridge earthquakes,

(1.c) Collect data on repair cost from the two earthquakes,

(1.d) Obtain ground motion levels at bridge sites either from recordings or from seismic hazard analysis conducted for the areas affected by the Loma Prieta and Northridge earthquakes.

(2) *Classification of bridges according to structural characteristics and damage states:*

(2.a) Review bridge inventory data,

(2.b) Classify bridges by structural characteristics,

(2.c) Review definitions for damage states,

(2.d) Analyze bridge damage and repair cost data.

(3) *Correlation studies:* Develop ground motion-damage and ground motion-repair cost ratio relationships based on the data from the Loma Prieta and the Northridge earthquakes.

The methods for compiling and analyzing the data are outlined in this section. The results for the Loma Prieta and the Northridge earthquakes are presented in Sections 3 and 4, respectively.

2.1 CHARACTERISTICS OF THE BRIDGE DATABASE

A database was compiled for each of the events, the 1989 Loma Prieta and the 1994 Northridge earthquakes. Each database includes five types of data: (*i*) structural characteristics, (*ii*) bridge damage, (*iii*) repair cost, (*iv*) ground motion levels at bridge sites, and (*v*) soil characteristics at bridge sites. A commercial relational database management system (RDBMS), dBase™, was used to compile the data and to perform queries on bridge damage, ground motion levels, structural characteristics and repair cost. In addition, a commercial geographic information system (GIS), Arc/Info™, was used to perform spatial queries on the ground motion levels observed in the two earthquakes and soil types.

Structural Characteristics: Several structural characteristics were compiled in a database for the bridges that were exposed to ground shaking in each of the two earthquakes. These structural characteristics include abutment type, number of spans, type of superstructure and substructure, length and width of the bridge, skew, number of hinges at joints and bents, abutment and column foundation types, and design year. These structural attributes were obtained from the Structural Maintenance System (SMS) database compiled and managed by Caltrans. The detailed descriptions for each of these attributes can be found in the OSM&I Guide [Caltrans, 1993].

Caltrans is currently in the process of compiling a database that includes information on abutment details (e.g., seat width, type and height of bearings), and column and footing details (e.g., minimum/maximum column height, footing type and column/footing connection). However, only about 15 percent of all the California bridges, 25 percent of the bridges damaged in the Northridge earthquake, and about 20 percent of the bridges damaged in the Loma Prieta earthquake are currently in this database.

Bridge Damage: Detailed damage descriptions and the corresponding damage states were compiled for the bridges damaged in the Loma Prieta and the Northridge earthquakes. The damage descriptions were obtained mainly from the bridge damage reports compiled by Caltrans [1989; 1994]. For the Northridge earthquake, these descriptions were cross-referenced with those provided by Buckle [1994], EERI [1995], and Yashinsky [1995]. Judgment was used to treat inconsistencies in the interpretation of the observed damage data.

The bridge damage data were used in correlation studies to obtain ground motion-damage relationships as described in Section 2.3. The database on bridge damage includes only *minor* and *major* damage states for bridges damaged in the Loma Prieta earthquake, and *minor*, *moderate*, *major* and *collapse* damage states for those damaged in the Northridge earthquake. These damage states were used in the correlation studies as well.

Damage State Definitions: Currently, no guidelines exist for evaluating physical bridge damage. The terms *minor*, *moderate* and *major* used to describe the severity of damage are subjective. Definitions of damage states for columns, abutments, joints and connections of concrete bridges were proposed by Basöz and Kiremidjian [1996] based on the observed bridge damage in the Northridge earthquake. Under this project a questionnaire was formulated to acquire expert opinion on the proposed damage state definitions and was given to bridge engineers at Caltrans. The questionnaire together with the responses from several bridge engineers is given in Appendix A. The results showed considerable agreement among the practicing engineers. Thus, the survey results can be used for formulating standard definitions for damage states for bridges.

Repair Cost: The estimated repair costs for bridges damaged in the Northridge and Loma Prieta earthquakes were compiled in a database. These estimated repair costs were obtained from supplementary bridge reports compiled by Caltrans following each earthquake. The database includes total estimated repair cost for a bridge, repair cost ratio, and a more detailed information on repair work and cost for each bridge that was repaired. The repair cost ratio was defined as the ratio of repair cost to replacement cost of a bridge. The replacement cost of a bridge was estimated to be \$90/ft^2 based on the 1995 cost books [Caltrans, 1994b].

Ground Motion Levels: In addition to structural characteristics, soil type and peak ground acceleration at each bridge site were compiled. For the Northridge earthquake, available ground shaking maps were used to obtain the peak ground acceleration (PGA) at each bridge site. Unlike the Northridge earthquake, however, ground shaking maps based on recordings from the Loma Prieta earthquake were not available. The PGA levels at bridge sites were obtained by simulating the Loma Prieta earthquake within GIS.

2.2 Classification of Bridges

The compiled inventory of bridges was reviewed to: (*i*) select bridges to be used in correlation studies, (*ii*) identify structural characteristics (attributes) that best describe the seismic response of bridges, and (*iii*) verify the correctness of the attribute values included in the bridge inventory database.

Data Sets: Several data sets were used for statistical analyses. All the analyses were performed for state bridges since most of the reported damage in both earthquakes pertained to state bridges. First, all state highway bridges were selected and gathered in the *highway bridge data set.* Statistics on design year and ground shaking levels were obtained for this data set. Most of the bridge damage pertained to concrete structures in both earthquakes. This might be due to smaller number of steel bridges in both the Greater Los Angeles area and the San Francisco Bay area. The number of steel bridges that were damaged in either of the two earthquakes was not large enough for detailed statistical analysis. Therefore, the correlation studies were mainly conducted for concrete highway bridges.

One of the objectives of the study was to identify the effect of various structural characteristics on bridge damage susceptibility. Two structural characteristics were considered: abutment type and column bent type. In order to determine the effect of each attribute, only bridges with homogeneous structural characteristics were selected from the *concrete highway bridge data set.* For the purposes of this study, bridges with unique abutment type and column bent type were defined as *homogeneous.* For example, a bridge with a seat type (non-monolithic) and a diaphragm type (monolithic) abutments, was defined as a *heterogeneous* bridge and was excluded from the *homogeneous data set.* Similarly, a bridge with both multiple and single column bents was defined as a *heterogeneous* bridge, and was excluded from the *homogeneous data set.* Bridges with incomplete information were also excluded from this data set. Bridges with *heterogeneous* characteristics were not analyzed separately in this study. However, several statistics were obtained for the *concrete highway bridge data set*, which included both *homogeneous* and *heterogeneous* bridges. Similarly, statistics were obtained for all damaged bridges, including steel bridges.

The available ground shaking maps for the Loma Prieta and the Northridge earthquakes are limited to certain geographic areas and do not cover the locations of all bridges that were exposed to lower ground shaking levels. A complete data set for correlation analyses requires that all the bridges exposed to a given ground shaking level be included in the data set. In order to satisfy this requirement, a minimum PGA level was selected as a threshold value. This PGA level was determined based on the available ground motion maps. Bridges that were exposed to the selected PGA level or higher were extracted from the *homogeneous data set* and the new data set was referred to as the *correlation data set.*

The bridges in the *correlation data set* were grouped first by the superstructure type and substructure material. Then, these bridges were further classified into sub-categories based on other structural characteristics, such as number of spans, abutment type, column bent type and span continuity. Bridges within the same sub-category are expected to experience similar damage under a given seismic loading. Table 2-1 lists the 21 bridge sub-categories (3 for single span bridges and 18 for multiple span bridges) used in the correlation analyses. In table 2-1, "continuous" refer to bridges with no joints at hinges or bents. The list of abutment types grouped as *monolithic*, *non-monolithic* and *partial* are listed in table 2-2. Figures 2-1 and 2-2 illustrate different column bent types and abutment types.

The bridge sub-categories were based on the bridge classification given by Basöz and Kiremidjian [1996]. Four of the sub-categories, C1S1, C1S2, C1M1 and C1M10[1], correspond to the sub-categories of a generic bridge class defined in that classification. These four sub-categories are expected to represent the least and the most vulnerable single and multiple span bridge groups, respectively. Accordingly, the fragility curves for the least and most vulnerable curves are expected to provide lower and upper boundaries for bridges in a given class. The fragility curve of any particular bridge can then be obtained by modifying these boundaries according to the structural characteristics of that bridge.

---

[1] C1S1: single span bridges with monolithic type abutments.

C1S2: single span bridges with non-monolithic type abutments.

C1M1: multiple span bridges with monolithic type abutments, continuous spans and multiple column bents.

C1M10: multiple span bridges with non-monolithic type abutments, discontinuous spans and single column bents.

TABLE 2-1 Description of Bridge Sub-categories (based on Basöz and Kiremidjian, [1996])

Bridge Sub-category	Abutment Type	Column Bent Type	Span Continuity
Single Span Bridges			
C1S1	Monolithic	Not applicable	Not applicable
C1S2	Non-monolithic	Not applicable	Not applicable
C1S3	Partial integrity	Not applicable	Not applicable
Multiple Span Bridges			
C1M1	Monolithic	Multiple	Continuous
C1M2	Monolithic	Multiple	Discontinuous
C1M3	Monolithic	Single	Continuous
C1M4	Monolithic	Single	Discontinuous
C1M5	Monolithic	Pier wall	Continuous
C1M6	Monolithic	Pier wall	Discontinuous
C1M7	Non-monolithic	Multiple	Continuous
C1M8	Non-monolithic	Multiple	Discontinuous
C1M9	Non-monolithic	Single	Continuous
C1M10	Non-monolithic	Single	Discontinuous
C1M11	Non-monolithic	Pier wall	Continuous
C1M12	Non-monolithic	Pier wall	Discontinuous
C1M13	Partial integrity	Multiple	Continuous
C1M14	Partial integrity	Multiple	Discontinuous
C1M15	Partial integrity	Single	Continuous
C1M16	Partial integrity	Single	Discontinuous
C1M17	Partial integrity	Pier wall	Continuous
C1M18	Partial integrity	Pier wall	Discontinuous

TABLE 2-2 Description of Abutment Types

Inventory Code	Description	Abutment Type
A	Diaphragm	*Monolithic*
E	Rigid Frame	*Monolithic*
B	Seat	*Non-monolithic*
C	Cantilever	*Non-monolithic*
D	Strutted	*Non-monolithic*
F	Bin	*Partial*
G	Cellular Closure	*Partial*

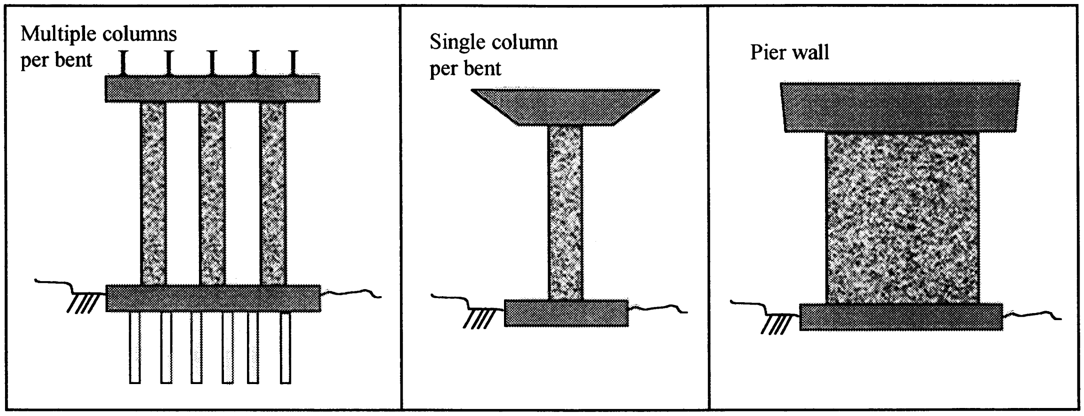


FIGURE 2-1 Different Column Bent Types Referred in Table 2-1

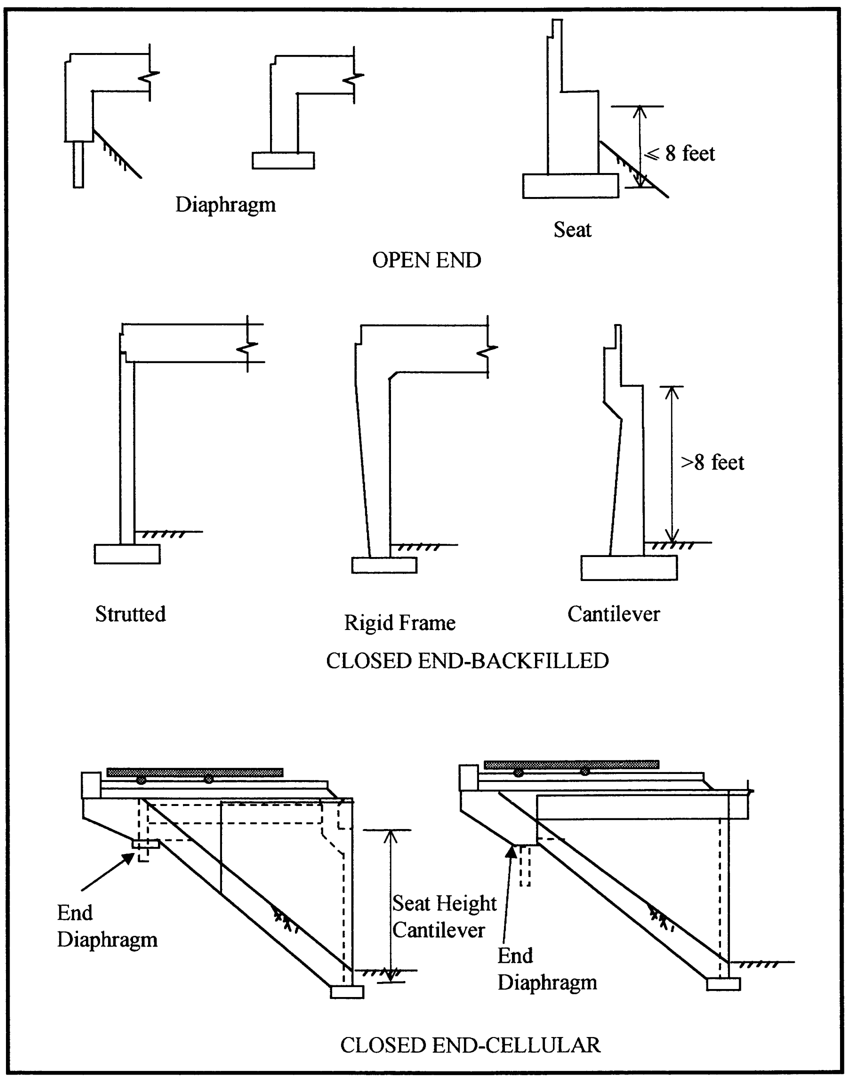


FIGURE 2-2 Different Abutment Types Referred in Table 2-2

Bridges were also grouped using the bridge classifications defined by HAZUS [1997] and ATC-13 [1985]. The HAZUS classification was used to compare the empirical fragility curves obtained in this study to those provided in HAZUS. The ATC classification was used to compare the repair cost ratios to damage probability matrices (DPMs) provided in ATC-13 [1985].

Reliability of the Database: Caltrans currently has two database systems: the first one, Structure Maintenance System (SMS), follows the Federal Highway Administration (FHWA) National Bridge Inventory System but is more detailed, and the second one, Bridge Inspection Records and Information Systems (BIRIS), is a database that stores the inspection records and the construction drawings for all the bridges. All the paper format information from each bridge inspection is scanned into BIRIS. For some of the 25,000 bridges in the state of California, discrepancies exist between the two databases.

The two databases were compared to verify the correctness of the attribute values for some of the bridges that were damaged in the Loma Prieta and the Northridge earthquakes. The abutment type and column bent type were found to be more likely to have errors than the other attributes that are of interest for this study. For example, only in few cases (for 2 to 3 percent of the damaged bridge data set) the values of the design year and skew attributes were found to be incorrect. Table 2-3 lists samples of potential inconsistencies for these attributes.

TABLE 2-3 Examples of Data Error in Bridge Inventory Databases

Bridge ID	**Attribute**	**BIRIS Code (Database 1)[2]**	**SMS Code (Database 2)[5]**
53 0645L	Abutment Type	C (cantilever)	N (pier wall)
53 0732K	Abutment Type	B (seat)	C (cantilever)
53 0739	Abutment Foundation Type	Both abutments are on S	One abutment is F, the other is S
53 1126	Abutment Type	A (diaphragm)	F (bin)
53 1133	Abutment Type	D (strutted)	C (cantilever)
53 1153	Abutment Type	closed end column abutments	B (seat)
53 0645L	Column Bent Type	RC pier	J (single column bent)
53 1985F	Column Bent Type	RC 2 column bents	J
53 2057	Column Bent Type	RC 3 column bents	J

[2] BIRIS and SMS are two separate databases obtained from Caltrans.

Where there were discrepancies between the two databases, the structural plans were investigated with the assistance of bridge engineers from Caltrans to determine the correct attribute values. In order to assess the effect of error in the databases on ground motion-damage relationships, correlation studies were performed using a data set with corrected and uncorrected attribute values. The correction of attribute values in the database is very time consuming and it is beyond the scope of this study to correct all the incorrect attribute values in Caltrans SMS database.

2.3 CORRELATION STUDIES

Correlation analyses were performed using the data from the Loma Prieta and the Northridge earthquakes to achieve the following objectives:

(*i*) To determine the structural characteristics which best reflect damage potential,

(*ii*) To obtain ground motion-damage relationships for bridges with similar structural characteristics,

(*iii*) To obtain ground motion-repair cost ratio relationships to estimate direct loss due to bridge damage,

(*iv*) To correlate damage and repair cost ratio.

2.3.1 Attributes Used in the Correlation Studies

As discussed in Section 2.2, bridges were grouped by their structural characteristics. For each of these classes, relationships between the following attributes and the damage state, and similarly between these attributes and repair cost ratio were obtained:

Peak ground acceleration: Observed and/or estimated PGA were used to characterize the ground shaking level at a bridge site. Spectral acceleration values at various periods were also available for the Northridge earthquake. However, a reliable estimate of the fundamental period of a bridge could not be determined from the available information. Therefore, PGA was used in the analyses.

Design year: The original bridge design year was used in this study. The design year was not updated based on the reconstruction or upgrading dates (either seismic or non-seismic).

Structural characteristics: The number of spans, abutment type, substructure type, span continuity, skew and span length were used to group bridges into various classes.

The data on bridge damage were compiled in the form of damage frequency matrices, i.e., the number of bridges with each level of observed damage at different PGA levels. Then, the damage probability matrices (DPMs), i.e., the probability of being in a damage state given the ground motion level, were obtained for each group of bridges. The damage matrices were used as input data to logistic regression analyses in order to obtain empirical fragility curves. Similar procedures were used to obtain empirical relationships between ground motion levels and repair cost ratios. The correlation analyses performed for each of the earthquakes are discussed in detail in Sections 3 and 4.

2.3.2 Logistic Regression Analysis

This sub-section provides a brief description of logistic regression analysis used in this study. Among other references, more detailed discussion on logistic regression analysis can be found in Hosmer and Lemeshow [1986], SAS Manual [1995], and Agresti [1996].

The goal of logistic regression analysis is to find the best fitting model to describe the relationship between an outcome and a set of independent variables. The fit of the model is then appraised, and the coefficients are evaluated to indicate the impact of the individual variables. Logistic regression model is used as a multivariate technique for estimating the probability that an event occurs. It is used for discrete outcome variables that take two or more possible values.

The relationship between the outcome and the independent variable is generally expressed in terms of the conditional mean, $E(Y|x)$. In general, the curve $E(Y|x)$ versus x generally resembles a plot of a cumulative distribution function of a random variable – in which case $E(Y|x)$ ranges between 0 and 1. The specific form of the logistic regression model used in the analysis is as follows:

$$\pi(x) = \frac{e^{\beta_0+\beta_1 x}}{1+e^{\beta_0+\beta_1 x}} \tag{2-1}$$

where $\pi(x) = E(Y|x)$ for logistic distribution. The *logit transformation* as described in equation (2-2) is used to attain many desirable properties of a linear regression model; e.g., the logit, $g(x)$ is linear in its parameters, may be continuous, and may range from $-\infty$ to $+\infty$, depending on the range of x.

$$\begin{aligned} g(x) &= \ln\left[\frac{\pi(x)}{1-\pi(x)}\right] \\ &= \beta_0 + \beta_1 x \end{aligned} \tag{2-2}$$

For a collection of k independent variables denoted by $\mathbf{x} = (x_1, x_2, \cdots, x_k)$, the logit of the logistic regression model takes the form of

$$g(\mathbf{x}) = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \cdots + \beta_k x_k \tag{2-3}$$

in which case

$$\pi(x) = \frac{e^{g(\mathbf{x})}}{1 + e^{g(\mathbf{x})}} \tag{2-4}$$

The parameters of the model are estimated using the maximum-likelihood method. The method of maximum likelihood yields values for the unknown parameters which maximize the likelihood of obtaining the observed set of data.

In this study, two types of models were used:

- a single variable model to obtain probabilities of exceedance at different levels of the discrete dependent variable, where the analysis based on the above equations was carried out.
- a linear logit model with multiple variables to identify the independent variables that have the most effect on the outcome, in which *stepwise logistic regression* algorithm was used. Stepwise logistic regression is a statistical algorithm which checks for the importance of variables [Hosmer and Lemeshow, 1987]. The importance of a variable is defined in terms of a measure of statistical significance of the coefficient for the variable. In logistic regression, the errors are assumed to follow a binomial distribution, and significance is assessed via the likelihood ratio chi-square test. Thus, at any step in the procedure the

most important variable, in statistical terms, is the one that produces the greatest change in the log-likelihood relative to a model not containing the variable. The p-values calculated in stepwise selection procedures are considered as indicators of relative importance among variables. One of the biggest advantages of stepwise multivariate analysis is that, given the attributes the model determines which attributes to include in the model. Furthermore, any type of interaction between variables can be included in the model statement. Variables can take continuous or discrete values.

Once the model is fit, then the significance of the variables in the model is assessed. This usually involves formulation and testing of a statistical hypothesis to determine whether the independent variables in the model are "significantly" related to the outcome variable. For this purpose, observed values of the response variable are compared to predicted values obtained from models with and without the variable in question. For statistical evidence the p-value associated with the χ^2 test is used. For example, a variable with p-value less than 0.05 is considered as *significant* in this study.

After fitting a model, it is necessary to interpret the values of the estimated coefficients. Testing the hypothesis that "the parameter estimate on the model is zero" is performed by chi-square test with $(t - 2)$ degrees of freedom. A chi-square statistics that is equal to zero implies that the independent variable does not affect the response variable. The estimated coefficients for the independent variables represent the rate of change of a function of the dependent variable per unit of change in the dependent variable. The *odds ratio*, denoted by ψ, is one of the measures of association and for logistic regression with a dichotomous independent variable is given by:

$$\psi = \frac{\pi(1) / [1 - \pi(1)]}{\pi(0) / [1 - \pi(0)]} \tag{2-5}$$

The *odds ratio* ψ approximates relative risk, i.e., how much more likely (or unlikely) it is for the outcome to be present among these with $x = 1$ than among those with $x = 0$. For example, if y denotes whether or not the bridge is damaged, and x denotes number of spans in a bridge (multiple, or single), then $\hat{\psi} = 2$ indicates that damage occurs twice as often among multiple span bridges than among single span bridges for the given data set. Here, $\hat{\psi}$ denotes the maximum likelihood value for ψ . For the multivariate case, the *odds ratio* shows the change in the probability of *Y*, when the value of x_1 advances one unit, from 0 to 1, while all other variables are held constant. Each β_j coefficient in equation (2-4) is the natural logarithm of the odds ratio for the variable x_j.

The univariate Wald statistics is used to test whether the independent variable significantly affects the outcome. The Wald-chi square value for each coefficient is calculated as:

$$Wald - chi\ square = \left(\frac{\beta_j}{std.error \text{ of } \beta_j} \right)^2 \qquad (2\text{-}6)$$

The statistical software package SAS was used to perform the logistic regression analysis. For each of the analyses performed in this study, the *p*-value associated with χ^2 test, parameter estimate, standard error, probability associated with Wald statistics and odds ratio are computed. A complete set of the empirical fragility curves obtained from the logistic regression analysis is provided in the Appendices.

SECTION 3
BRIDGE DAMAGE DATA FROM THE LOMA PRIETA, CA EARTHQUAKE

The 1989 Loma Prieta earthquake of magnitude 7.1 was the largest earthquake to occur in the San Francisco Bay area after the 1906 San Francisco earthquake. The earthquake caused damage and collapse of bridges in the area, 60 miles away from the epicenter located in the Santa Cruz Mountains. Sixty-two lives were lost including forty-one deaths from the collapse of the Cypress Viaduct, and one death on the San Francisco-Oakland Bay Bridge. The extent of the damage from the earthquake was limited due to the short duration of the earthquake. The impact of the earthquake was much more than the loss of life and direct damage. The Bay Bridge was out of service for 31 days; five of the six viaducts of the San Francisco Freeway system were also closed to traffic following the earthquake. Because of the San Francisco Bay area's geography, the closure of the Bay Bridge and other major arteries had significant economical consequences.

The cost of the earthquake to the transportation system was $1.8 billion. Damage to state-owned viaducts totaled about $200 million and damage to other state-owned bridges was about $100 million [Thiel, 1990]. About 5 percent of all the bridges that were affected by the earthquake sustained damage. Majority of the bridge damage was in the San Francisco Bay area, about 60 miles away from the epicenter. Thirteen of the state-owned bridges in the Bay area sustained major damage and were closed to traffic following the earthquake. Soft soil at bridge sites contributed to the extensive damage observed this far away from the epicenter.

Following the earthquake, Caltrans gathered bridge damage and repair cost data. The data, however, have not been studied in detail. In this study, the characteristics of the bridges exposed to ground shaking during the Loma Prieta earthquake were reviewed to obtain statistics for damaged bridges. Correlation studies were performed to identify the structural characteristics that most contribute to the observed damage. Several difficulties were encountered in performing the statistical analyses and the correlation studies. These difficulties included the limited damage data, lack of information on the ground motion levels at bridge sites and the definitions of damage states, which were too vague. Nevertheless, the available data were reviewed, and used for statistical analyses. Following the procedures described in Section 2, ground motion-damage relationships were developed for groups of bridges for which damage data are available. In this section, the data on bridge damage and the repair cost from the Loma Prieta earthquake, the correlation analyses and findings from these analyses are presented.

3.1 GROUND-MOTION CHARACTERISTICS

The epicenter of the Loma Prieta earthquake was located in the southern Santa Cruz Mountains. An outer zone of modified Mercalli intensity of VII extended more than sixty miles northwest to San Francisco and Oakland, and 30 miles southeast to Salinas and Hollister. Within these regions, free-field, peak horizontal ground accelerations exceeded 0.6g close to the source and were as high as 0.26g at a distance of 60 miles. Strong shaking lasted less than 15 seconds. Local soil conditions significantly influenced the spatial distribution of damage. The ground motions at soft soil sites in the Bay area, where much of the damage to bridges and viaducts occurred, were significantly greater than the motions recorded at nearby rock and stiff soil sites [Thiel, 1990].

At the time of this study, no contour maps based on the ground motion recordings were available for the Loma Prieta earthquake. In order to evaluate ground motion-damage relationships, the ground motion level at each bridge site was estimated from a scenario event. The scenario event for the Loma Prieta earthquake was generated using geographic information system (GIS). The resulting buffer zones for the bedrock level ground shaking are shown in figure 3-1. Because the available geology map was limited to a portion of the San Francisco Bay area, the ground motion levels were estimated only for the section of the Bay area shown in figure 3-1. The surface level ground shaking was obtained using the geology map developed by USGS and the attenuation relationship by Boore et al. [1997] for strike-slip faults. The maximum peak ground acceleration (PGA) level obtained using this attenuation relationship was 0.61g in the epicenter area.

The estimated PGA levels were compared to the available strong motion recordings. During the main shock of the Loma Prieta earthquake, strong motion records were obtained at 98 free-field stations maintained by the California Strong Motion Instrumentation Program (CSMIP) and U.S.

stations maintained by the California Strong Motion Instrumentation Program (CSMIP) and U.S. Geological Survey (USGS) [Thiel, 1990]. Table 3-1 lists the recorded PGA levels at 29 recording stations and those estimated by using Boore et al. [1997] attenuation relationship at these recording stations. The ground motion levels estimated by this attenuation relationship deviate from the recorded PGA levels at most of these locations.

The surface level PGA values were also estimated by using the attenuation relationship developed by Campbell [1991]. This attenuation relationship is solely based on data only from the Loma Prieta earthquake. The ground motion levels in the San Francisco Bay area obtained by using Campbell's attenuation relationship are mapped in figure 3-2. The locations of recording stations listed in table 3-1 are also mapped in figure 3-2.

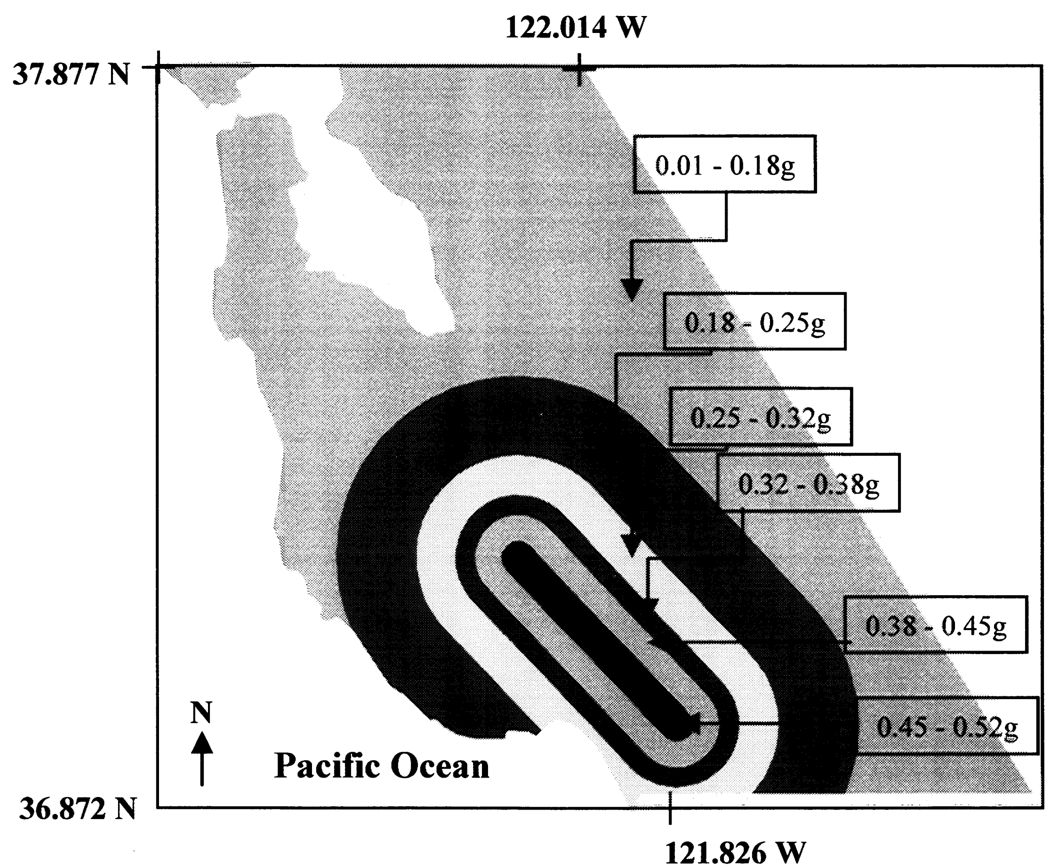


FIGURE 3-1 Buffer Zones for Ground Shaking at Bedrock Level from the Loma Prieta Scenario Event

TABLE 3-1 Estimated and Recorded PGA Levels from the Loma Prieta Earthquake

Name of the Recording Station	PHA1 (g) [1]	PHA2 (g) [1]	PHA BJF[2]	PHA Campbell[3]
Agnew	0.16	0.17	0.2122	0.205
Capitola	0.47	0.54	0.2963	0.2866
Corralitos	0.5	0.64	0.495	0.6092
Coyote Lake Dam – Downstream	0.19	0.17	0.1671	0.1682
Fremont - Calaveras Array	0.15	0.2	0.139	0.13
Fremont - Mission San Jose	0.11	0.13	0.1419	0.1331
Gavilon College	0.37	0.33	0.2821	0.2731
Gilroy #1	0.5	0.43	0.1663	0.1949
Gilroy #2	0.33	0.37	0.2692	0.2608
Gilroy #3	0.37	0.55	0.2468	0.2391
Gilroy #4	0.22	0.42	0.2371	0.2296
Gilroy #6	0.17	0.13	0.1151	0.1346
Gilroy #7	0.33	0.23	0.1763	0.1687
Gilroy - 2 st. Bldg.	0.25	0.28	0.2821	0.2731
Halls Valley	0.11	0.13	0.1716	0.1638
Hayward - BART Station	0.16	0.16	0.1071	0.099
Oakland - 2 st. Office Building	0.2	0.26	0.0852	0.099
Oakland - Outer Harbor Wharf	0.29	0.27	0.0825	0.1861
Piedmont Jr. High School	0.08	0.07	0.0473	0.0673
San Francisco International Airport	0.33	0.24	0.0985	0.1861
Santa Cruz	0.44	0.47	0.133	0.1561
Saratoga	0.34	0.53	0.3694	0.3719
SF - Diamond Heights	0.12	0.1	0.0478	0.0673
SF - Pacific Heights	0.05	0.06	0.0453	0.0673
SF - Rincon Hill	0.09	0.08	0.0463	0.0673
South SF - Sierra Point	0.11	0.06	0.0525	0.0673
Stanford - SLAC	0.29	0.19	0.0938	0.1083
Sunnyvale	0.22	0.19	0.2051	0.1979
Woodside	0.08	0.08	0.1235	0.1217

[1] PHA1 and PHA2 are the recorded values reported in Boore et al., [1993].

[2] Boore et al., [1997].

[3] Campbell, [1991].

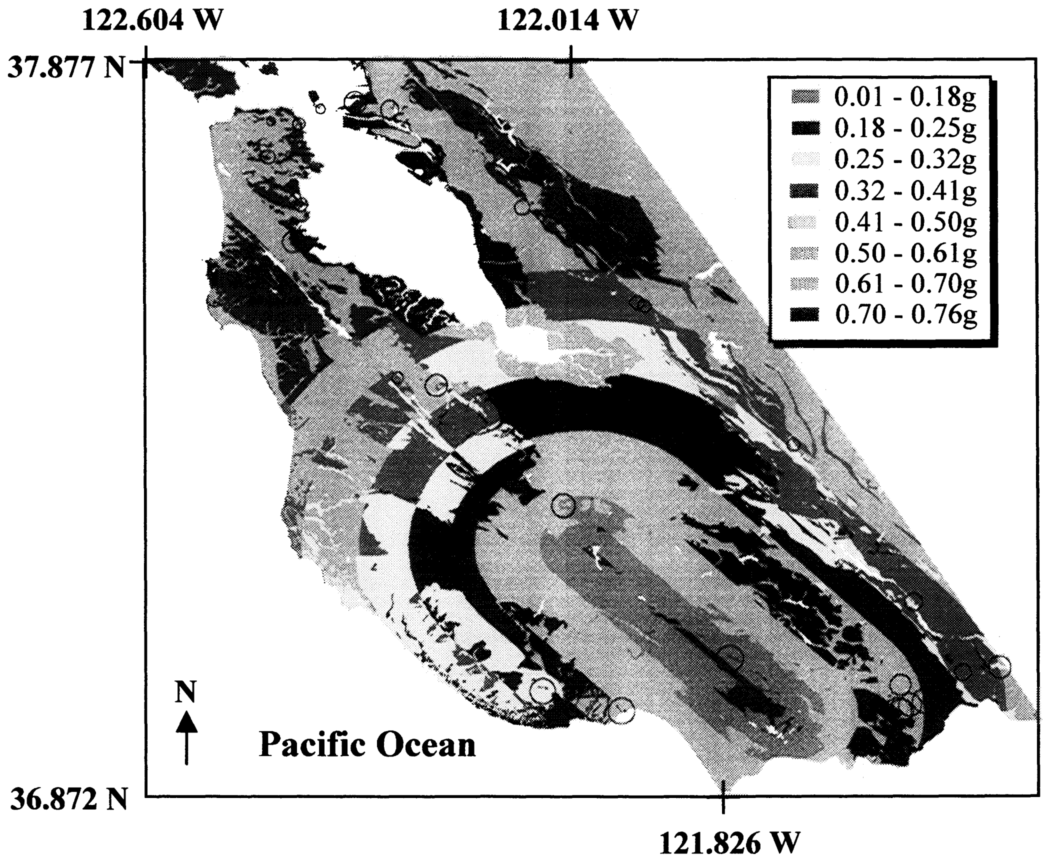


FIGURE 3-2 Sample Recording Stations and Estimated Ground Shaking Levels for the Loma Prieta Earthquake

The PGA levels at the recording stations obtained from the scenario event using Campbell's attenuation relationship are listed in table 3-1. Comparison of the results based on the two attenuation relationships, i.e., Boore et al. [1997] and Campbell [1991], shows that Campbell's attenuation relationship gives better estimates of the observed ground motion levels at some of the recording stations. The estimated values, however, do not agree well with the recorded ground motion levels at all locations. Figure 3-3 shows the comparison of the average recorded PGA levels with those estimated by using Campbell's [1991] attenuation relationship. Despite the discrepancies in the recorded and estimated ground motion levels, the estimated PGA levels based on Campbell's attenuation relationship were used in the correlation analyses.

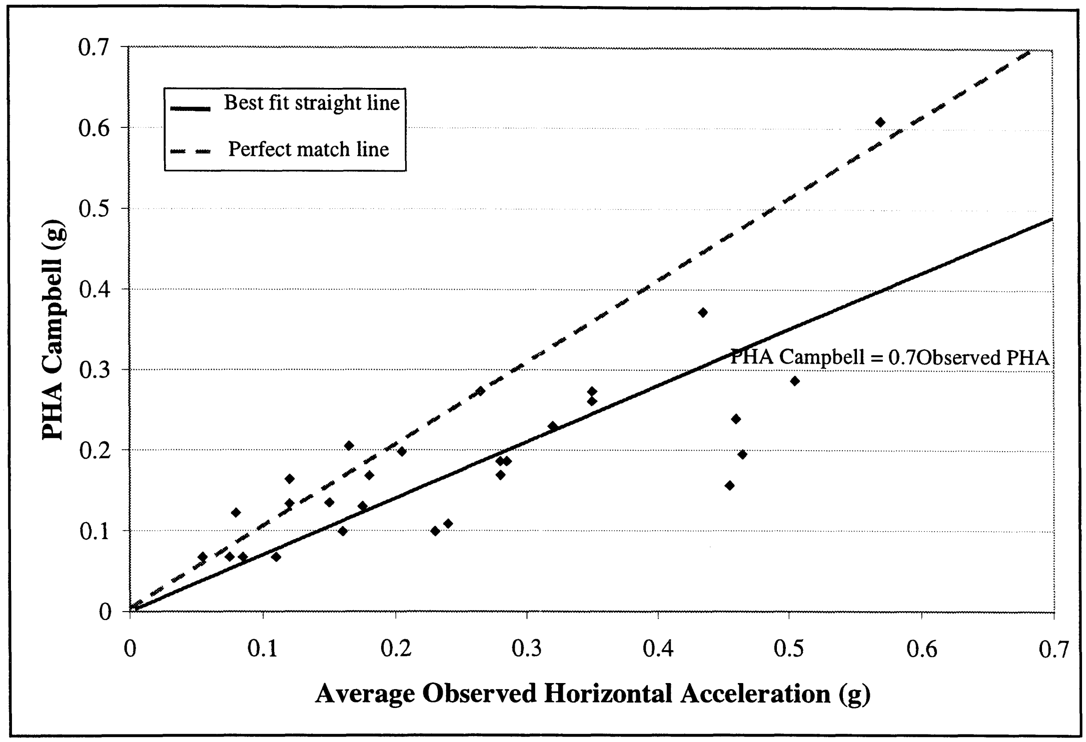


FIGURE 3-3 Comparison of Recorded and Estimated PGA Levels, Loma Prieta Earthquake

3.2 INVENTORY OF BRIDGES AFFECTED IN THE LOMA PRIETA EARTHQUAKE

The San Francisco Bay area encompasses twelve counties: Alameda, Contra Costa, Marin, Monterey, Napa, San Benito, San Francisco, San Mateo, Santa Clara, Santa Cruz, Solano and Sonoma Counties. The locations of state bridges in the twelve counties are shown in figure 3-4. The number of state and local bridges and the number of damaged state bridges in the twelve counties are listed in table 3-2.

The inventory of state and local bridges for the counties of the San Francisco Bay area was extracted from the Bridge Maintenance Database compiled by Caltrans [1993]. The structural characteristics, structural type and material, number of spans, abutment type, span continuity, design year indicating the seat width and column longitudinal reinforcement, substructure type, skew, and foundation type, were included in this database.

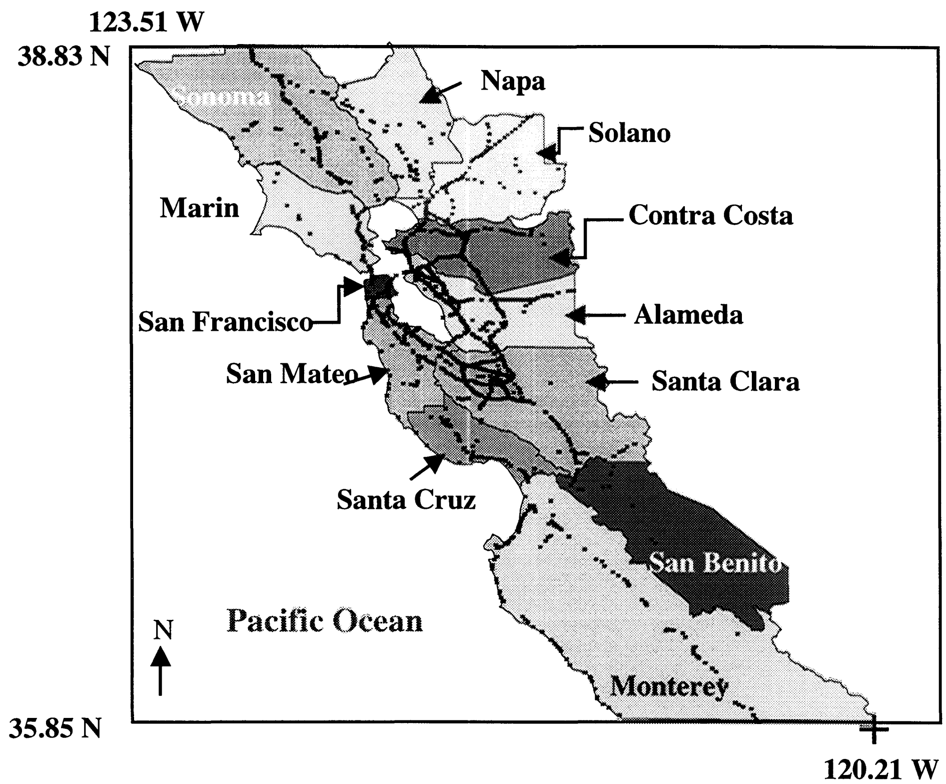


FIGURE 3-4 State Bridges in the San Francisco Bay Area

TABLE 3-2 Distribution of State and Local Bridges and the Number of Damaged State Bridges in the San Francisco Bay Area

County	Number of State Bridges	Number of Local Bridges	Total Number of Bridges	Number of Damaged State Bridges
Alameda	485	320	805	10
Contra Costa	277	331	558	10
Marin	88	119	207	2
Monterey	150	204	354	0
Napa	50	104	154	0
San Benito	23	44	67	3
Santa Clara	498	461	959	13
Santa Cruz	85	106	191	20
San Francsico	97	53	150	5
San Mateo	235	143	378	11
Solano	163	181	344	2
Sonoma	178	390	568	0
Total	*2,329*	*2,456*	*4,785*	*76*

As shown in table 3-2, there are 2,329 state and 2,456 local bridges in the twelve counties of the San Francisco Bay area. In this study, inventory and damage data were studied for only the state bridges since most of the damage reported in the Loma Prieta earthquake was for state bridges. Of the 2,329 state bridges, 2,131 carry highway traffic[1] and were gathered in the *highway bridge data set*. The number of bridges in the *highway bridge data set* classified by superstructure type and substructure material is shown in table 3-3. As shown in table 3-3, most of the bridges (1,883 out of 2,131) are concrete structures. Steel bridges form only about 7 percent of the bridge inventory in the area. Furthermore, more than 80 percent of the bridges damaged in the Loma Prieta earthquake were concrete structures.

Bridges exposed to ground shaking during the Loma Prieta earthquake were grouped in various data sets. For example, the set of 1,869 highway state bridges (with concrete superstructure and concrete substructure for multiple span bridges and with concrete superstructure for single span bridges) was referred to as the *concrete highway bridge data set*. Examples of concrete superstructure include concrete box gider, concrete girder, precast concret girders, cast-in place prestressed slab, precast prestressed box girder and precast prestressed I girder.

Next, bridges with homogeneous structural characteristics were selected from the *concrete highway bridge data set*. Bridges with heterogeneous characteristics (i.e., bridges with mixed

TABLE 3-3 Distribution of State Highway Bridges in Counties of the San Francisco Bay Area by Superstructure Type and Substructure Material

		Superstructure Type					
		Concrete	**Steel**	**Truss**	**Arch**	**Suspension**	**Others**
Substructure Type	**Concrete**	1,500	139	14	14	1	4
	Steel	0	1	0	2	1	0
	Masonry	0	0	0	2	0	0
	Unknown	14	3	0	0	0	0
	Single Span Bridges	369	11	0	25	0	23

[1] Structures identified with the descriptions given in Appendix B are excluded from the analyses. These descriptions are extracted from the Caltrans OSM&I Guidelines.

abutment or column bent types) were excluded from this data set in order to concentrate on studying the effect of structural component types on bridge damage, such as effect of abutment type (monolithic or non-monolothic), and number of columns per bent. Bridges with incomplete information were also excluded from this data set and the resulting data set was referred to as the *homogeneous bridge data set.*

Next, to obtain a complete data set for the correlation studies, only bridges that were exposed to PGA levels of 0.10g or higher were selected from the *homogeneous bridge data set.* The new data set was called the *correlation data set.* The threshold PGA level was selected as 0.10g because it is likely that bridges outside of the areas covered by the ground shaking map used in this study (see Section 3.1) experienced PGA levels less than 0.10g. Including only some of the bridges that were exposed to PGA levels below 0.10g would bias the results of the correlation studies. In this sub-section, statistics are presented for both the *highway bridge data set* and the *correlation data set.*

Figure 3-5 shows the distribution of the bridges in the *highway bridge data set* by design year. Seventy three percent of these bridges were designed using pre-1971 design standards. The distribution of the bridges in the *correlation data set* by design year is shown in figure 3-6. Sixty seven percent of the bridges in the *correlation data set*, i.e., concrete highway bridges (with homogeneous characteristics) exposed to PGA levels of 0.10g or higher in the Loma Prieta earthquake, were designed according to pre-1971 design standards. Figure 3-7 shows the distribution of single span bridges in the *correlation data set* by abutment type. The distribution of multiple span bridges by abutment type and column bent type in this data set are depicted in

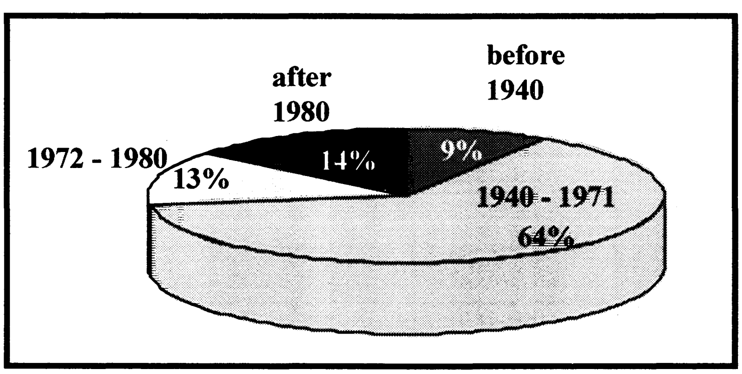


FIGURE 3-5 Distribution of State Highway Bridges in the San Francisco Bay Area by Design Year

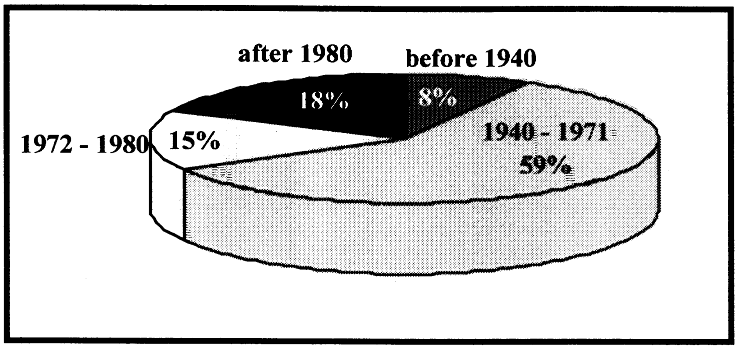


FIGURE 3-6 Distribution of Bridges in the San Francisco Bay Area by Design Year (*correlation data set*)

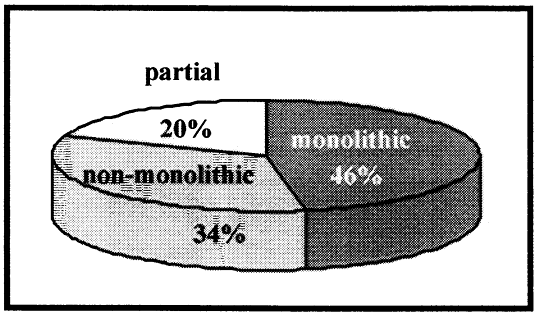


FIGURE 3-7 Distribution of Single Span Bridges by Abutment Type (*correlation data set*)[1]

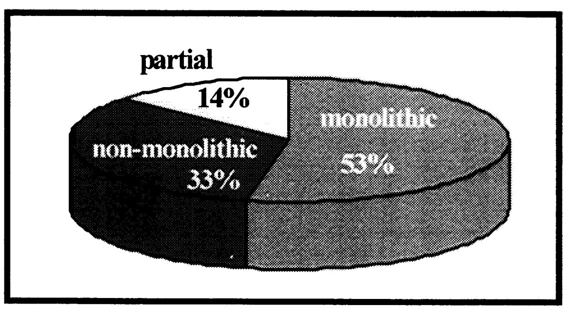


FIGURE 3-8.a Distribution of Multiple Span Bridges by Abutment Type (*correlation data set*)

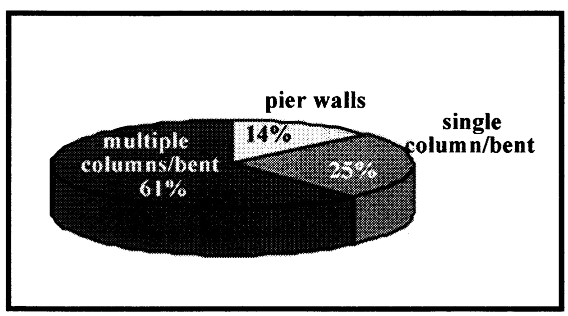


FIGURE 3-8.b Distribution of Multiple Span Bridges by Column Bent Type (*correlation data set*)[2]

---

[1] See table 2-2 for description of abutment types.

[2] Frame bents and pile bents were grouped as multiple column/bent.

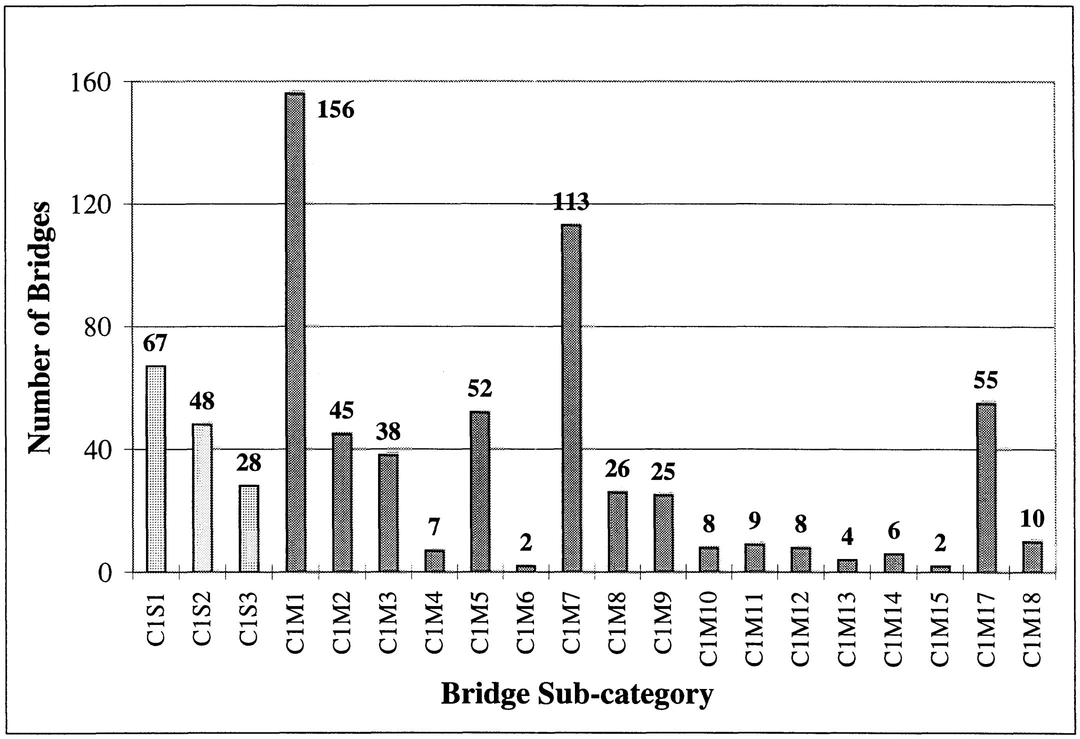


FIGURE 3-9 Distribution of Bridges by Bridge Sub-category (*correlation data set*)[1]

figures 3-8.a and 3-8.b, respectively. Almost half of the bridges in the *correlation data set* had monolithic abutments and about two thirds of the multiple span bridges had multiple columns per bent.

Next, the *correlation data set* of 1,112 bridges was grouped into bridge sub-categories listed in table 2-1. Figure 3-9 shows the distribution of bridges in the *correlation data set* by these sub-categories. Eighty percent of the bridges were multiple span bridges. The majority of the multiple span bridges was classified into sub-categories C1M1, C1M7, C1M17[2]. This classification was performed to examine the relationships between the structural characteristics and the ground motion levels and the estimated repair cost. The results from the correlation studies based on this bridge classification are presented in Section 3.4.

---

[1] See table 2-1 for description of the bridge sub-categories.

[2] C1M1: multiple span bridges with monolithic type abutments, continuous spans and multiple column bents.

C1M7: multiple span bridges with non-monolithic type abutments, continuous spans and multiple column bents.

C1M17: multiple span bridges with bin or cellular closure type abutments, continuous spans and pier walls.

Bridge Retrofit Status

Following the 1971 San Fernando earthquake, Caltrans adopted a cable restrainer seismic retrofit program to prevent superstructure unseating at piers and in-span hinges. At the time of the Loma Prieta earthquake, phase I retrofitting, i.e., use of cable restrainers, was completed for 1,260 of the bridges in California. About 200 of these bridges were affected in the Loma Prieta earthquake. Several bridges with phase I retrofitting suffered damage due to deficiencies in the joint restrainers. Upgrading of non-ductile columns was not given priority before the Loma Prieta earthquake since the failure of these components was expected to cause loss of serviceability rather than collapse. Consequently, phase II retrofitting, i.e. retrofitting of single column bents, was completed for only 262 bridges before the Loma Prieta earthquake. Following the Loma Prieta earthquake, Caltrans proposed an accelerated schedule for retrofitting 392 bridges that have single column bents and included 700 more bridges with multiple column bents in the program. The Caltrans database on retrofit history of bridges is not complete and includes only about 30 of these bridges. Therefore, correlation studies for retrofitted bridges were not performed.

3.3 COMPILATION AND REVIEW OF BRIDGE DAMAGE DATA

Following the earthquake, ten counties in the San Francisco Bay area were declared as state and federal disaster areas: Alameda, Contra Costa, Marin, Monterey, San Benito, San Francisco, San Mateo, Santa Clara, Santa Cruz and Solano Counties. Seven of these counties (Alameda, Contra Costa, Marin, San Francisco, San Mateo, Santa Clara, and Santa Cruz) were within the jurisdiction of Caltrans District 4. Caltrans post-earthquake investigation teams (PEQIT) examined about 350 bridges in the San Francisco Bay area immediately after the earthquake. District 4, whose jurisdiction approximates the area of the most intense earthquake damage, was responsible for 1,896 state bridges of which about 4 percent sustained some degree of damage (mostly *minor*) during the earthquake. Six state bridges were damaged in other counties. State bridges in Monterey and San Benito (District 5) counties sustained only *minor* damage, while only one state bridge in Solano County (District 10) sustained damage.

Forty-three bridges maintained by local government were reported as damaged. Five of those bridges were closed to traffic for some period of time, but none of the local bridges collapsed [Thiel, 1990].

Bridge damage data gathered by the PEQIT were compiled by Caltrans engineers following the earthquake [Caltrans, 1989]. In this study, the list of damaged bridges given by the PEQIT was reviewed. A total of 76 damaged bridges, which were reported with *minor* or *major* damage, were identified.

3.3.1 Damage State Descriptions

In the Caltrans PEQIT report [1989], damaged bridges were reported either without any detailed description of the damage or with *minor* or *major* damage. The definitions for *minor* and *major* damage covered a broad range. For example, *major* bridge damage included total or partial collapse, as well as damage to bearings and anchor plates, and significant cracking and spalling of the columns. *Minor* damage referred to failure or damage of keeper plates and anchor bolts, spalling and cracking of columns, cracking of abutment walls, railing damage, shear key failure, joint seal failure, approach settlement and damage to restrainers. The broad definition for *minor* and *major* damage states made it more difficult to identify structural characteristics that most contribute to damage. During the post-earthquake investigation for the Northridge earthquake, bridges were reported to be in one of the four (*minor*, *moderate*, *major* and *collapse*) rather than two damage states. The four level damage assessment provided a better description of damage details. Furthermore, the empirical ground motion-damage relationships developed for various damage states are more informative. The detailed damage state definitions defined for the concrete bridges and the empirical ground motion-damage relationships for the Northridge earthquake are presented in Section 4. A comparison of the damage descriptions for the Loma Prieta and the Northridge earthquakes revealed that *minor* and *moderate damage* in Northridge earthquake reconnaissance reports correspond roughly to *minor damage* reported in those of the Loma Prieta earthquake. Similarly, *major damage* and *collapse* in the Northridge earthquake damage reports correspond approximately to *major damage* used in the damage reports for the Loma Prieta earthquake.

3.3.2 Description of Bridge Damage

Several statistics related to bridge damage were obtained in this study. Table 3-4 gives the characteristics of the collapsed bridges, summary description of damage and estimated repair costs. The san Francisco-Oakland Bay Bridge was a steel structure with 42 spans located on bay mud and was built in 1936. It suffered partial collapse. The Cypress Viaduct, a concrete box girder bridge built in 1957, was also located on bay mud. Forty-eight bents of the double deck structure suffered collapsed. The Struve Slough Bridge, a concrete slab bridge was subjected to relatively higher PGA levels (0.41g) and suffered damage to its pile bents. In addition to the two double deck structures that collapsed, four others suffered damage in the Loma Prieta earthquake. These viaducts were part of the San Francisco Freeway system (Terminal Separation, Embarcadero, Central Viaduct, Southern Freeway Viaduct). All of the four viaducts and the China Basin Viaduct - a single deck viaduct in the San Francisco Freeway system - were closed to traffic following the earthquake. The damage to the viaducts was mostly shear fracture and/or shear failure of columns and knee joints.

The distribution of the 76 bridges by design year is given in figure 3-10. About 80 percent of the damaged bridges were designed by pre-1971 design standards. Only one bridge that was designed according to more recent design standards (bridge No. "33 0483F" - Southbound Connection Overcrossing in Alameda County) sustained *major* damage to its outriggers. Figure 3-11 shows the distribution of bridge damage by service type on the bridge. Two of the 76 damaged bridges were concrete railroad bridges and suffered minor damage.

Fourteen of the 76 damaged bridges were steel structures. Among these fourteen bridges only the San Francisco-Oakland Bay Bridge suffered *major* damage (partial collapse) while other steel bridges suffered *minor* damage. For example, anchor bolt failures were observed on San Mateo-Hayward Bridge. No evidence of structural steel-girder failure was observed. All steel bridges that suffered damage were built before 1972. Estimated ground motion levels at these bridge sites ranged from 0.19g to 0.29g and the soil type at these site was either soft soil or bay mud.

Fourteen of the 60 damaged concrete bridges were outside the boundaries of the ground motion map shown in figure 3-2, thus PGA levels could not be estimated for these bridges. Three

TABLE 3-4.a Characteristics of Bridges that Collapsed in the Loma Prieta Earthquake

Bridge No.	Bridge Name	Route	No. of Spans	Superstr. Type[1]	Abut. Type[2]	Substr. Type[3]	Span Continuity	Design Year	Skew[4]	Soil Type[5]	Retrofit History
33 0025	San Francisco - Oakland Bay Bridge	I-80	42	STL, STT	N	H, N	discontinuous	1936	0	D	Phase I, 1962
33 0178	Cypress Street Viaduct	I-880	42	CB	B	H, I, J	discontinuous	1957	99	D	Phase I, 1977
36 0088L/R	Struve Slough	SR-1	22	CSC	A	I	continuous	1964	30	C	Phase I, 1984

TABLE 3-4.b Damage Description of Collapsed Bridges in the Loma Prieta Earthquake

Bridge No.	PGA (g)	Damage Description	Estimated Repair Cost ($)[6]
33 0025	0.19	Upper and lower closure of spans at Pier E9 fell; spans at Pier E23 were near failure; concrete pedestal base of Pier E17 cracked; connection bolts at Piers E17 through E23 damaged.	7,033,600
33 0178	0.06 - 0.27	Collapse of 48 bents causing the upper roadway to collapse onto the lower roadway; demolished.	250,484,000
36 0088L/R	0.41	Many of the pile extension bents failed and a major portion of the structure collapsed; demolished	6,335,400

[1] CB: concrete box girder bridge; CSC: continuous concrete slab bridge; STL: through continuous steel truss bridge, STT: through steel truss bridge.
[2] A: diaphragm; B: seat; N: wall type.
[3] H: multiple columns/bent; J: single column/bent; I: pile bents; N: pier walls.
[4] 99 represents varying skew along the bridge.
[5] C: younger and softer alluvial deposits; D: bay mud [Campbell, 1991]
[6] Source: Bridge damage assessment forms for the Loma Prieta earthquake, [Caltrans, 1989]; in 1989 dollars.

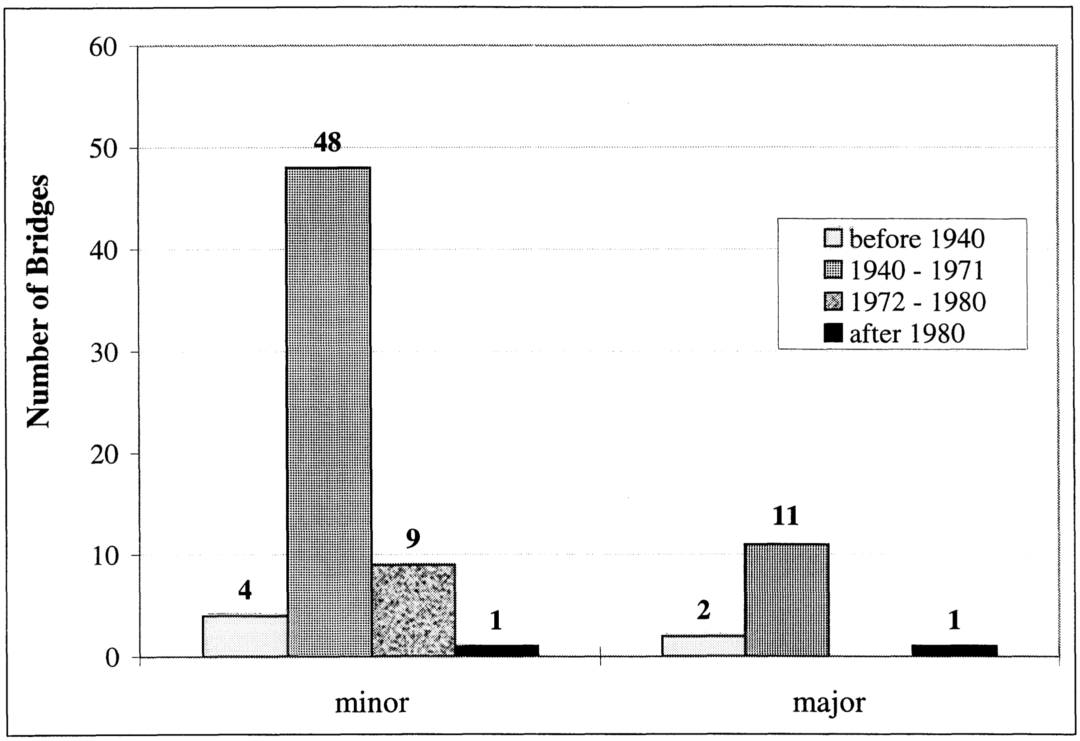


FIGURE 3-10 Distribution of All Damaged Bridges by Design Year

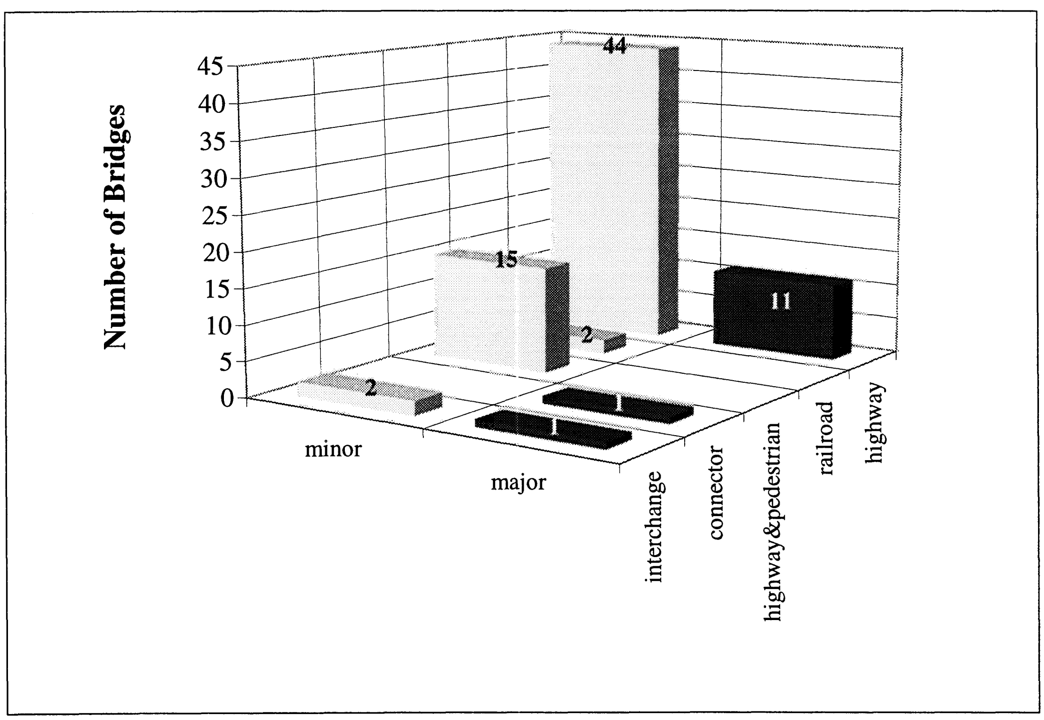


FIGURE 3-11 Distribution of All Damaged Bridges by Service Type and Damage State

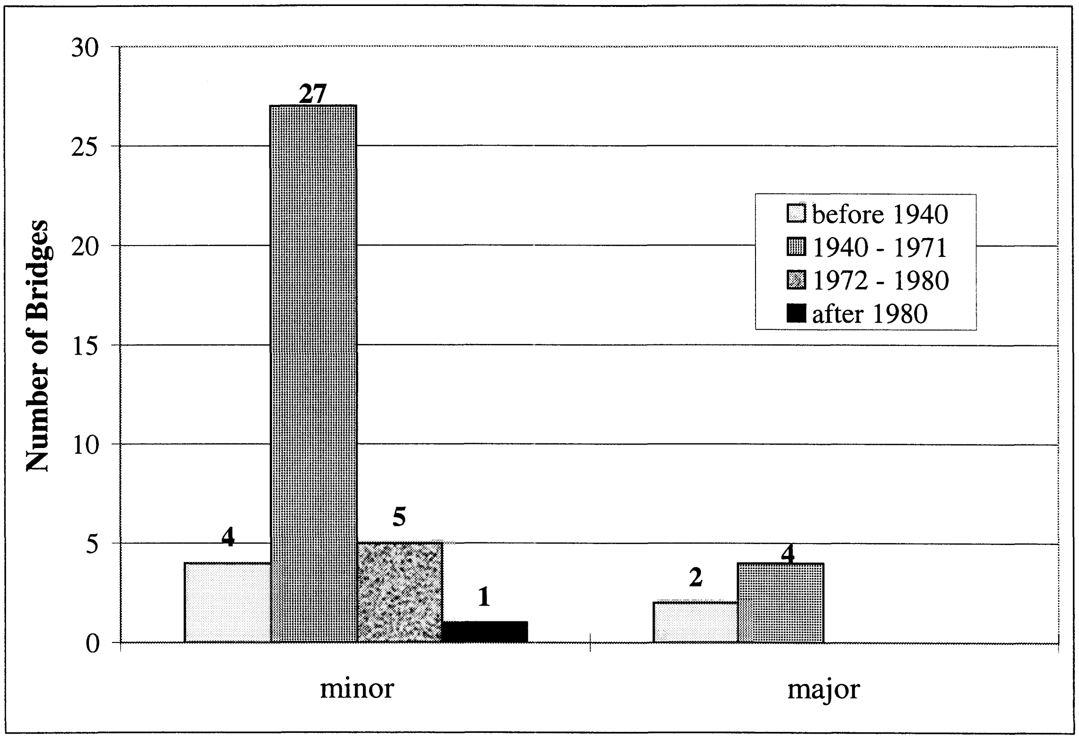


FIGURE 3-12 Distribution of Damaged Concrete Highway Bridges by Design Year for PGA $\geq$ 0.10g

Other bridges experienced PGA levels below 0.10g. Figure 3.12 shows the design year distribution of the 43 damaged highway bridges with concrete super and sub-structure subjected to PGA of 0.10g or larger. Fifteen of these damaged concrete highway bridges had heterogeneous characteristics. Consequently, only 28 of the 62 concrete bridges, which were reported as damaged, were included in the *correlation data set.* It should be noted that for the reasons described above, only 6 of the 14 bridges that suffered *major* damage were included in the *correlation data set.*

Distribution of damaged bridges in the *correlation data set* by bridge sub-categories, listed in table 2-1, is shown in figure 3-13. All damaged single span bridges had non-monolithic type of abutments. Damage for multiple span bridges was observed mostly for non-monolithic abutment types (sub-categories C1M7, C1M8 and C1M9[1]).

[1] C1M7: Multiple span bridges with non-monolithic abutment type, continuous span, multiple column bents.
C1M8: Multiple span bridges with non-monolithic abutment type, discontinuous span, multiple column bents.
C1M9: Multiple span bridges with non-monolithic abutment type, continuous span, single column bents.

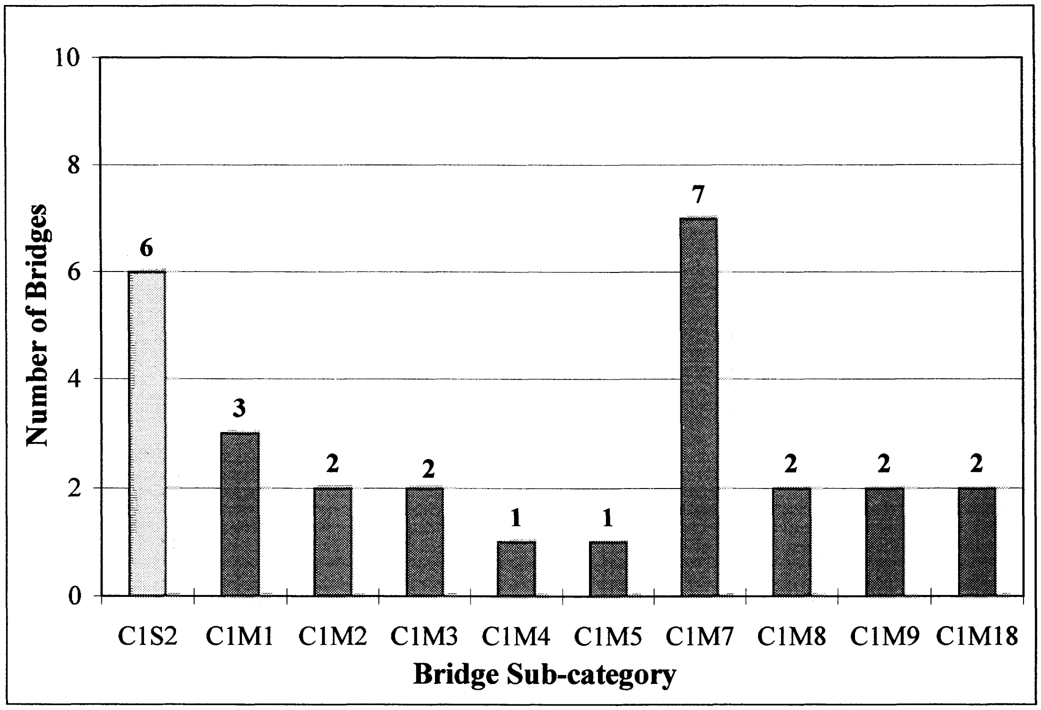


FIGURE 3-13 Distribution of Damaged Bridges by Bridge Sub-Categories (*correlation data set*)

In order to compare the fragility curves provided in HAZUS [1997] with the observed data, bridges in the *highway bridge data set* were classified according to HAZUS bridge classification. Figure 3-14 shows the distribution of damaged bridges, which were exposed to PGA levels 0.10g or higher, according to that classification. Twenty-four of the bridges in the highway data set had span lengths larger than 500 feet, therefore were classified into HAZUS classes HBR1 and HBR7 (*major bridges*). However, only two of the twenty-four major bridges were reported with a damage state in the PEQIT reports and the PGA level at one of these two bridges was 0.07g. Hence, there is only one HBR1 bridge listed in figure 3-14.

3.3.3 Data Error in the Inventory

As discussed in Section 2.1, some of the attribute values for abutment type, column bent type and joint location were found to be incorrect in the database used in this study. For all the bridges damaged in the Loma Prieta earthquake, the SMS and BIRIS databases were compared and attributes with discrepancies were identified. Due to insufficient resources, however, these attributes were not corrected.

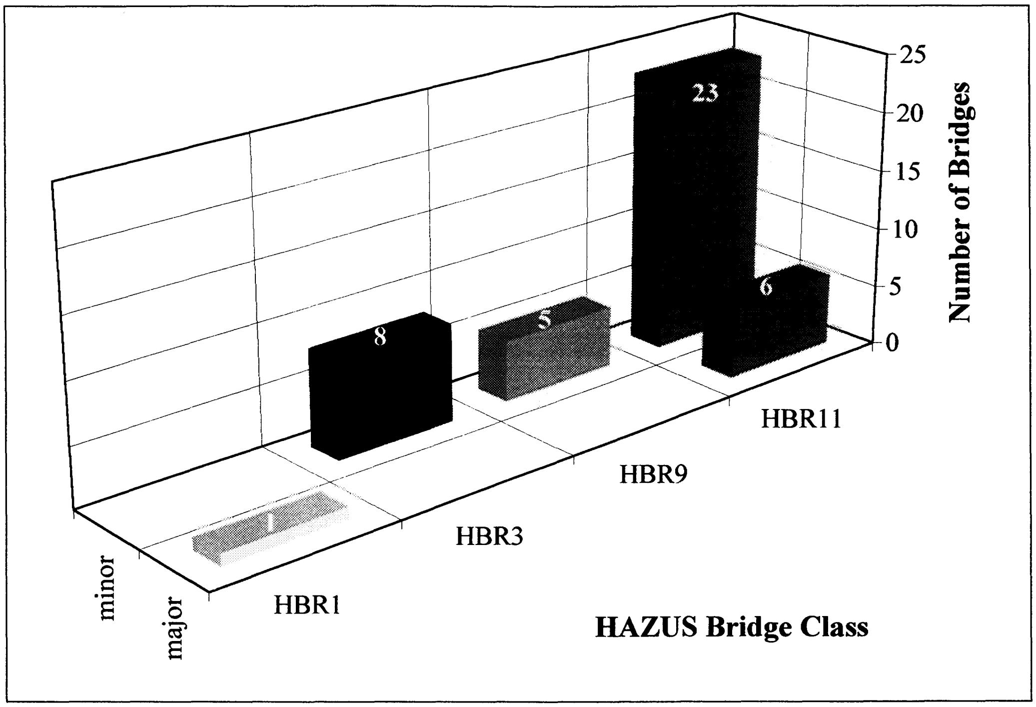


FIGURE 3-14 Distribution of Damaged Concrete Bridges by HAZUS Bridge Classification and Damage State for PGA $\geq$ 0.10g

3.3.4 Relative Importance of Attributes that Most Contribute to Damage

A multivariate logistic regression analysis was performed to identify the characteristics that contribute most to bridge damage. The analyses were conducted for: (*i*) multiple span bridges in the *correlation data set*, and (*ii*) all bridges in the *correlation data set*. Peak ground acceleration and abutment type were found to be the most important characteristics in both analyses. The design year was found to be an important attribute when all bridges were analyzed but it was not among the important attributes for the multiple span bridge data set. Skew, however, was found to be among the important structural characteristics for multiple span bridges.

3.4 CORRELATION STUDIES FOR BRIDGE DAMAGE

3.4.1 Empirical Fragility Curves

Correlation studies were performed to identify structural characteristics that are most susceptible to damage. The data set was first grouped to obtain damage frequencies at different PGA levels and damage probability matrices (i.e., probability of being in a damage state). Tables 3-5 and 3-6 show the damage matrix and the damage probability matrix for multiple span bridges included in the *correlation data set.*

Empirical fragility curves were developed from the damage matrices using logistic regression analysis. Table 3-7 lists the different types of attributes considered in grouping bridges to obtain empirical fragility curves. The empirical fragility curves developed based on these damage matrices are presented in Appendix C. As can be inferred from figure 3-13, no damaged bridges

TABLE 3-5 Damage Matrix for Multiple Span Bridges

	Peak Ground Acceleration (g)						
Observed damage	**0.1 - 0.2**	**0.2 - 0.3**	**0.3 - 0.4**	**0.4 - 0.5**	**0.5 - 0.6**	**0.6 – 0.7**	**total**
None	338	176	24	4	1	1	544
Minor	4	10	3	0	3	0	20
Major	0	0	0	2	0	0	2
Total	342	186	27	6	4	1	566

TABLE 3-6 Damage Probability Matrix for Multiple Span Bridges (%)

	Peak Ground Acceleration (g)					
Observed damage	**0.1 - 0.2**	**0.2 - 0.3**	**0.3 - 0.4**	**0.4 - 0.5**	**0.5 - 0.6**	**0.6 - 0.7**
None	98.83	94.62	88.89	66.67	25	100
Minor	1.17	5.38	11.11	0	75	0
Major	0	0	0	33.33	0	0

TABLE 3-7 Characteristics of Data Groups Used in Correlation Analyses

Dependent Variable	**Independent Variable**	**Data Set Grouped by**	**Results are Presented in**
damage state	estimated PGA values (Campbell's attenuation relationship [1991])	• number of spans • abutment type • column bent type • bridge sub-categories given in table 2-1 • design year • HAZUS classes	Appendix C

were reported for some of the bridge sub-categories. For some of the bridge sub-categories, although a number of bridges were reported to sustain damage, the number of bridges or the distribution of bridges at different PGA levels were not adequate to obtain a statistically satisfactory fit of the fragility curves. Due to lack of data, correlation studies were not performed for different sub-categories of steel bridges.

Figure 3-15 shows the empirical fragility curves for multiple span bridges. The comparison of the empirical fragility curves with the observed data for each damage state is shown in figure 3-16. Examples of fragility curves listed in table 3-8 are shown in figures 3-17 through 3-19. Each figure consists of two graphs:

- ***GRAPH A*** shows the probability of being in or exceeding a given damage state at different PGA levels,
- ***GRAPH B*** shows the probability of being in a damage state at different PGA levels.

The ratio of bridges in each damage state and statistical significance test results are presented in these figures. The number of bridges in each damage state, i.e., *no damage*, *minor damage* and *major damage*, are listed. The statistical parameters are valid for both graphs and were briefly discussed in Section 2.3. In general, a model is accepted with a p-value of the Wald chi-square statistic at a significance level of less than or equal to 0.05. A chi-square test with (t - 2) degrees of freedom is used to test whether the parameter estimate on the model is zero. The variable t represents the number of response variable levels. When data exist only for two damage states, the chi-square statistics is not applicable and is denoted by N/A. The odds ratio reported in these figures is the exponentiated value of the corresponding parameter estimate. Note that for parameter estimates larger than about 6.5, the odds ratio becomes larger than 1,000 and is reported as 999 indicating sparse data.

TABLE 3-8 Description of Example Fragility Curves for Damage States

Dependent Variable	Independent Variable	Data Set Grouped by	Shown in Figure
Damage state	Estimated PGA values (Campbell's attenuation relationship [1991])	• single span bridges	3-17
		• multiple span bridges with monolithic abutment type and multiple columns/bent (C1M1)	3-18
		• bridges designed between 1940 and 1971	3-19

3.4.2 Comparison of HAZUS Fragility Curves and Observed Damage

Empirical fragility curves were developed also for bridges grouped according to the HAZUS bridge classification. Damage was reported for bridge classes HBR-1, 3, 9 and 11. Number of bridges in bridge class HBR1 was not sufficient for statistical analysis. Therefore, empirical fragility curves were obtained using logistic regression analysis for the bridge classes HBR-3, 9 and 11. The fragility curves provided in HAZUS and the empirical values were compared for each of these classes. Figure 3-20 shows the comparison of exceedance probabilities and probabilities of being in a given damage state for the HAZUS bridge class HBR11. For class HBR11, the chi-square statistics suggests a good fit for the data points. However, as observed in figure 3-20, the curves obtained from the regression analyses do not match to those provided in HAZUS [1997]. The curves for HBR11 overestimate the probability of being in *major* damage state at most of the PGA levels. They overestimate the probability of being in *minor* damage state at lower PGA levels and underestimate it at higher PGA levels. Comparisons of empirical and the HAZUS fragility curves for the other classes are given in Appendix C. The observed damage data do not appear to agree well with the empirical ground motion-damage relationships for the majority of the classes. Similar to HBR11, the fragility curves for other HAZUS bridge classes overestimate the exceedance probabilities at all ground motion levels for any given damage state.

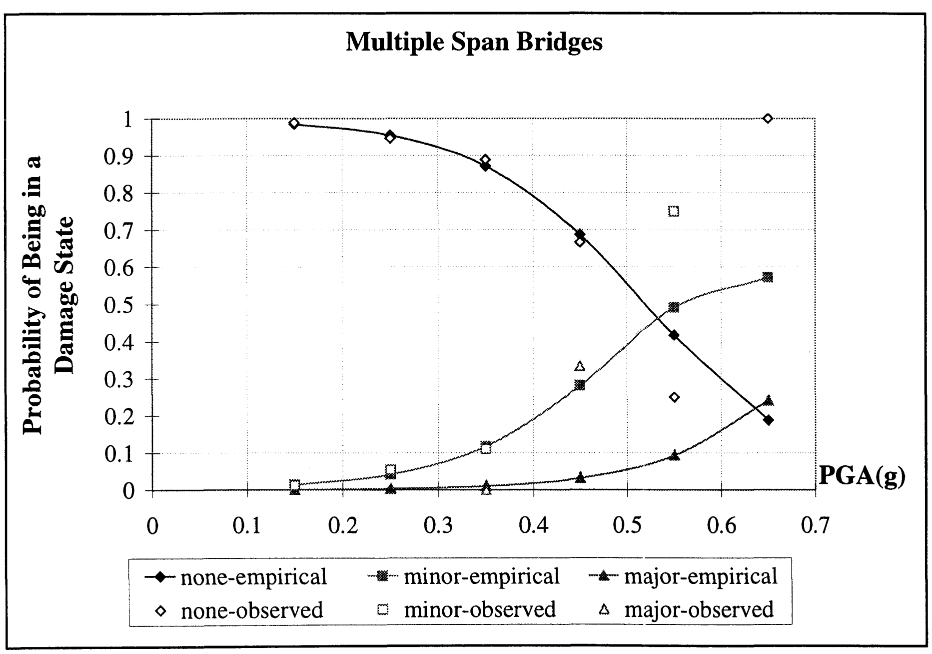
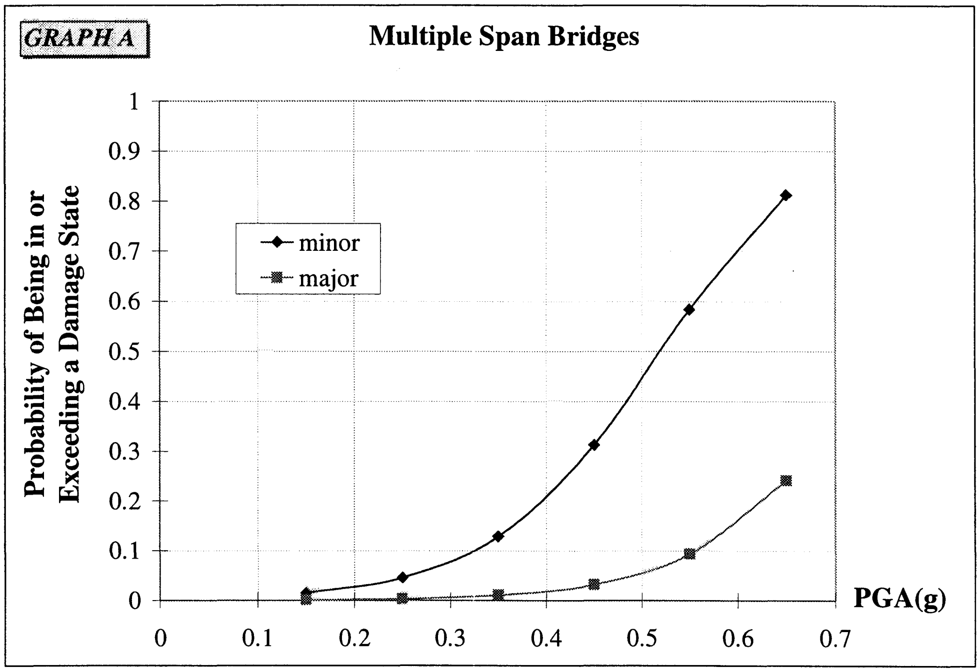


FIGURE 3-16 Empirical Fragility Curves and Observed Data for Multiple Span Bridges for Different Damage States

Number of bridges Chi-square with 1 DF	544, 20, 2 1.3654; p = 0.0001		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
11.2618	1.9823	0.0001	999



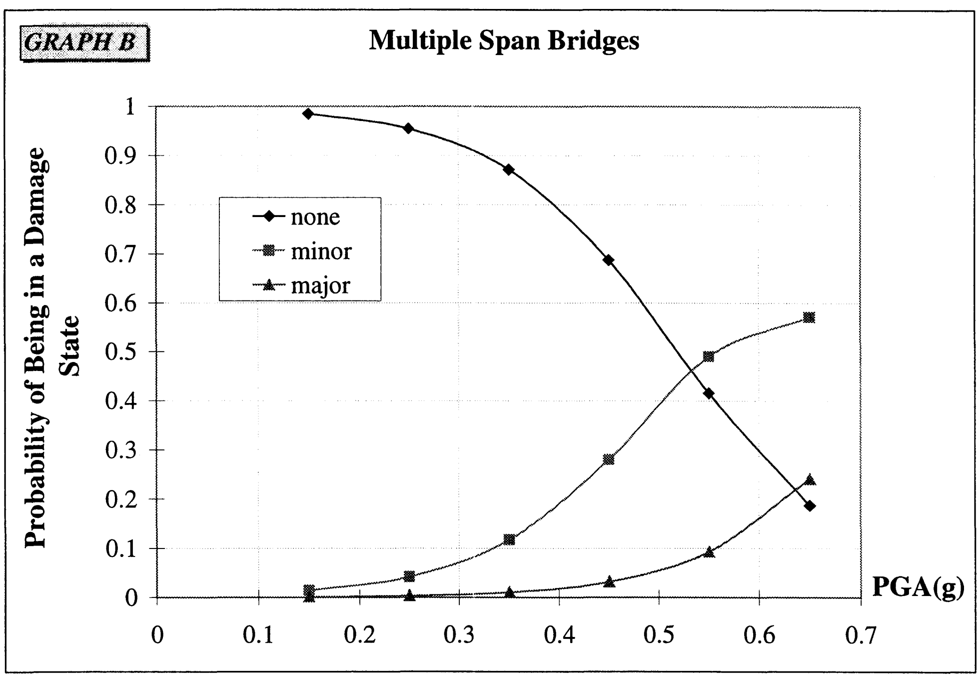


FIGURE 3-15 Empirical Fragility Curves for Multiple Span Bridges

Number of bridges Chi-square with 1 DF	137, 2, 4 9.5881; p = 0.002		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
21.9938	5.3284	0.0001	999

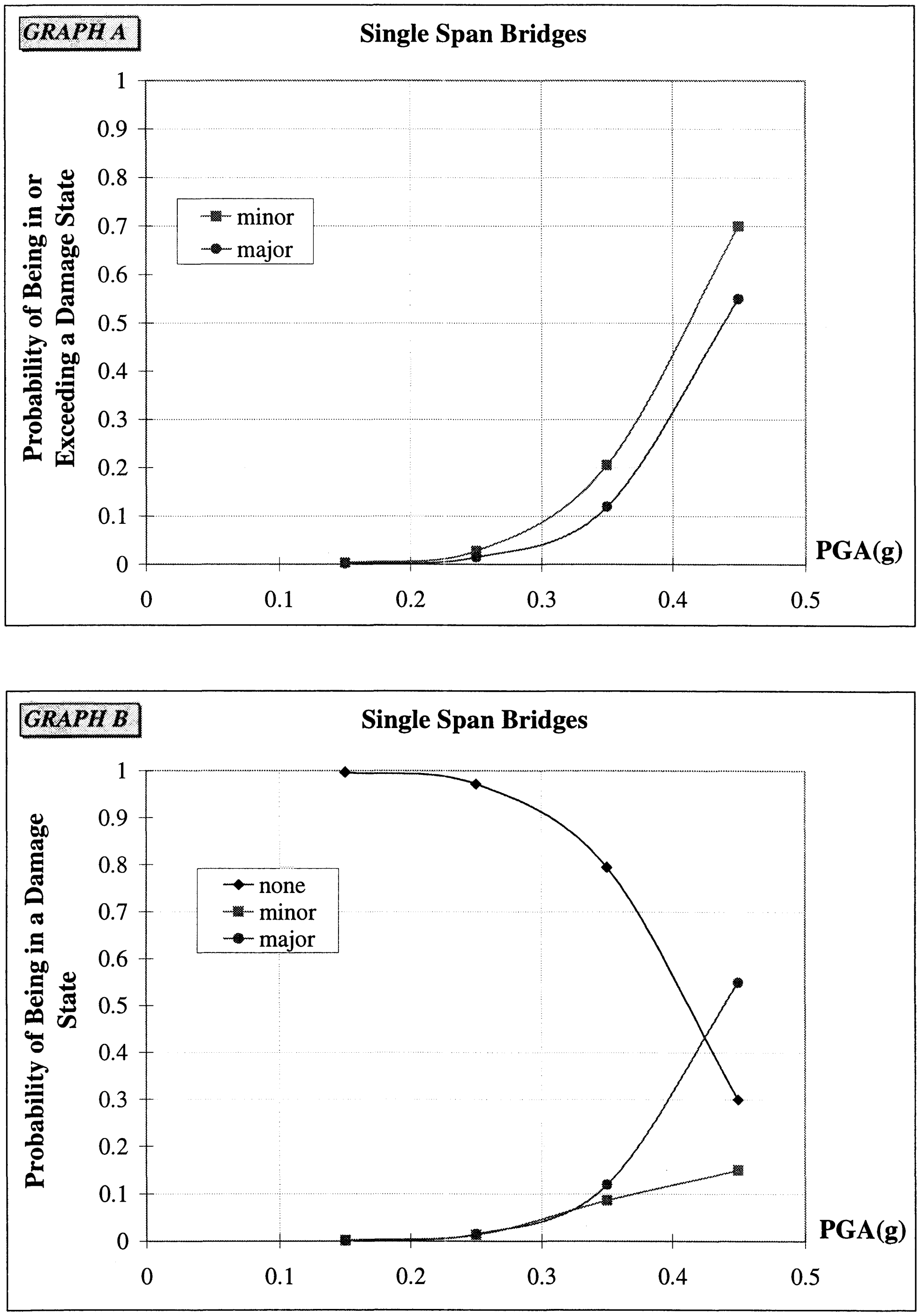
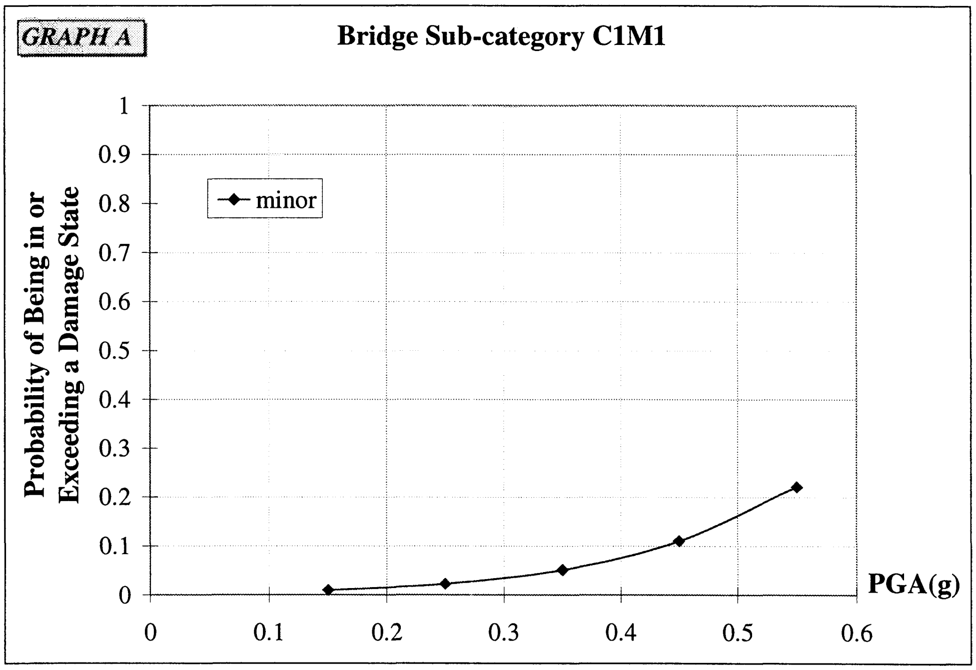


FIGURE 3-17 Empirical Fragility Curves for Single Span Bridges

Number of bridges Chi-square	153, 3, 0 N/A		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
8.3065	4.4761	0.0635	999



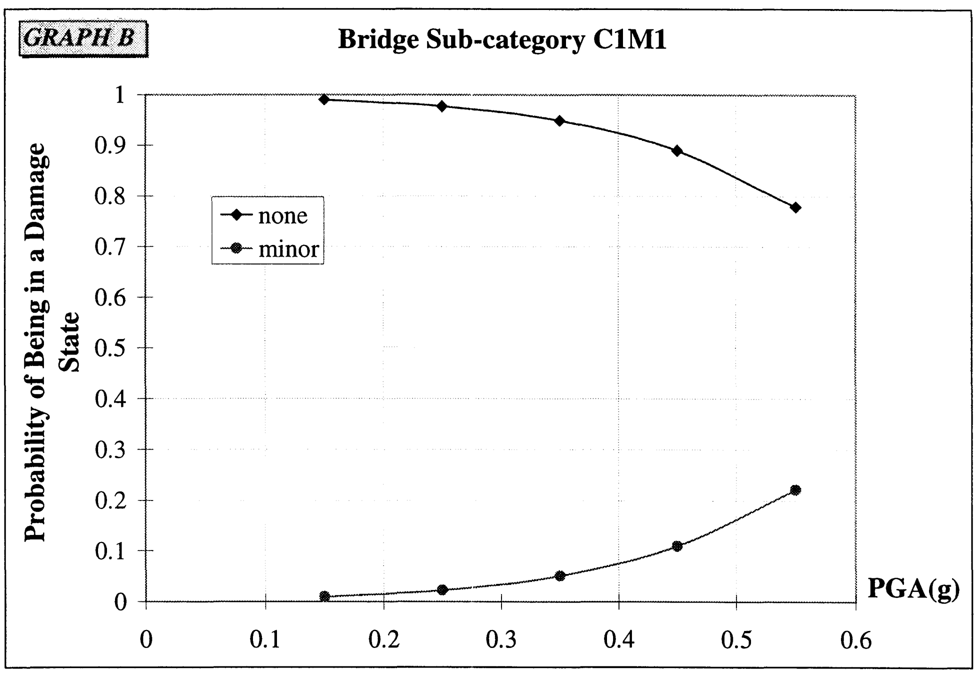
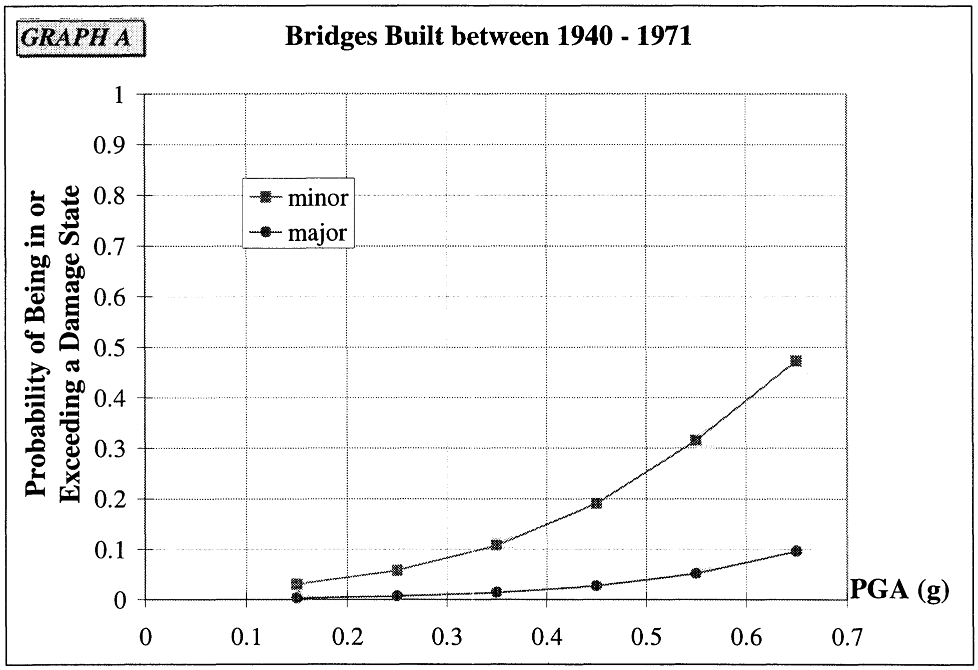


FIGURE 3-18 Empirical Fragility Curves for Multiple Span Bridges with Monolithic Type Abutments, Continuous Spans and Multiple Column Bents (Sub-category C1M1)

Number of bridges **Chi-square with 1 DF**	639, 27, 4 0.4658, p = 0.4949		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
6.6853	1.4996	0.0001	800.54



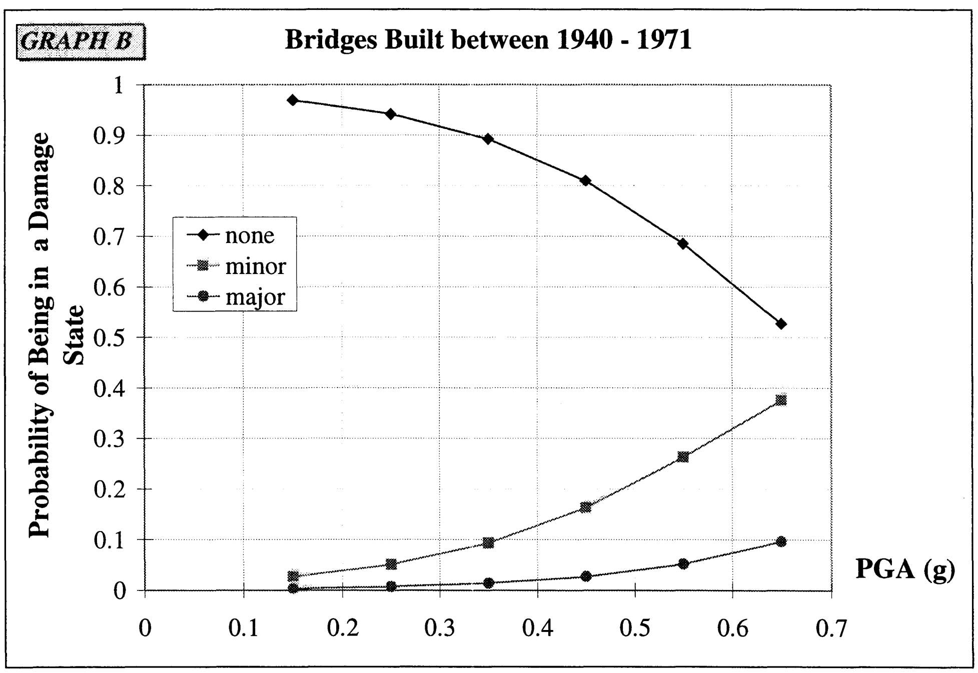
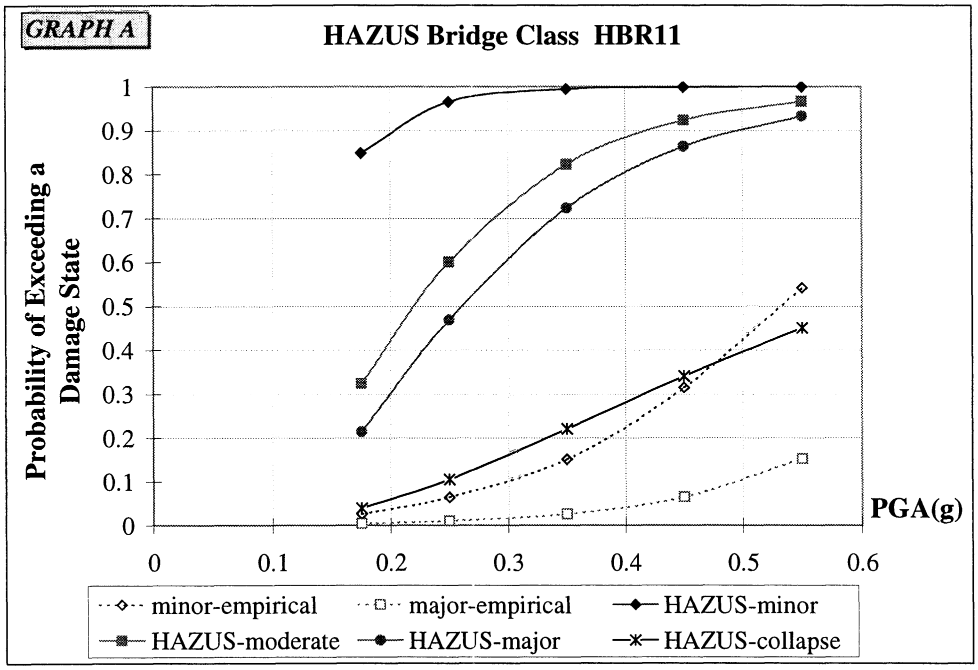


FIGURE 3-19 Empirical Fragility Curves for All Bridges Built between 1940 and 1971

Number of bridges Chi-square with 1 DF	522, 24, 6 0.3591, p = 0.841		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
9.0553	1.8627	0.0001	999



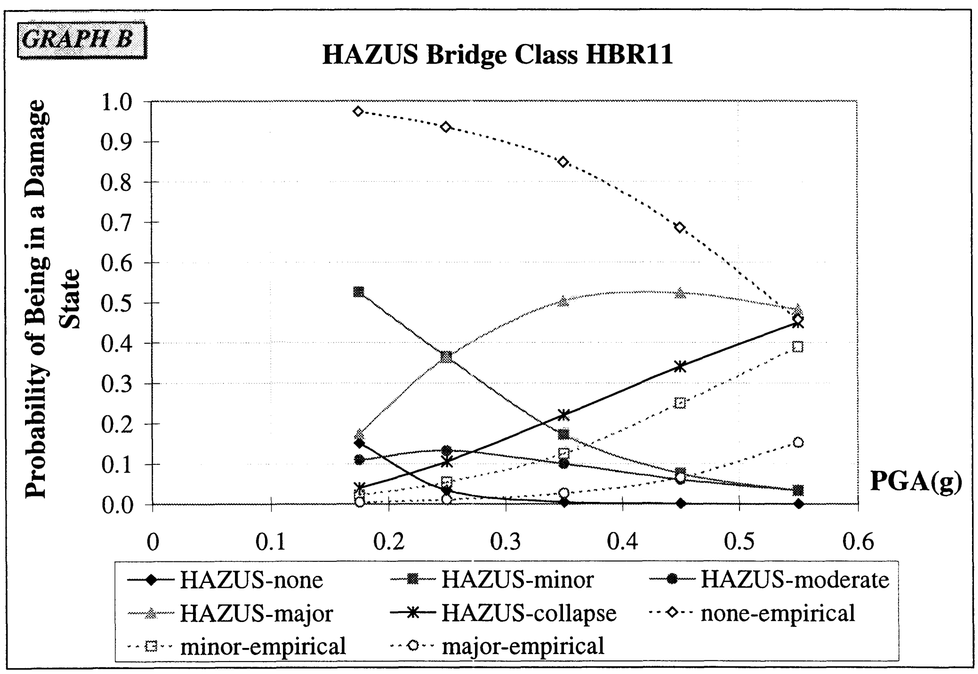


FIGURE 3-20 Comparison between the Empirical and the HAZUS Fragility Curves for HAZUS Bridge Class HBR11

3.5 ESTIMATED REPAIR COST

3.5.1 Description of Repair Cost Data

An extensive database on the estimated repair costs was compiled from the damage assessment forms obtained from Caltrans [1989]. This database includes total estimated repair cost and more detailed information on repair work and cost for 84 bridges. A total of about $280,000,000 was reported as repair cost in these reports. Ninety percent of the total estimated repair cost was for the repair of the Cypress Viaduct. The majority of the 84 bridges are located in Alameda, San Francisco, San Mateo, Santa Clara and Santa Cruz Counties. Repair cost estimates were reported for only two bridges in other counties: one in San Benito County ($30,000) and the other in Monterey County ($5,600). Tables 3-9 and 3-10 show estimated repair cost for damaged bridges by damage state, and structural type and number of spans. Repair costs were reported for 47 bridges which were not reported as damaged in the PEQIT reports. The damage state distribution for ranges of estimated repair cost is shown in figure 3-21. In addition, the repair cost for various components were included in the repair cost database. Data on repair cost by structural components, however, were not reported for all bridges. For sixty-six of the 84 bridges, a breakdown of repair cost by component was provided, while for others only the total repair cost and a general description of the repairs were given. No repair cost was reported for 19 bridges that suffered *minor* damage.

TABLE 3-9 Distribution of Estimated Repair Cost by Damage State

Damage state	Number of Bridges	Estimated Repair Cost
major	14 (12 with repair cost estimate)	$272,674,010
minor	62 (25 with repair cost estimate)	$2,400,510

TABLE 3-10 Distribution of Estimated Repair Cost by Structural Type and Number of Spans

Superstructure Type	Number of Spans	Estimated Repair Cost
concrete girder	> 1	$268,771,590
concrete girder	= 1	$18,500
steel girder	> 1	$12,117,650

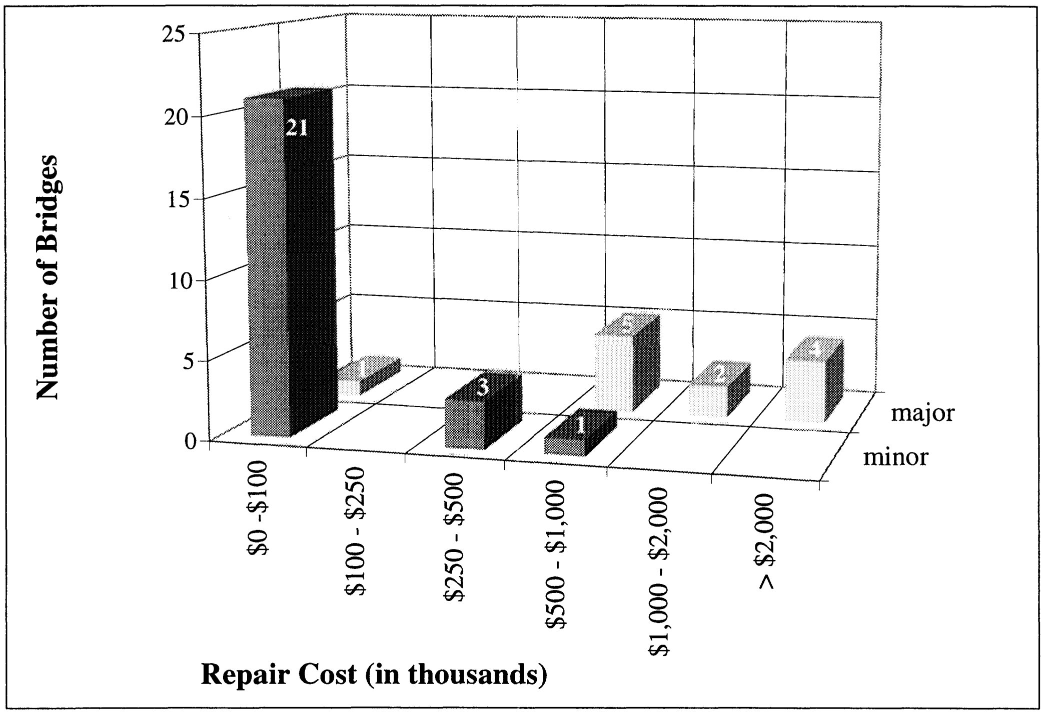


FIGURE 3-21 Distribution of All Damaged Bridges by Repair Cost and Damage State

Figure 3-22 shows the distribution of repair cost by bridge components for different damage states. The estimated repair cost for the Cypress Viaduct is not included in this figure since a detailed description of repair by component type was not available. According to figure 3-22, column damage was the costliest for bridges with *minor* damage. Damage to columns, joints and the deck contributed almost equally to repair cost for bridges with *major* damage. Reported repair cost due to abutment damage was minimal. As one might expect the repair cost for bridges that suffered *major* damage was larger than for those with *minor* damage. *Miscellaneous* repairs include repairs to railings, curbs, sidewalks and electrical conduits.

In order to investigate the distribution of repair cost by structural type, bridges were grouped by the bridge sub-categories given in table 2-1. Figure 3-23 shows the distribution of repair cost by these sub-categories. Only five of the sub-categories are included in this figure. Data were not available for the other sub-categories. The number of bridges in each bridge sub-category ranged from 2 to 4 as shown above each bar in figure 3-23. Note that the repair cost shown in this figure is only a small fraction of the total estimated repair cost. Therefore, the results should

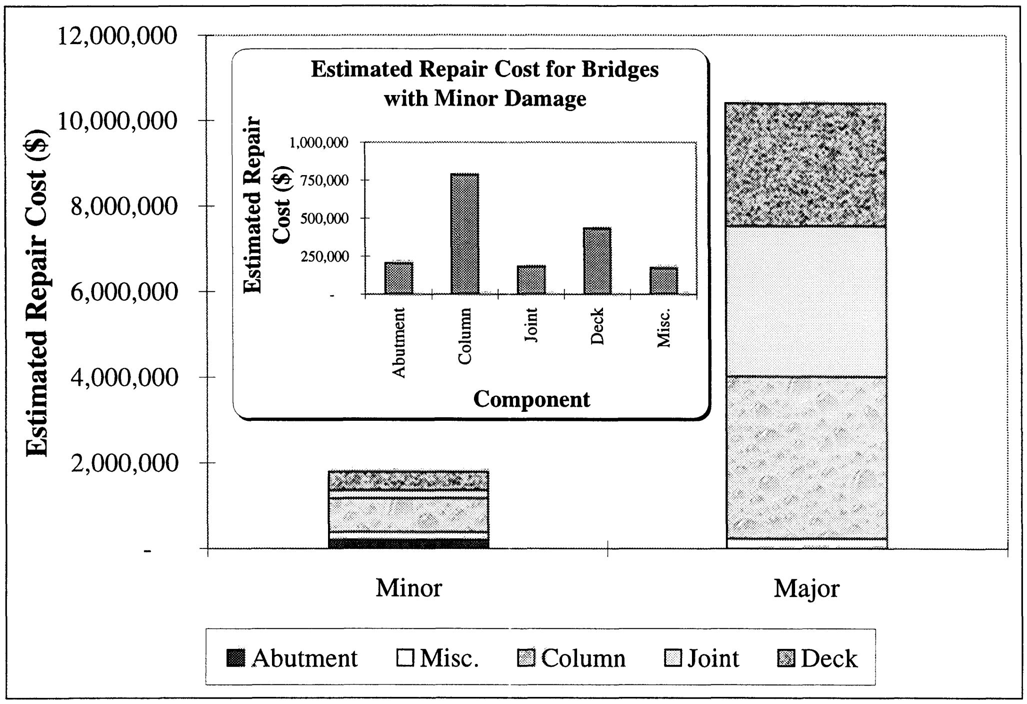


FIGURE 3-22 Distribution of Estimated Repair Cost by Bridge Components for Different Damage States

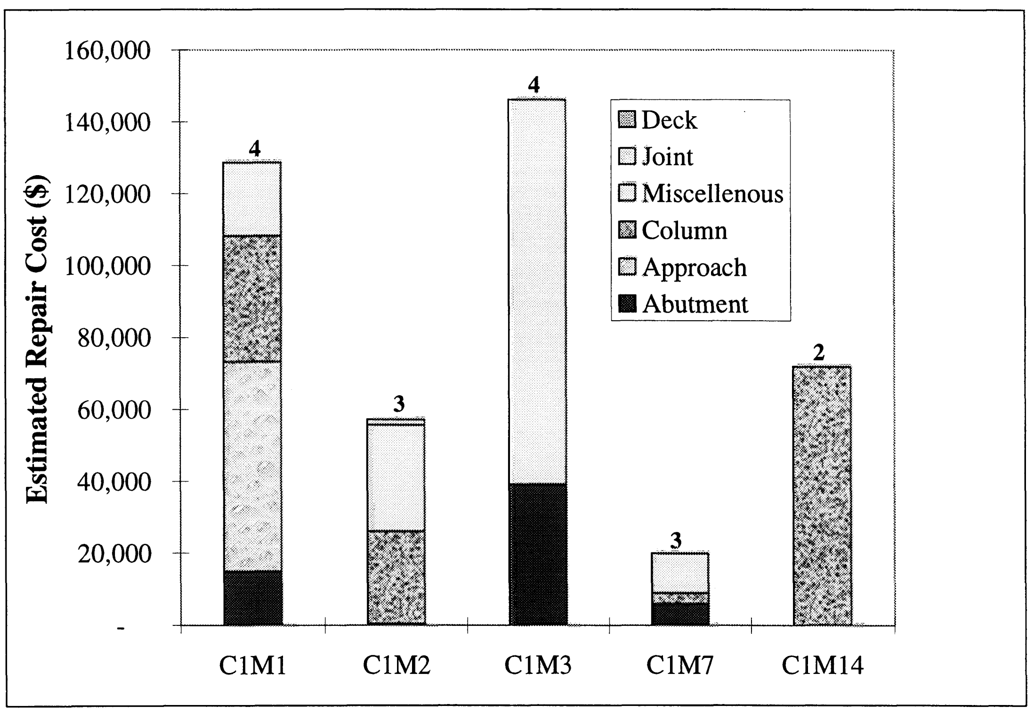


FIGURE 3-23 Distribution of Estimated Repair Cost by Bridge Components for Different Bridge Sub-categories

be interpreted with caution. Estimated repair cost due to traffic control was only available for 14 bridges; therefore it was not included in the analysis.

3.5.2 Correlation Studies on Repair Cost Ratio

Similar to ground motion-damage relationships, empirical ground motion-repair cost ratio relationships were developed. The repair cost ratio was defined as the ratio of repair cost to replacement cost. The replacement cost was obtained as the product of the unit cost per square feet ($90/ft^2) by bridge width and length. Seven repair cost ratio intervals were used: 0; 0-10%; 10-20%; 20-30%; 30-40%; 40-50%; >50%. The empirical fragility curves listed in table 3-11 are presented in Appendix D. Example fragility curves listed in table 3-12 are presented in figures 3-24 to 3-26. In each figure two graphs are presented: (*i*) ***GRAPH A*** shows the probability of being in or exceeding a repair cost ratio at different PGA levels, and (*ii*) ***GRAPH B*** shows the probability of being in a repair cost ratio at different PGA levels. The number of bridges in each repair cost ratio interval, i.e., 0; 0-10%; 10-20%; 20-30%; 30-40%; 40-50%; >50% and statistical significance test results are presented in these figures.

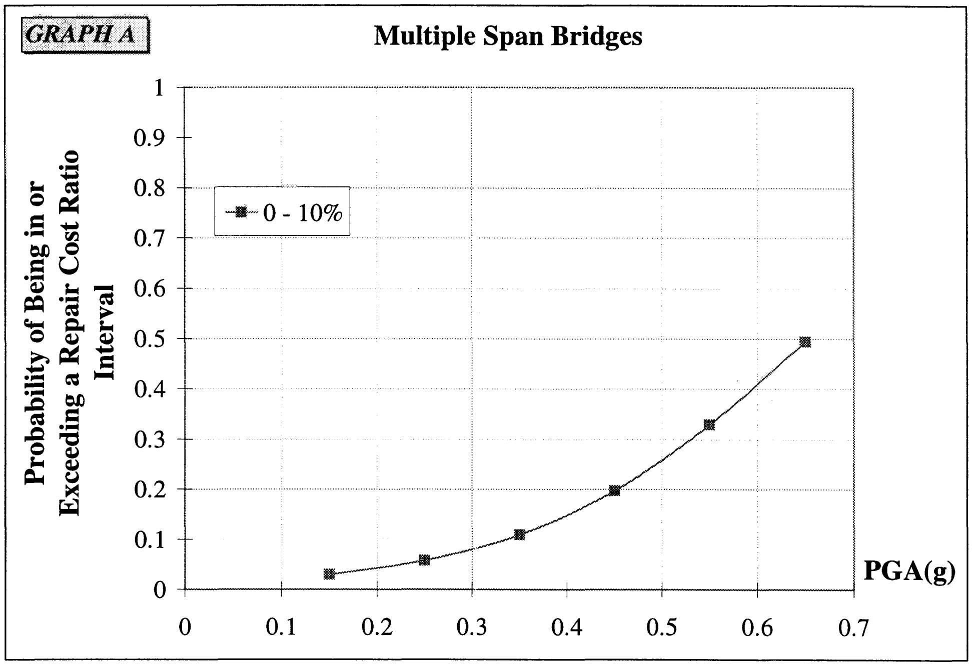
TABLE 3-11 Characteristics of Data Groups Used in Correlation Analyses

Dependent Variable	Independent Variable	Data Set Grouped by	Results are Presented in
repair cost ratio	estimated PGA values (Campbell's attenuation relationship [1991])	• number of spans • design year • bridge sub-categories given in table 2-1	Appendix D

TABLE 3-12 Description of Example Fragility Curves for Repair Cost Ratio

Dependent Variable	Independent Variable	Data Set Grouped by	Shown in figure
repair cost ratio	estimated PGA values (Campbell's attenuation relationship [1991])	• multiple span bridges	3-24
		• multiple span bridges with monolithic abutment type and multiple columns/bent (C1M1)	3-25
		• design year	3-26

Number of bridges	539, 27, 0, 0, 0, 0, 0		
Chi-square	N/A		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
6.9101	1.8833	0.0002	999



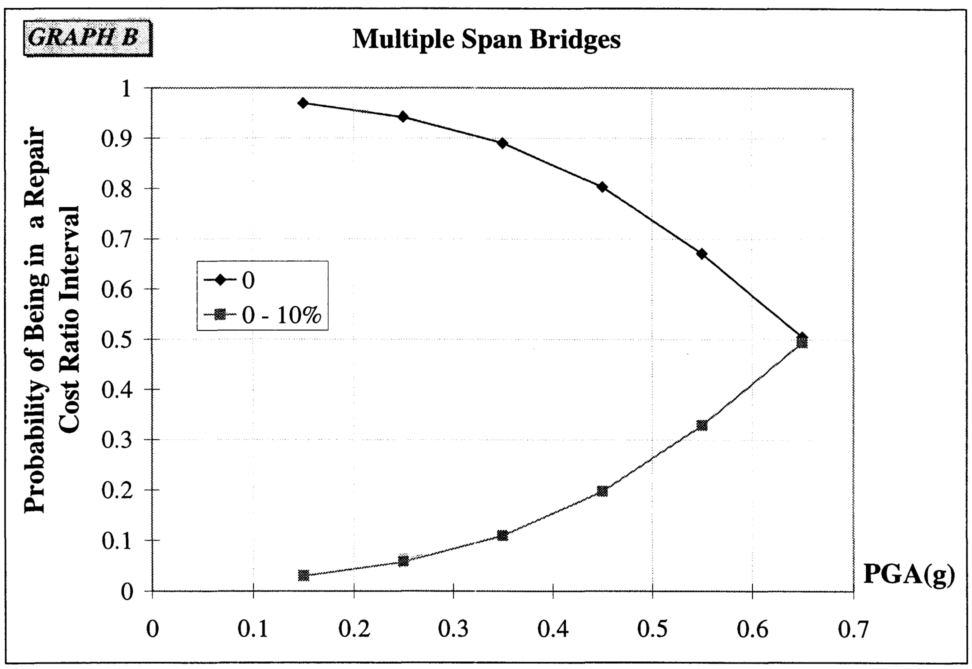
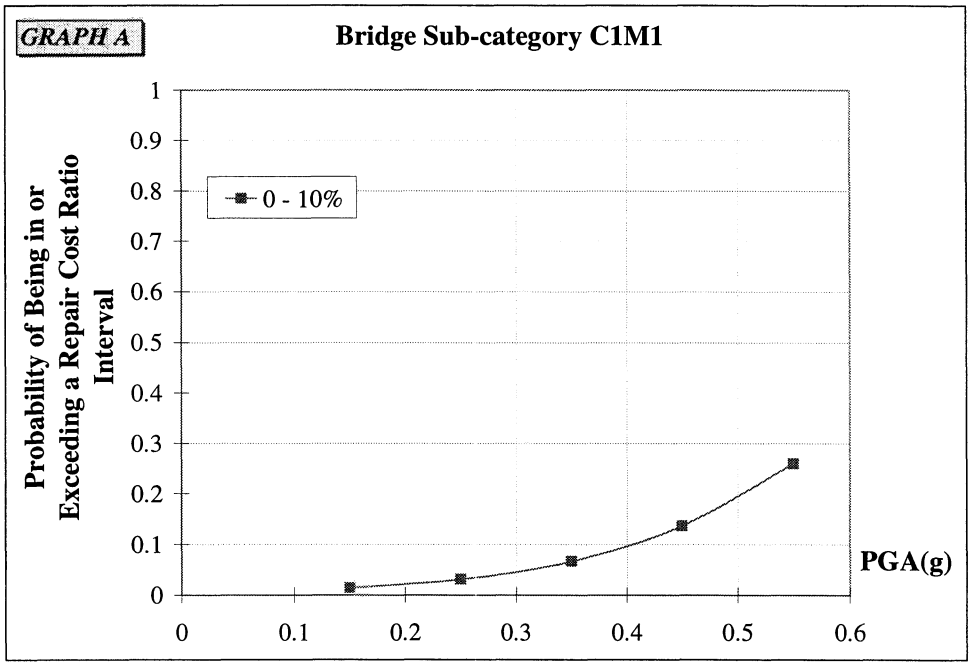


FIGURE 3-24 Empirical Fragility Curves for Multiple Span Bridges, Repair Cost Ratio

Number of bridges Chi-square	152, 4, 0, 0, 0, 0, 0 N/A		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
7.9759	4.0593	0.0494	999



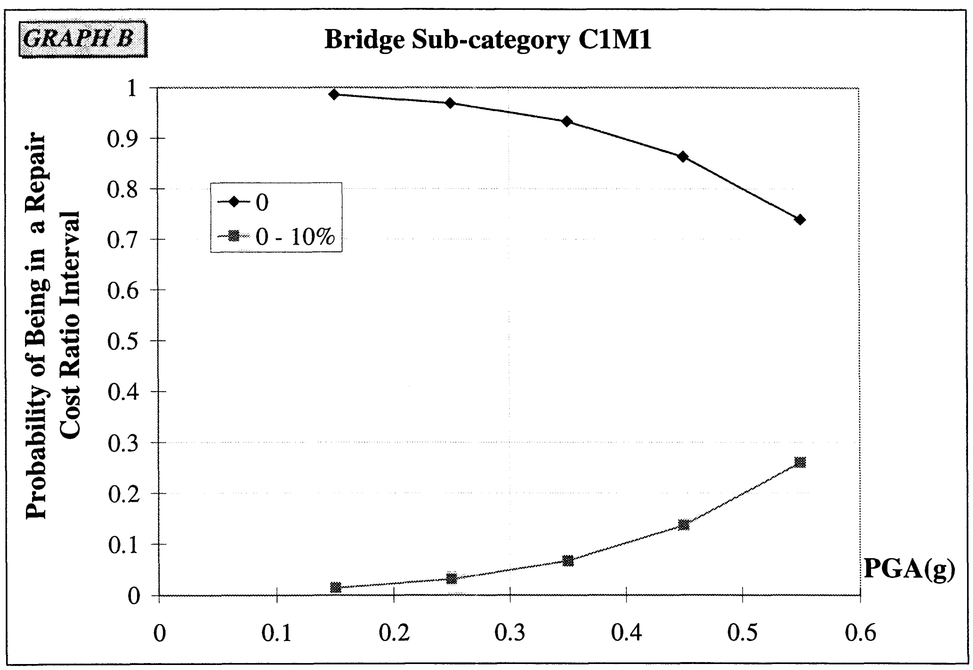
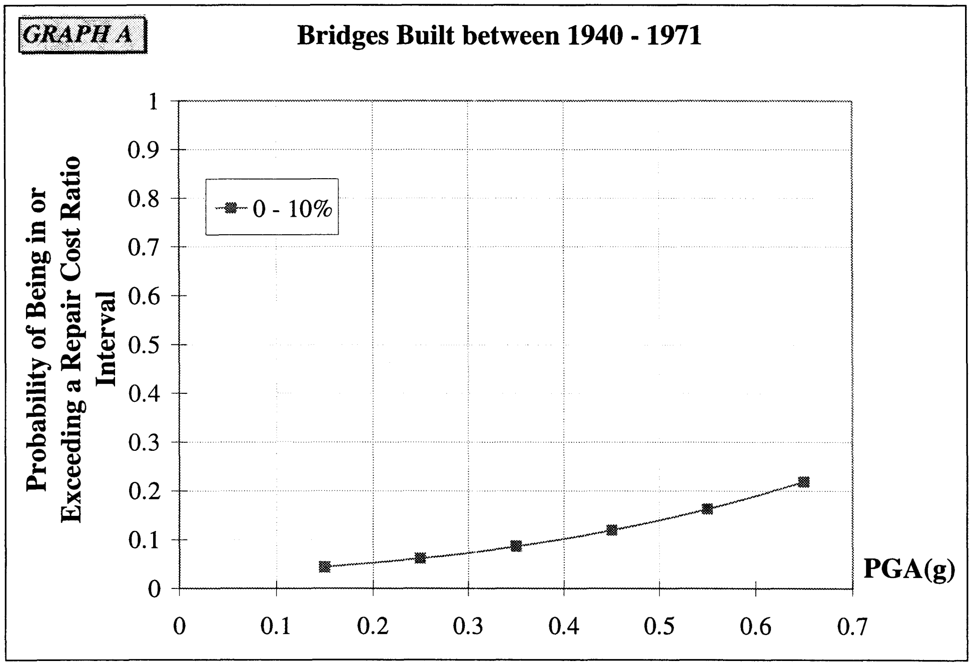


FIGURE 3-25 Empirical Fragility Curves for Multiple Span Bridges with Monolithic Type Abutments and Multiple Column Bents (Sub-category C1M1), Repair Cost Ratio

Number of bridges Chi-square	635, 35, 0, 0, 0, 0, 0 N/A		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
3.6167	1.6738	0.0307	37.216



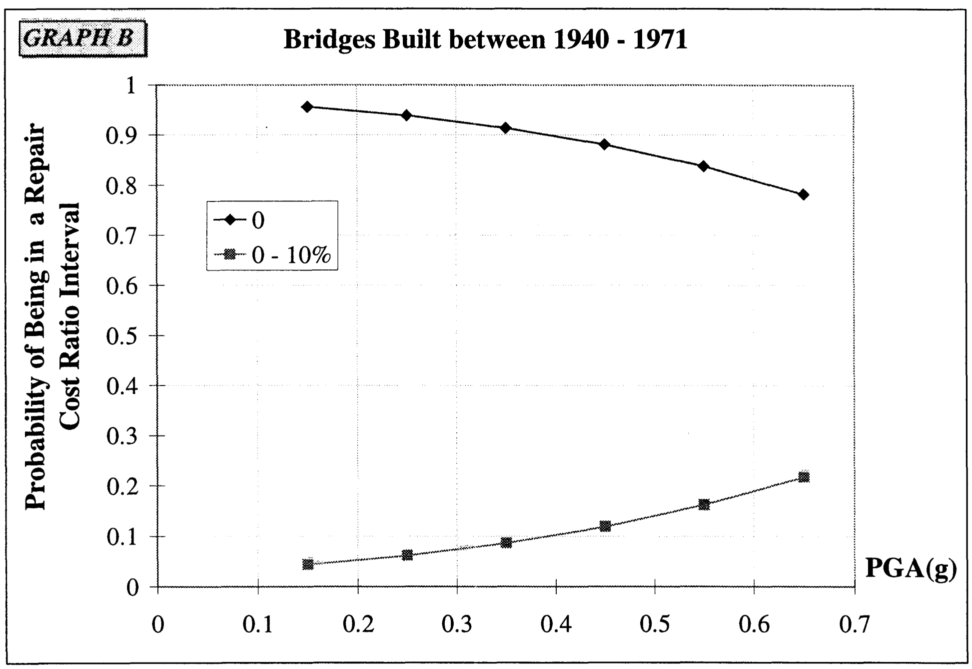


FIGURE 3-26 Empirical Fragility Curves for Bridges Built between 1940 and 1971, Repair Cost Ratio

3.6 Discussion

Based on the analysis of data presented in this section, several important observations can be made. These are summarized as follows:

Ground motion levels: Peak ground acceleration was used to characterize the ground shaking levels observed in the Loma Prieta earthquake. Due to lack of a map based on recorded ground motions, a ground motion map was generated based on a Loma Prieta scenario event. The surface ground shaking levels obtained in this study, however, do not agree well with the recorded ones at all locations. Figure 3-27 shows the discrepancy between the estimated and the recorded peak ground accelerations (denoted as PHA1 and PHA2 in figure 3-26). The effect of the variability in the estimated ground shaking levels (denoted as BJF and Campbell in figure 3-26) on the results obtained in the correlation studies should be investigated further.

The ground motion levels could only be obtained for a portion of the San Francisco Bay area (see figure 3-1), because information on the local geology was available only for that area. In order to use a data set with consistent characteristics in the logistic regression analysis, only bridges subjected to 0.10g or larger PGA levels were included in the analysis. This criterion for data selection was adapted instead of selecting bridges within a geographical boundary.

Damage states: The damage states that were reported during the post-earthquake bridge inspections were used in the analyses. Damaged bridges were grouped into two damage states: bridges with *minor damage* and bridges with *major damage*. These damage states covered a broad range of physical damage making it difficult to identify structural characteristics that contribute to damage. Different physical damage states were lumped together, although their consequences were not the same. For example, a bridge with minor cracks at columns and a bridge with shear key failure and abutment movement were both considered to sustain *minor damage*. However, the repair cost and the reduction of bridge functionality were higher for the bridge with shear key failures and abutment movement. In addition, a detailed investigation of the damage state definitions demonstrated the inconsistency and the subjectivity in bridge damage assessment among practicing engineers, and the need for a more systematic procedure to assess bridge damage.

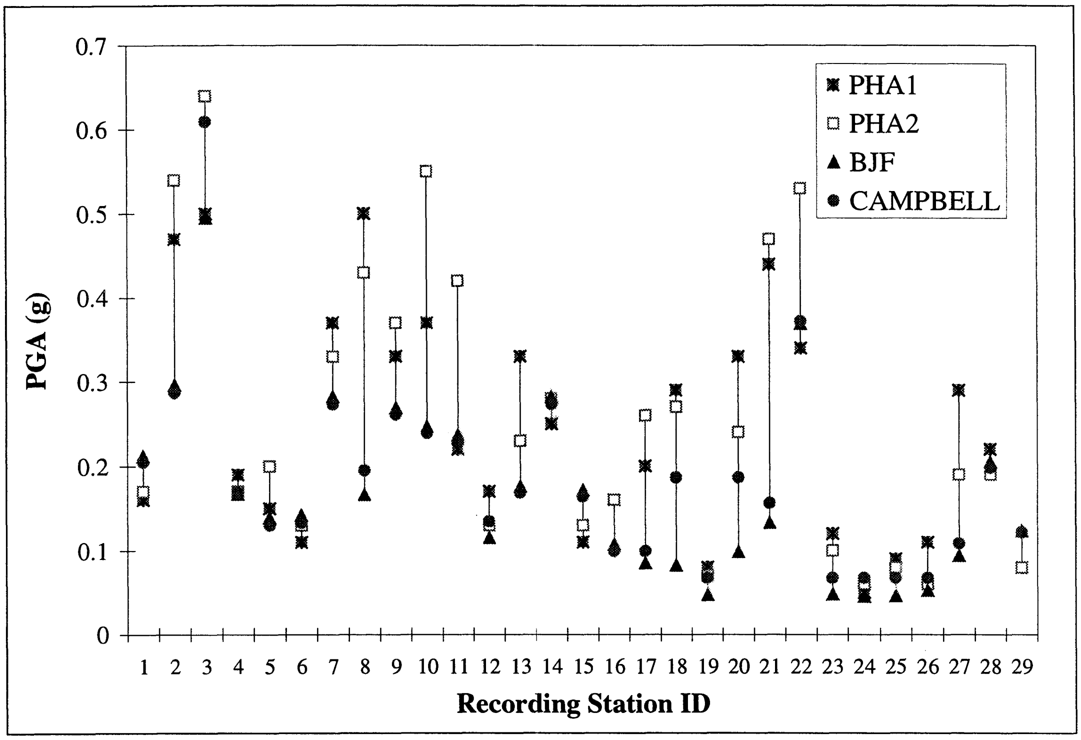


FIGURE 3-27 Recorded and Estimated PGA Levels at Sample Recording Stations - Loma Prieta Earthquake

Structural characteristics and damage: A total of 76 bridges were reported as damaged. Two of these bridges were railroad bridges. Fourteen of the damaged bridges had steel superstructure. Empirical fragility curves were developed for all steel bridges exposed to peak ground accelerations of 0.10g or higher and were compared to empirical fragility curves developed for all concrete bridges.

Figure 3-28 shows the empirical fragility curves for steel and concrete bridges. The largest PGA level experienced at the site of any steel bridge was 0.45g in comparison to 0.75g at the site of concrete bridges. The empirical fragility curves obtained for steel bridges, however, do not satisfy the statistical significance requirements for Wald chi-square test at the 0.05 significance level. Therefore the fragility curves for steel bridges obtained from the Loma Prieta data do not provide useful information.

Because the majority of the bridges in the San Francisco Bay area and the bridges damaged in the Loma Prieta earthquake were concrete bridges, analyses were carried out mainly for concrete highway bridges. Concrete highway bridges with homogeneous characteristics were used in the

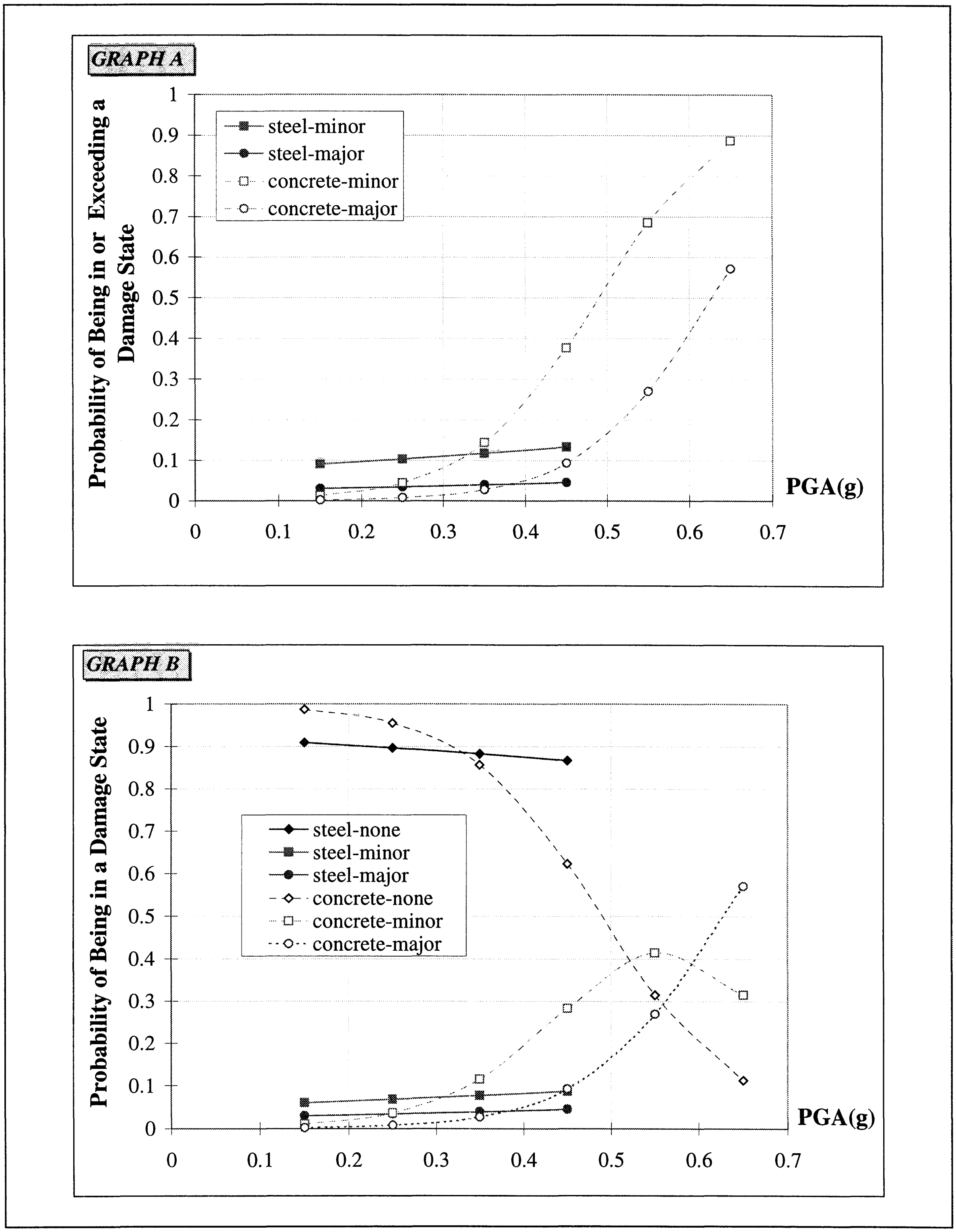


FIGURE 3-28 Empirical Fragility Curves for Steel and Concrete Bridges

correlation studies. This criterion reduced the data set for damaged concrete bridges, however, it provided a database with homogeneous bridge characteristics. The effects of different abutment and column bent types on bridge damage susceptibility were determined based on this data set.

The majority of the damaged bridges were designed according to pre-1971 standards. For analysis purposes, four design year periods were used: before 1940, between 1940 and 1971, between 1972 and 1980, and after 1980. Based on the results from logistic regression analysis, the performance of bridges was best for bridges built after 1980.

Bridges in the *correlation data set* were grouped into three categories according to their abutment types: (*i*) monolithic, (*ii*) non-monolithic, and (*iii*) partial abutment types, i.e., bin or cellular closure type abutments. Figure 3-29 shows empirical fragility curves for multiple span bridges with monolithic, non-monolithic and partial abutment types. As shown in this figure, bridges with monolithic abutments were observed to perform better than those with non-monolithic abutments, while bridges with partial type abutments performed better than the bridges with monolithic abutments at higher PGA levels.

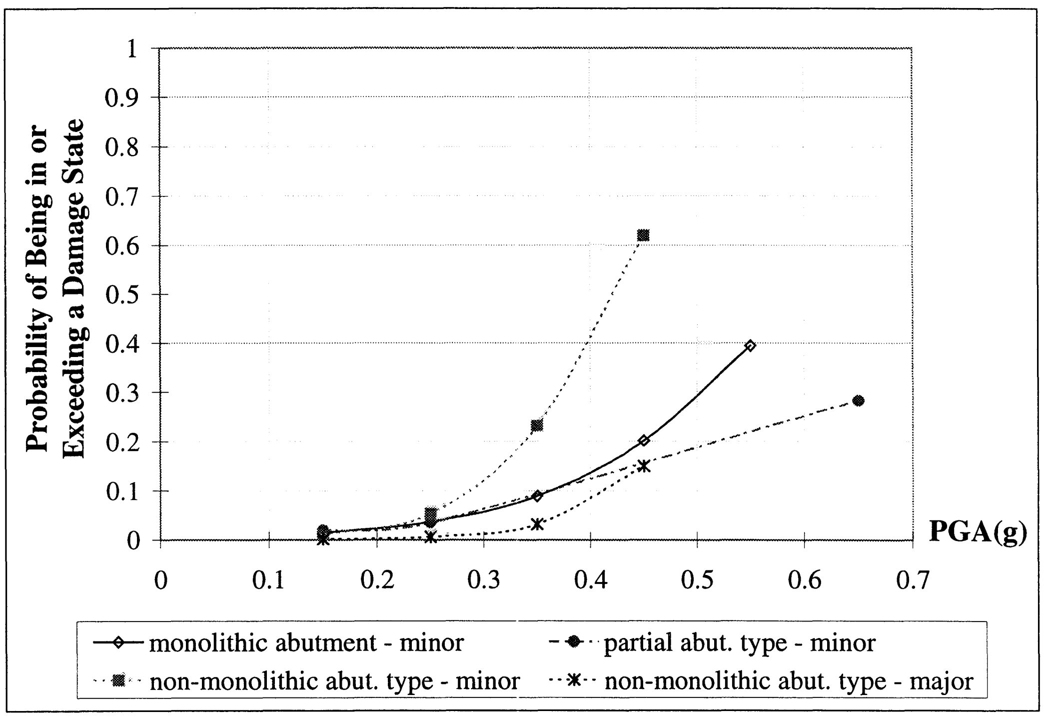


FIGURE 3-29 Performance Comparison of Different Types of Abutments

Figure 3-30 shows a comparison of probabilities of being in or exceeding different damage states for bridges with multiple column bents, single column bents and pier walls. Bridges with single column bents performed worse than those with multiple column bents and pier walls. Two of the bridges with multiple column bents experienced *major* damage, and the damage was concentrated at the outrigger knee bent joints.

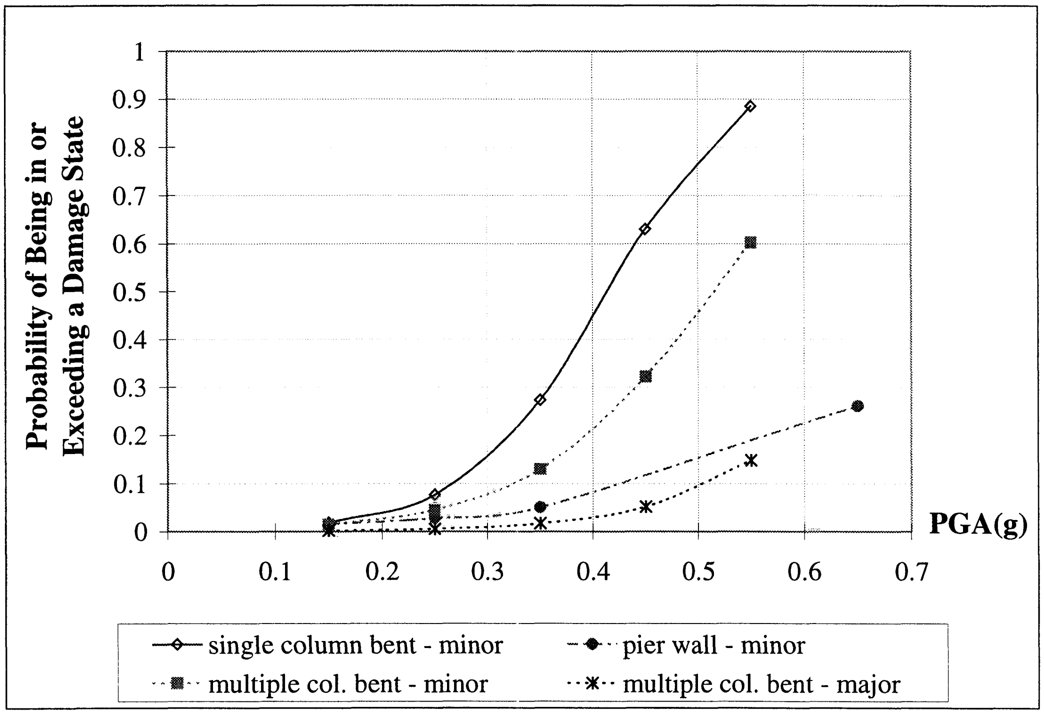


FIGURE 3-30 Performance Comparison of Different Types of Column Bents

Bridge classification and ground motion-damage relationships: In order to explore the effect of structural characteristics, such as number of spans, abutment type, column bent type and span continuity, the bridges were grouped into twenty-one sub-categories, listed in table 2-1. Basöz and Kiremidjian [1996] proposed bridge classes based on these structural characteristics. In that study, the authors defined the least and the most vulnerable sub-categories of a given bridge class for single and multiple spans. These four sub-categories for concrete bridges were included in the sub-categories used in this study.

There were no damaged bridges in the least vulnerable sub-category for single span bridges, i.e., single span bridges with monolithic abutments. Single span bridges with non-monolithic

There were no damaged bridges in the least vulnerable sub-category for single span bridges, i.e., single span bridges with monolithic abutments. Single span bridges with non-monolithic abutment types, in contrast, sustained both *minor* and *major damage.* Figure 3-31 shows fragility curves for multiple span bridge sub-categories C1M1 and C1M7. The sub-category C1M1 represents bridges with monolithic type abutment, multiple column bents, continuous span and is the least vulnerable sub-category. There was not sufficient data on bridges with non-monolithic type abutments, discontinuous span and single column bents (bridge sub-category C1M10). Bridges with non-monolithic abutment, multiple column bents and continuous spans (bridge sub-category C1M7) are expected to be less vulnerable than bridges included in sub-category C1M10 but more vulnerable than those included in sub-category C1M1. As shown in figure 3-31, the probability of being in or exceeding a given damage state was higher for bridges in sub-category C1M7 than it was for C1M1 for *minor damage* and probability of exceeding *major damage* was non-zero only for C1M7.

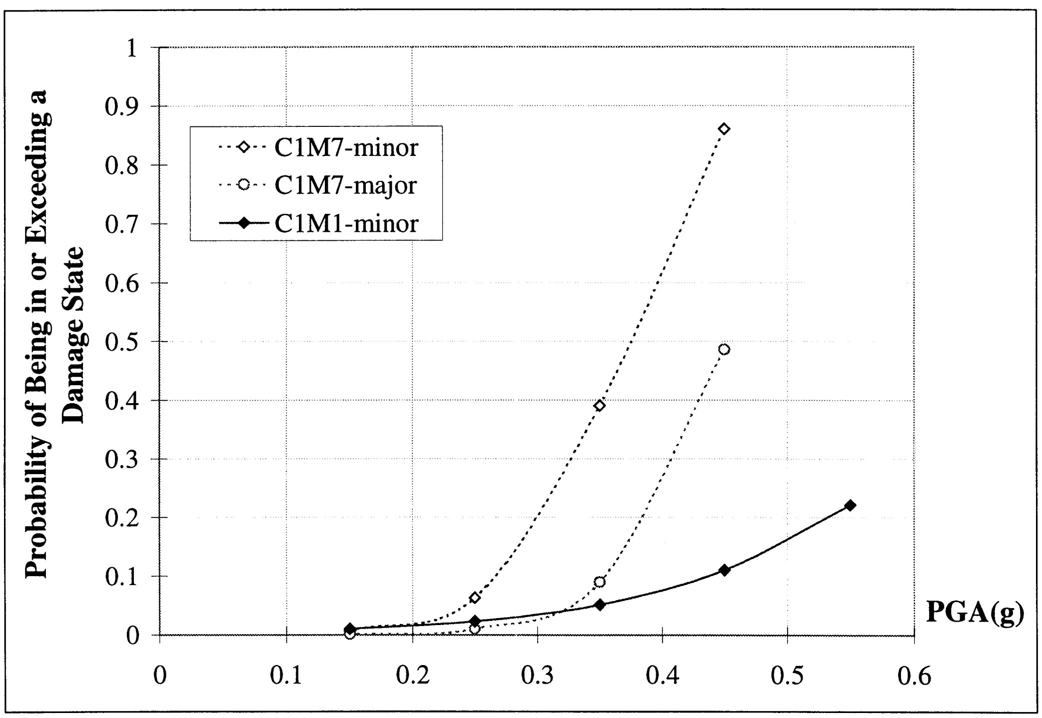


FIGURE 3-31 Empirical Fragility Curves for Sub-categories C1M1 and C1M7

In addition to the bridge classes defined in this study, the bridges were grouped according to HAZUS [1997] bridge classes. The observed damage data do not appear to agree well with the available ground motion-damage relationships for most of the bridge classes. The poor agreement between the observed and the predicted damage can be partially attributed to the size of the data set. In general, it is also expected that damage data from a single earthquake will not agree precisely with the heuristic fragility curves or DPMs since these functions are intended to represent average values over many earthquakes. A more detailed analysis of the damage data, however, indicated potential problems with the structural characteristics considered in the available ground motion-damage relationships. The bridge classes currently used in practice do not consider the effect of the structural material and type, substructure type, and design details, such as column reinforcement and/or seat width. The HAZUS classification identifies bridges with older design characteristics (designed before 1960), superstructure irregularity or span discontinuity as high risk. However, a bridge with any of these characteristics is assumed to be high risk. For example, a continuous bridge with no skew constructed in late 1950s is assumed to be as vulnerable as a bridge constructed in late 1960s with discontinuous span and high skews. Since both bridges will be classified in the same bridge class according to HAZUS classification, damage estimates and fragility curves used in damage analyses for either of these bridges will yield the same results, even though the bridges may perform in a completely different manner.

<u>Estimated repair cost:</u> Similar to damage states, several statistical analyses were carried out using the estimated repair cost reported for the damaged bridges. For some of the bridges, no repair cost was reported. Likewise no damage state was assigned to some bridges for which repair cost estimates were reported. Repair cost was reported by structural components for some of the bridges. Among these bridges with complete data on component-based repair cost data, about 50 percent of the repair cost was due to column damage, about 21 and 18 percent of the damage was due to deck and joint damage, respectively. Estimated repair cost due to abutment damage was only 2 percent. It should be noted that these percentages were based on the estimated repair cost only for bridges with repair cost information reported by structural components.

The component repair costs were also grouped by bridge class sub-categories, however, the available data were too limited to draw general conclusions.

Similar to damage states, empirical relationships between the repair cost ratio and ground motion levels were developed. The maximum observed repair cost ratio was 14 percent. However, the repair cost ratio-ground motion relationships did not include the Cypress Viaduct since it was not included in the *correlation data set.*

As discussed in this section, several factors made it difficult to derive general conclusions from the analyses of the bridge damage and repair cost data from the Loma Prieta earthquake. These include quality and the size of available data sets and broad definitions for the damage states. Data on bridge damage from the Northridge earthquake, discussed in the next section, provided the means to substantiate and elaborate on the analysis results obtained in this section. In addition to the analyses presented in this section, analyses of data with respect to retrofit history and reliability of the data set were conducted using the data from the Northridge earthquake. Summary and comparison of results from the Loma Prieta and Northridge earthquakes are presented in Section 5.

SECTION 4

BRIDGE DAMAGE DATA FROM THE NORTHRIDGE, CA EARTHQUAKE

The January 17, 1994 Northridge, California earthquake caused serious damage to bridges in the region, resulting in disruptions to the transportation system. The earthquake was of moderate size (a moment magnitude of 6.7) and occurred at 4:31 am local time. These two factors contributed to the low number of casualties. The impact of bridge closures on emergency response activities immediately after the earthquake was negligible. Closure of the damaged bridges, however, caused major rerouting of traffic during the months following the earthquake. Despite the high ground motion levels observed in the 1994 Northridge earthquake, only about three percent of all the bridges in the area experienced major damage.

The Northridge earthquake provided valuable information to study damage to bridges. In this study, the data from the earthquake were analyzed to correlate the observed damage to ground motion levels, structural characteristics, and repair costs. Statistical analyses were performed for the inventory of bridges impacted in the Northridge earthquake. In addition, empirical damage probability matrices and fragility curves were developed based on observed bridge damage. The correlation between structural characteristics and observed damage was investigated. In this section, statistics on structural characteristics of bridges affected in the earthquake, ground shaking levels at bridge sites, bridge damage characteristics and estimated repair costs are presented. Findings of the correlation studies for bridges impacted in the Northridge earthquake are also discussed.

4.1 Inventory of Bridges Affected by the Northridge Earthquake

The bridges in the Greater Los Angeles area, including Los Angeles, Ventura, Riverside, and Orange Counties, were exposed to ground shaking during the 1994 Northridge earthquake. Table 4-1 lists the number of state and local bridges, and the number of damaged state bridges in each of the four counties. A database of state and local bridges for these four counties was extracted from the Bridge Maintenance Database compiled by Caltrans [1993]. The structural characteristics included in this database were structural type and material, number of spans, abutment type, span continuity, substructure type, skew, foundation type and design year indicating the seat width and column longitudinal reinforcement.

As shown in table 4-1, there are 3,533 state and 2,571 local bridges that are located in the four counties. Of the 3,533 state bridges, 3,318 (1,902 bridges in Los Angeles County, 312 in Ventura County, 462 in Orange County and 642 in Riverside County) carry highway traffic[1] and are grouped in the *highway bridge data set*. The number of bridges in the *highway bridge data set* listed by superstructure type and substructure material is shown in table 4-2. Figure 4-1 shows the distribution of these bridges by design year. Seventy six percent of the bridges in the *highway bridge data set* were designed according to pre-1971 design standards.

TABLE 4-1 Distribution of State and Local Bridges and the Number of Damaged State Bridges in the Greater Los Angeles Area

County	Number of State Bridges	Number of Local Bridges	Total Number of Bridges	Number of Damaged Bridges
Los Angeles	2,097	1,553	3,650	228
Riverside	644	338	982	-
Orange	463	505	968	-
Ventura	329	175	504	5
Total	*3,533*	*2,571*	*6,104*	*233*

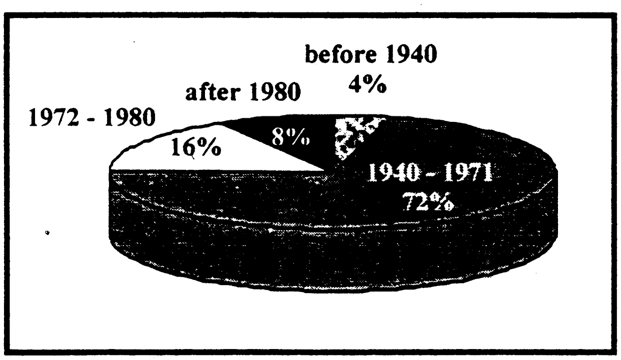


FIGURE 4-1 Distribution of State Highway Bridges in Los Angeles, Ventura, Orange and Riverside Counties by Design Year

[1] Structures identified with the descriptions given in Appendix B are excluded from the analyses. These descriptions are extracted from the Caltrans OSM&I Guidelines.

TABLE 4-2 Distribution of State Highway Bridges in Los Angeles, Ventura, Orange and Riverside Counties by Superstructure Type and Substructure Material Type

Substructure type		Superstructure Type and Material						
		Concrete	**Steel**	**Truss**	**Timber**	**Arch**	**Suspension**	**Unknown**
	Concrete	2,394	119	4	0	12	1	4
	Steel	9[1]	3	0	1	0	0	0
	Timber	0	0	0	4	0	0	0
	Single Span Bridges	708	32	0	0	15	0	6

The majority of the bridges in the four counties listed in table 4-2 are concrete structures. Furthermore, more than 85 percent of the bridges damaged in the Northridge earthquake were concrete structures. Of the 3,318 state bridges that carry traffic 3,102 bridges (with concrete superstructure and concrete substructure for multiple span bridges and with concrete superstructure for single span bridges) are grouped into the *concrete highway bridge data set.* Examples of concrete superstructure include concrete box gider, concrete girder, precast concret girders, cast-in place prestressed slab, and precast prestressed box girder.

In order to study the effect of structural component types on bridge damage, such as effect of abutment type (monolithic or non-monolithic) and number of columns per bent, bridges with heterogeneous characteristics were excluded from the *homogeneous data set.* Bridges with incomplete information were also excluded from this data set. Furthermore, to attain a complete data set for the correlation studies, only bridges that were exposed to peak ground acceleration (PGA) of 0.15g or higher were compiled in the *correlation data set.* The minimum PGA level was selected as 0.15g since it is not likely that bridges outside these four counties experienced PGA levels larger than 0.15g. Therefore, all bridges subjected to PGA levels of 0.15g or larger are expected to be included in this data set. Moreover, only four of the damaged bridges were exposed to PGA levels less than 0.15g. Table 4-3 gives a summary of number of bridges included in various data sets used in this study. The distribution of single and multiple span bridges in each data set is also given in table 4-3. The statistics for *highway bridge data set* and the *correlation data set* are presented in this sub-section.

---

[1] These 9 bridges have concrete slab type superstructure and have both concrete and steel column bents.

TABLE 4-3 Number of Bridges Included in Various Data Sets

		Highway Bridges Data Set	Concrete Highway Bridges Data Set	Homogeneous Data Set	Correlation Data Set
Los Angeles County	*single span*	490	458	414	291
	multiple span	1,412	1,321	1,109	700
Ventura County	*single span*	80	72	61	32
	multiple span	232	209	189	100
Orange County	*single span*	69	62	54	7
	multiple span	394	362	306	38
Riverside County	*single span*	129	116	101	3
	multiple span	512	502	417	10
Total		3,318	3,102	2,651	1,181

Figure 4-2 shows distribution of the bridges in the *correlation data set* by design year. A comparison of figures 4-1 and 4-2 shows that the *correlation data set* is a representative sample for the design year attribute. Figure 4-3 shows the distribution of single span bridges in the *correlation data set* by abutment type. The distribution of multiple span bridges by abutment type and column bent type are depicted in figures 4-4.a and 4-4.b, respectively. Monolithic abutment types were common for both single and multiple span bridges. Number of bridges with multiple column bents was about four times the number of bridges with single column bents.

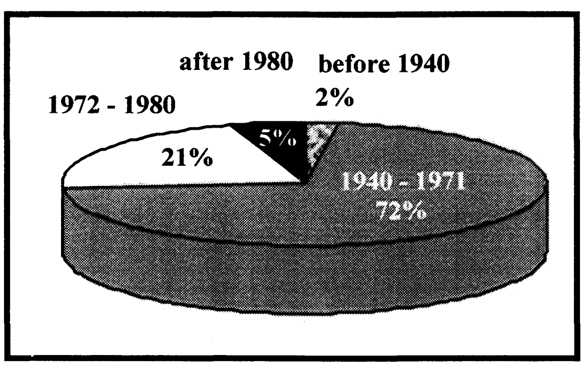


FIGURE 4-2 Distribution of Bridges by Design Year (*correlation data set*)

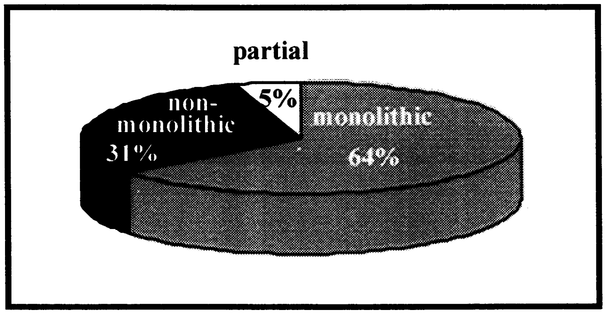


FIGURE 4-3 Distribution of Single Span Bridges by Abutment Type (*correlation data set*)[1]

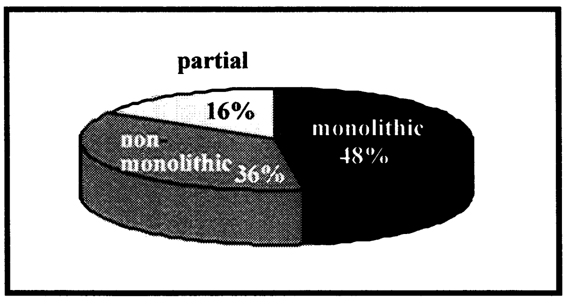


FIGURE 4-4.a Distribution of Multiple Span Bridges by Abutment Type (*correlation data set*)

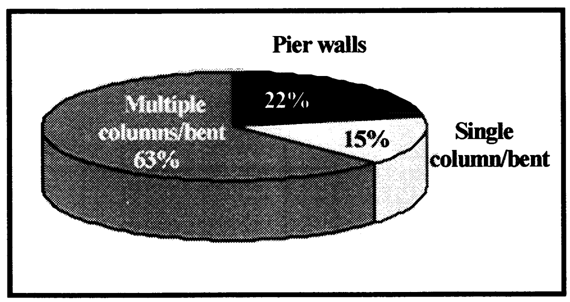


FIGURE 4-4.b Distribution of Multiple Span Bridges by Column Bent Type[2] (*correlation data set*)

Next, the *correlation data set* (i.e., 1,181 concrete state highway bridges with homogeneous abutment and column bent types that were exposed to 0.15g or higher PGA levels) were classified into bridge sub-categories listed in table 2-1. Figure 4-5 shows the distribution of bridges in the *correlation data set* by these bridge sub-categories. The results from the correlation studies based on this bridge classification are presented in Section 4.4.

---

[1] See table 2-2 for description of abutment types.

[2] Frame bents and pile bents are grouped as multiple column/bent.

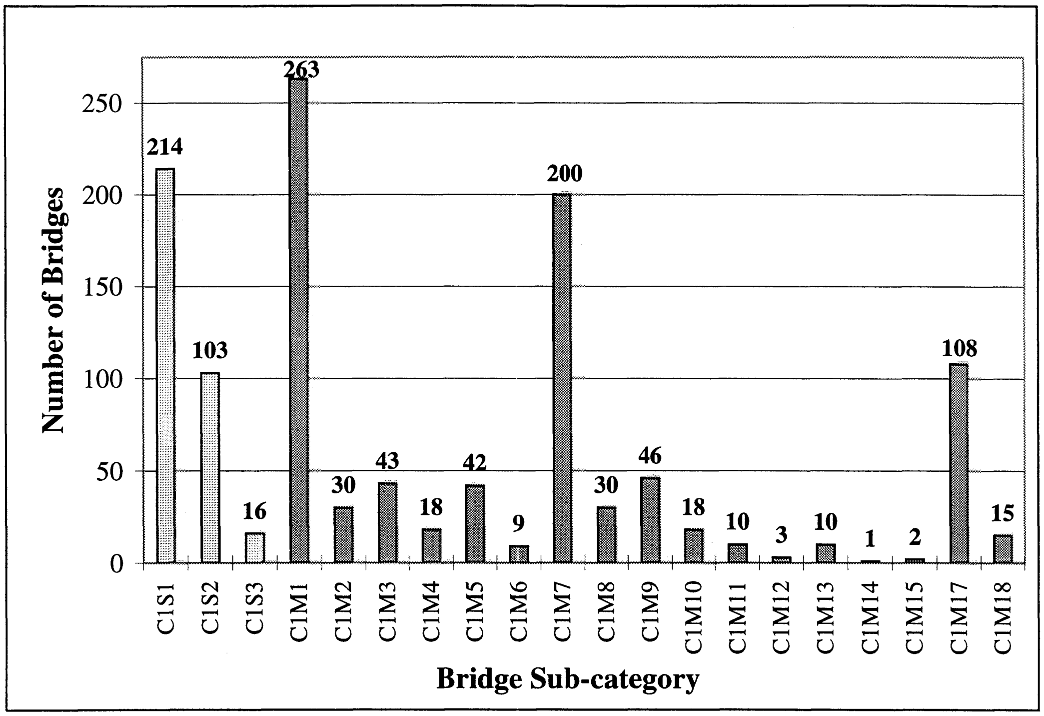


FIGURE 4-5 Distribution of Bridges by Bridge Sub-category (*correlation data set*)

4.2 GROUND-MOTION AT BRIDGE SITES

In order to obtain empirical ground motion-damage relationships for the bridges affected by the Northridge earthquake, two sets of PGA values were used:

(i) PGA values reported by USGS, which were obtained from the contours of observed PGA recordings in horizontal direction [USGS, 1994],

(ii) PGA values reported by WCFS [1995] that were obtained from the contours of average of the PGA values measured in the E-W and N-S directions. Recorded ground motion levels were used to scale the empirical Green's functions which were used in simulating the ground shaking levels [Somerville et al., 1996].

The PGA value at a given bridge site was obtained within a geographic information system (GIS) by overlaying the ground shaking map and the bridge location map. Subsequently, the highest PGA values obtained at a bridge site were 1.55g and 0.66g for the USGS [1994] and WCFS [1994] maps, respectively. Since the PGA levels from the two data sets varied considerably, the correlation studies were performed for both data sets. Figures 4-6 and 4-7 show the recorded

PGA values reported by USGS [1994] and WCFS [1995], and the locations of state highway bridges of the Los Angeles, Ventura, Orange and Riverside Counties. From these maps it can be observed that most of the bridges in Riverside and Orange Counties did not experience ground shaking levels higher than 0.15g in the Northridge earthquake.

4.3 COMPILATION AND REVIEW OF BRIDGE DAMAGE DATA

4.3.1 Damage State Descriptions

In various damage reports, bridges were reported to be in one of the four damage states: *minor*, *moderate*, *major* and *collapse*. In this study, any bridge that was not reported as damaged was assumed to have sustained no damage. *Major* damage generally referred to cases where column spalling and rebar buckling extended over a length of one column diameter or more, or to cases where severe hinge damage and near unseating occurred. *Moderate* damage referred to cases where column spalling or shear cracking occurred without buckling, or where abutment/pier damage was substantial. *Minor* damage was used for bridges with no danger of imminent structural collapse or easily repairable damage [EERI, 1995].

The subjectivity in damage evaluation leads to inconsistencies in bridge damage reports by different bridge engineers. For example following the Northridge earthquake, different bridge engineers reported the same bridges with *minor* and *major* damage during different inspections. In this study, detailed damage descriptions were obtained mainly from bridge damage reports compiled by Caltrans [1994b]. These descriptions were cross-referenced with those provided by Buckle [1994], EERI [1995], and Yashinsky [1995]. Judgment was used to treat inconsistencies in the interpretation of the observed damage data. Table 4-4 shows detailed damage descriptions for a sample of damaged bridges as compiled in this study. As discussed in Section 2.1, damage state descriptions were developed based on Northridge damage and were proposed for post-earthquake bridge damage assessment.

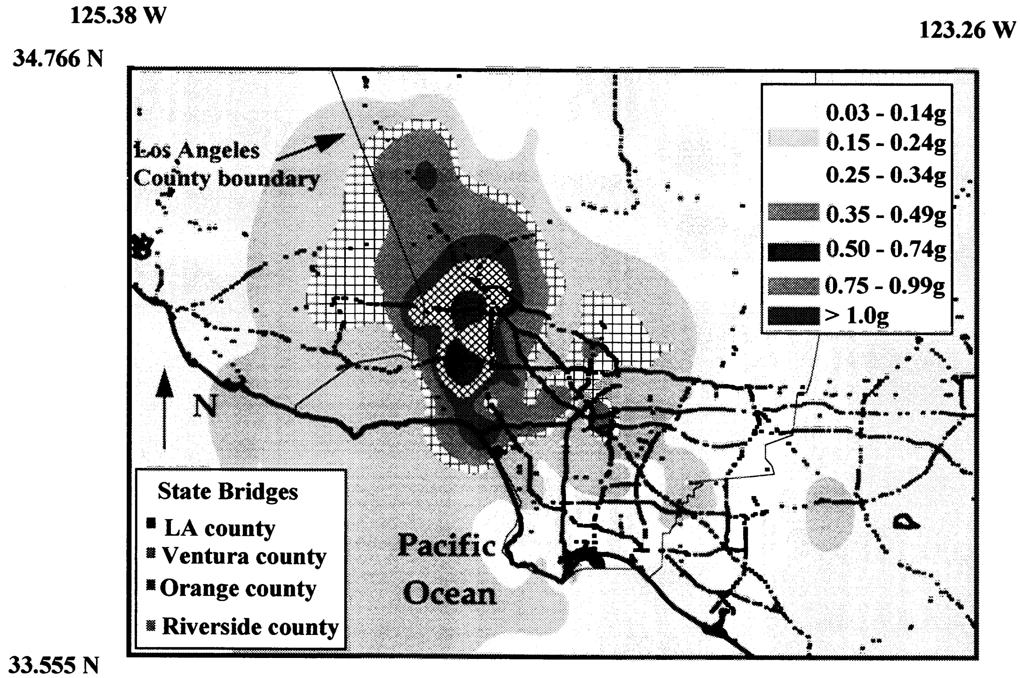


FIGURE 4-6 Locations of the State Highway Bridges in the Greater Los Angeles Area and the PGA Levels Observed from the Northridge Earthquake [USGS, 1994]

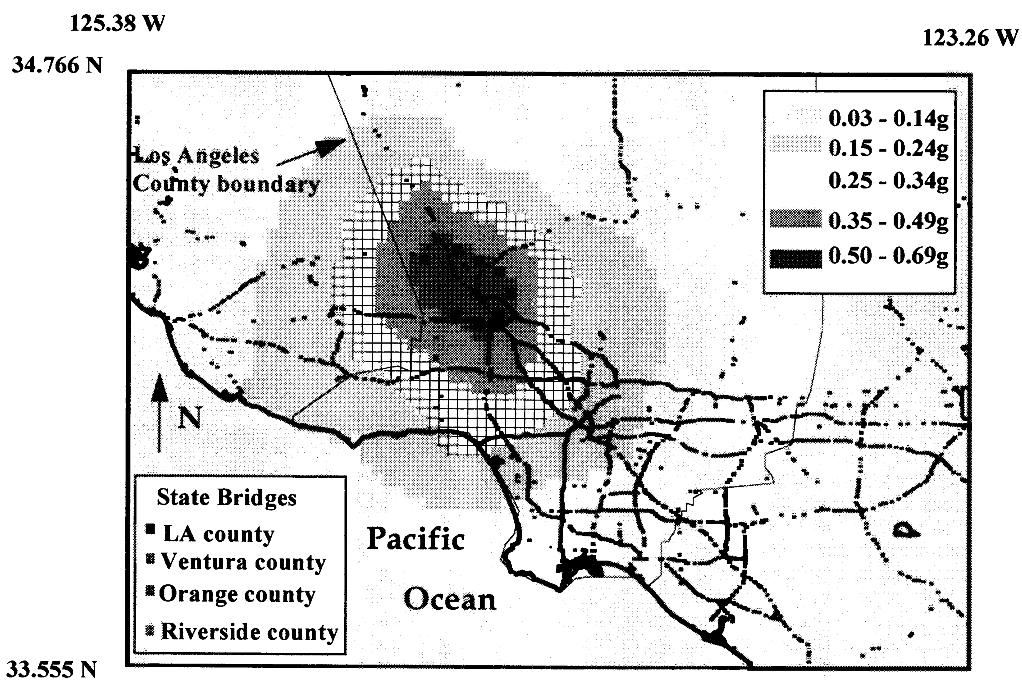


FIGURE 4-7 Locations of the State Highway Bridges in the Greater Los Angeles Area and the PGA Levels Observed from the Northridge Earthquake [WCFS, 1995]

TABLE 4-4 A Sample from the List of Damaged Highway Bridges in the Northridge Earthquake

ROUTE	POSTMILE	BRIDGE NO.	DAMAGE DESCRIPTION	DAMAGE	BRIDGE NAME
10	8.83	53 1609	Most of the columns of mainline structure buckled (high skew contributed). Replaced.	Collapse	LA CIENEGA-VENICE SEP
118	8.63	53 2205	Partial collapse. Approximately 2' sag. 2 columns buckled at bent 1.	Collapse	MISSION GOTHIC
			1 column buckled at bent 2. Replaced.		
14	24.73	53 1960F	2 spans collapsed onto Rte. 5. Need shoring for 1 hinge H=50'. Replaced.	Collapse	RTE 14/5 SEP SB14 TO SB5/5 SEP
14	24.97	53 1964F	Span drop from abutment 1 to bent 3. Replaced.	Collapse	NORTH CONNECTOR 14W TO 5N
5	47.83	53 1797L	Collapse of spans 2 and 4 due to heavy skew/narrow hinge seats.	Collapse	GAVIN CANYON UC
5	47.83	53 1797R	Collapse of spans 2 and 4 due to heavy skew/narrow hinge seats.	Collapse	GAVIN CANYON UC
5	45.52	53 1960G	Severe hinge and abutment damage. Replaced.	Major	RTE 14/5
5	52.42	53 1815	Abutment 1 displaced approximately 6" left of original position.	Major	VALENCIA BLVD
10	4.51	53 1604	*	Major	BUNDY DR UC
10	4.89	53 1620	*	Major	GATEWAY BLVD UC
10	5.43	53 1628	*	Major	RTE 10/405 SEP
10	5.56	53 1623	*	Major	SEPULVEDA BLVD UC
10	5.76	53 1640	*	Major	MILITARY AVE UC
10	8.83	53 1609S	Column damage. Replaced.	Major	LA CIENEGA-VENICE SEP
14	31.88	53 2027L	Possible damage to restrainers and bearing pad.	Major	SANTA CLARA RIVER
118	9.70	53 2210G	Spalls at abutments. Cracks in wingwalls.	Major	CHATSWORTH
405	41.86	53 1408	*	Major	VANOWEN ST UC
10	5.28	53 1627G	Bent 3 is 2" offset of span 3 towards bent 4. Minor spalls. Restrainer damage.	Major	NORTHWEST CONNECTOR OC (E10/N405)
10	6.31	53 1615	Sheared keeper plate bolts. Spalls at columns and rails. Cracks in curtain walls.	Major	NATIONALBLVD OC
10	9.31	53 1580	Bent 3 and bent 5 columns buckled causing spans 2 and 4 to drop. Replaced.	Major	FAIRFAX WASHINGTON UC
101	11.60	53 1493S	Minor spalls at columns. Cracks in curtain wall of abutment 1. Joint seal failure.	Major	RIVERSIDE DRIVE OC
101	11.63	53 1339F	Top of column damage at all bents. Bearing and approach slab damage.	Major	ROUTE 134/101, 170 SEP
101	11.75	53 1336R	Columns in bents 4, 6 and 8 are cracked and spalled at top. Shear key failure.	Major	ROUTE 101/134/170 SEPARATION
101	15.38	53 1371	Sheared keeper bolts and spalls at abutment 6. Spalls at tops of columns.	Major	LOS ANGELES RIVER
101	31.05	53 1442	Vertical offset at abutments 1 and 5. Possible pile damage.	Major	LAS VIRGENES OC
118	11.41	53 2328G	Abutment offset. Column movement (soil displacement). Barrier rail damage.	Major	PACOIMA WASH
118	11.42	53 2324L	Bridge rail cracking SW end of bridge. Some approach settling in shoulders.	Major	RTE 118/5 SEP
118	11.52	53 2327F	4" abutment movement (major damage). Damage at hinge bearings and restrainers.	Major	SE CONN (W118/S005)
118	13.94	53 2102G	Hinge restrainer damage. Vertical offset. Ground displacement at base of columns.	Major	RTE 118/210 SEP
118	2.55	53 2182	Cracked shear keys. Delaminated bearing pads.	Major	BROWN CANYON WASH
118	3.13	53 2498	Separation of wingwalls from abutment diaphragms. Cracked slope paving.	Major	RINALDI ST
			Possible pile damage.		
118	8.34	53 2204	Approach settlement (~1').	Major	HAVENHURST

4.3.2 Description of Bridge Damage

The bridge damage from the Northridge earthquake pertained mostly to state bridges in Los Angeles and Ventura Counties. Bridges in these two counties also experienced much higher accelerations than bridges in Riverside and Orange Counties where there were 63 bridges exposed to peak ground acceleration (PGA) levels of 0.15g or higher. Immediately after the Northridge earthquake, Caltrans inspected about 850 state bridges in the vicinity of the epicenter, and reported 233 of them as damaged [Caltrans, 1994b; Buckle, 1994; EERI, 1995]. Only 5 of the damaged bridges were in Ventura County. The remaining bridges were in Los Angeles County.

Several statistics were obtained for the bridges that sustained damage from the Northridge earthquake. Figure 4-8 shows the distribution of the damaged bridges by service type and damage state. Ten pedestrian and two railroad bridges were reported as damaged by Caltrans. The inventory of steel bridges in Greater Los Angeles area is small compared to that of concrete bridges, as shown in table 4-2. A detailed description of damage to steel bridges is given by Astaneh-Asl [1995]. Pounding damage between adjacent elements, buckling of cross bracing, bending of cross-brace gussets, restrainer fractures, damage to the interface between the steel superstructure and concrete substructure and damage to anchor bolts due to shearing forces and high tensile forces were the most common forms of damage to steel bridges [EERI, 1995]. Three steel bridges were reported to have *major* damage, 11 bridges sustained *moderate* damage and another 5 sustained *minor* damage. Due to the limited number of steel bridges that were damaged, steel bridges were excluded from the correlation studies.

Two hundred of the damaged bridges were concrete highway bridges, of which 28 were single span and 172 were multiple span bridges. However, only 164 of the damaged bridges (27 single span, 137 multiple span bridges) had homogeneous abutment and/or column bent types. Four of these 164 bridges were exposed to PGA levels less 0.15g, therefore, a total of 160 bridges were included in the *correlation data set*. Table 4-5 summarizes the number of bridges in each data set used in statistical analyses.

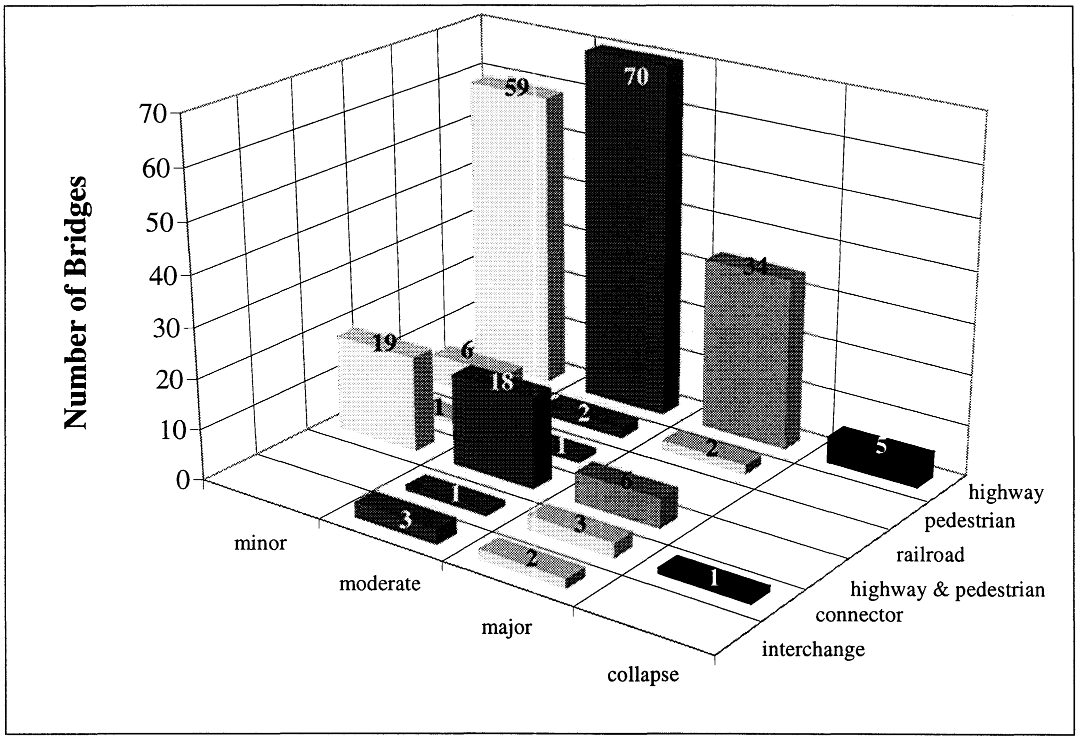


FIGURE 4-8 Distribution of All Damaged Bridges by Service Type and Damage State

TABLE 4-5 Number of Damaged Bridges Included in Various Data Sets

		Highway Bridge Data Set	Concrete Highway Bridge Data set	Homogeneous Data Set	Correlation Data Set
Los Angeles County	*Single span*	27	27	26	26
	Multiple span	189	170	135	131
Ventura County	*Single span*	1	1	1	1
	Multiple span	4	2	2	2
Total		221	200	164	160

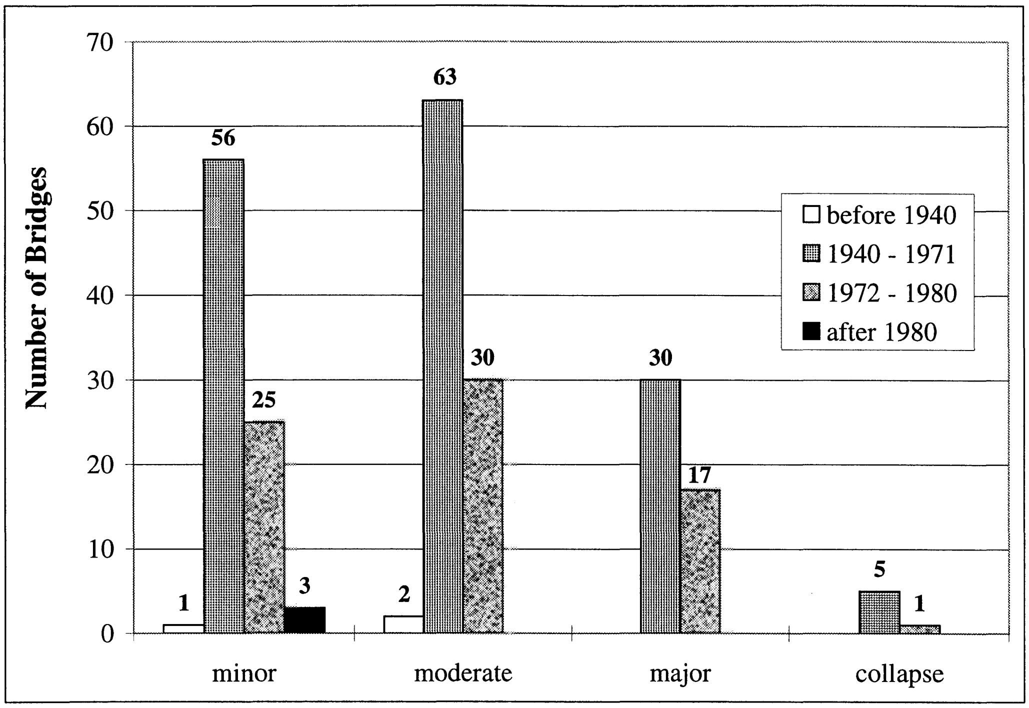


FIGURE 4-9 Distribution of All Damaged Bridges by Design Year and Damage State

Figure 4-9 shows the distribution of all damaged bridges by design year and damage state. Sixty eight percent of the damaged bridges were designed by pre-1971 design standards, a ratio close to that of all the bridges in the *highway bridges data set* (76 percent). Seventy-three bridges that were designed during 1972 to 1980 period sustained damage. Fifty-five of these bridges were located on soft alluvial (Type C) soils mostly with skews greater than 20°. The PGA levels at these bridge sites ranged from 0.35g to 1.10g and 0.20g to 0.66g according to USGS and WCFS ground shaking maps, respectively.

The distribution of damaged bridges by HAZUS classification and damage state is shown in figure 4-10. The bridge classes HBR-3, 6, 9 and 11 represent respectively the seismically designed/retrofitted continuous bridges, seismically designed/retrofitted simply supported bridges, seismically designed/retrofitted "high risk" continuous bridges and seismically designed/retrofitted "high risk" simply supported bridges.

In Section 4-4, the ground motion-damage relationships provided in HAZUS [1997] are compared with the observed damage for these classes.

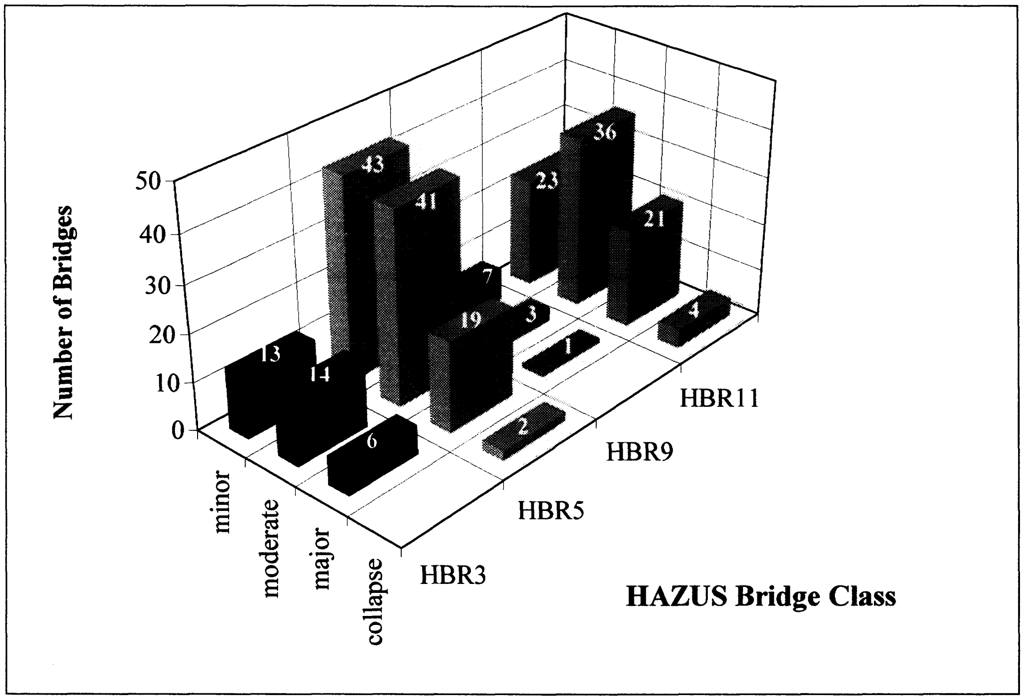


FIGURE 4-10 Distribution of All Damaged Bridges by HAZUS Bridge Classification and Damage State

The six bridges that collapsed in the Northridge earthquake are listed in table 4-6. The damage descriptions, ground motion levels at bridge sites, structural characteristics and repair costs for these bridges are also included in table 4-6. All collapsed bridges were of concrete box girder type with multiple spans constructed with pre-1971 design standards. Despite the relatively low level of ground shaking, high skew, irregularity in substructure stiffness, and inadequate seat width in some cases caused the collapse of these bridges. Structural characteristics, observed damage and the failure analysis of collapsed bridges are discussed in detail by Caltrans [1994a], Buckle [1994] and EERI [1995].

Only four of the six bridges that collapsed in the Northridge earthquake had homogeneous abutment and/or column bent types. Thus, only four of the six collapsed bridges were included in the *correlation data set*. Distribution of damaged bridges in the *correlation data set* by bridge sub-categories listed in table 2-1 is shown in figure 4-11. Figures 4-12 through 4-15 show the distribution of damaged bridges in the *correlation data set* by number of spans, design year, abutment type and column bent type. Twenty percent of the damaged bridges were single span

TABLE 4-6.a Characteristics of Bridges that Collapsed in the Northridge Earthquake

Bridge No.	Bridge Name	Route	No. of Spans	Superstr. Type[1]	Abut. Type[2]	Substr. Type[3]	Span Continuity	Design Year	Skew[4]	Soil Type[5]	Retrofit History
53 1609	La Cienega-Venice UC	I-10	9	CBC	G	H	discontinuous	1964	99	C	Phase I, 1978
53 1960F	Route 14/5 Separation&OH	I-5 / SR114	10	QBC, CBC	C	J	discontinuous	1971	0	C	Phase I, 1974
53 1964F	North Connector	I-5 / SR114	10	CBC, QB	A	J	discontinuous	1971	0	C	Phase I, 1975
53 1797L	Gavin Canyon UC	I-5	5	QBC, CBC	A	H	discontinuous	1967	67	C	Phase I, 1974
53 1797R	Gavin Canyon UC	I-5	5	QBC, CBC	A	H	discontinuous	1967	67	C	Phase I, 1974
53 2205	Mission-Gothic UC	SR118	4	QBC	B	H	continuous	1973	99	C	-

TABLE 4-6.b Damage Description of Bridges that Collapsed in the Northridge Earthquake

Bridge No.	PGA (g)	MMI	Damage Description	Repair Cost ($)[6]
53 1609	0.35	VIII	Most of the columns of mainline structure buckled (high skew contributed).	26,993,750
53 1960F	0.95	IX	2 spans collapsed onto route 5. Needed shoring for 1 hinge H = 50' (15.25 m.)	17,260,000
53 1964F	0.95	IX	Span drop from abutment 1 to bent 3.	30,186,000
53 1797L	0.85	VIII	Collapse of spans 2 and 4 due to heavy skew and narrow hinge seats.	22,045,000
53 1797R	0.85	VIII	Collapse of spans 2 and 4 due to heavy skew and narrow hinge seats.	22,045,000
53 2205	1.05	VIII	Partial collapse; approximately 24 cm sag; 2 columns buckled at bent 1.	13,540,000

1 CBC: continuous concrete box girder bridge; QBC: continuous cast in place prestressed box girder bridge; QB: cast in place prestressed box girder bridge.

2 A: diaphragm; B: seat; C: cantilever; G: cellular closure.

3 H: multiple columns/bent; J: single column/bent.

4 99 represents varying skew along the bridge.

5 Type C soil refers to younger and softer alluvial deposits as defined by Boore et al. [1993].

6 Source: Bridge repair cost reports for the Northridge earthquake, [Caltrans, 1994b].

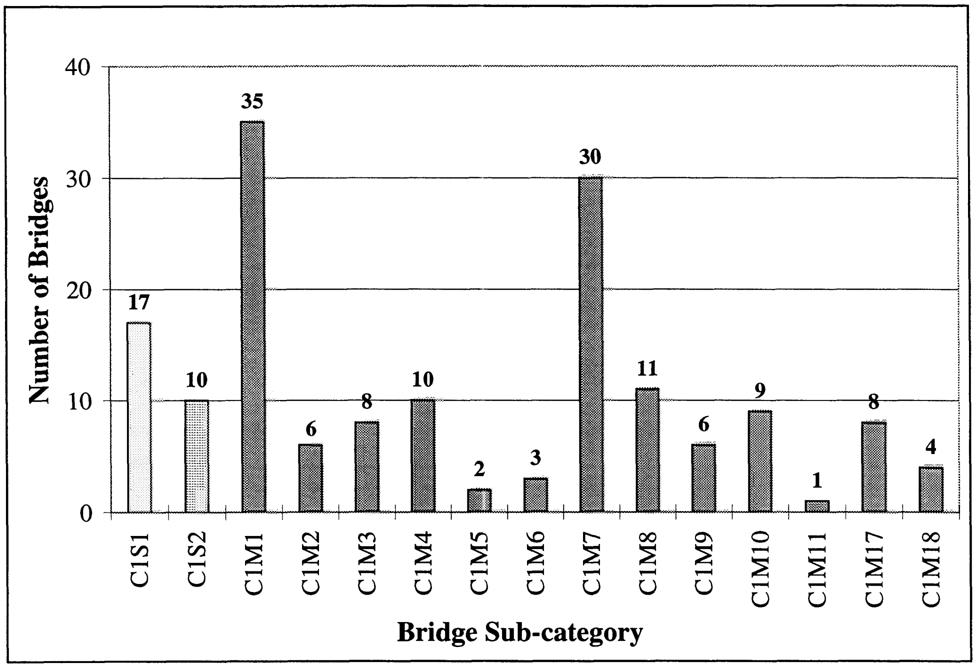


FIGURE 4-11 Distribution of Damaged Bridges by Bridge Sub-category (*correlation data set*)[1]

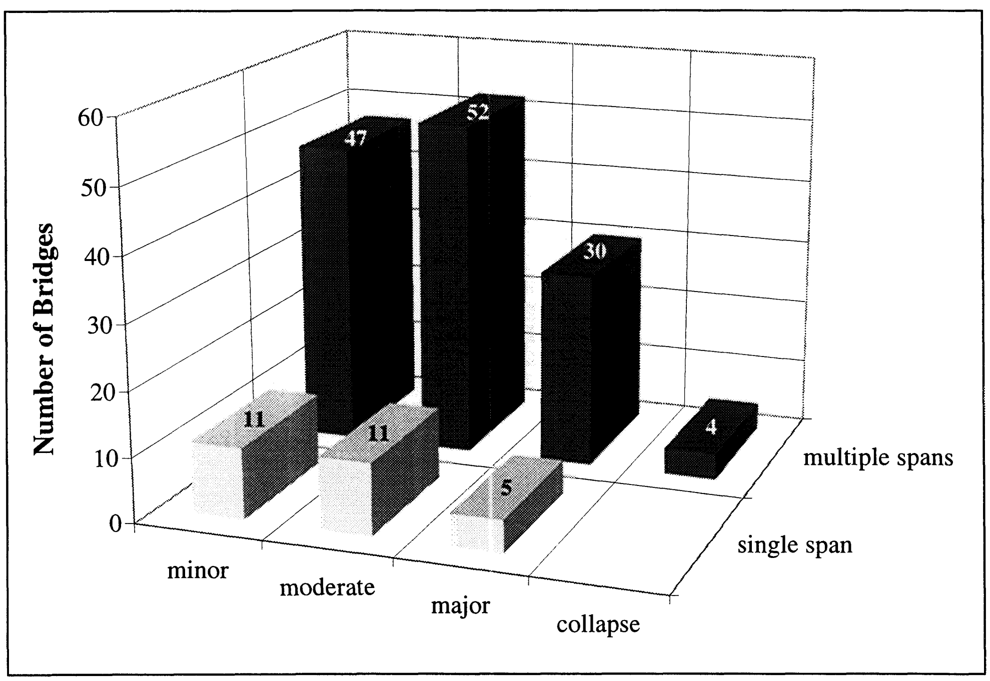


FIGURE 4-12 Distribution of Damaged Bridges by Number of Spans and Damage State (*correlation data set*)

[1] See table 2-1 for description of bridge sub-categories.

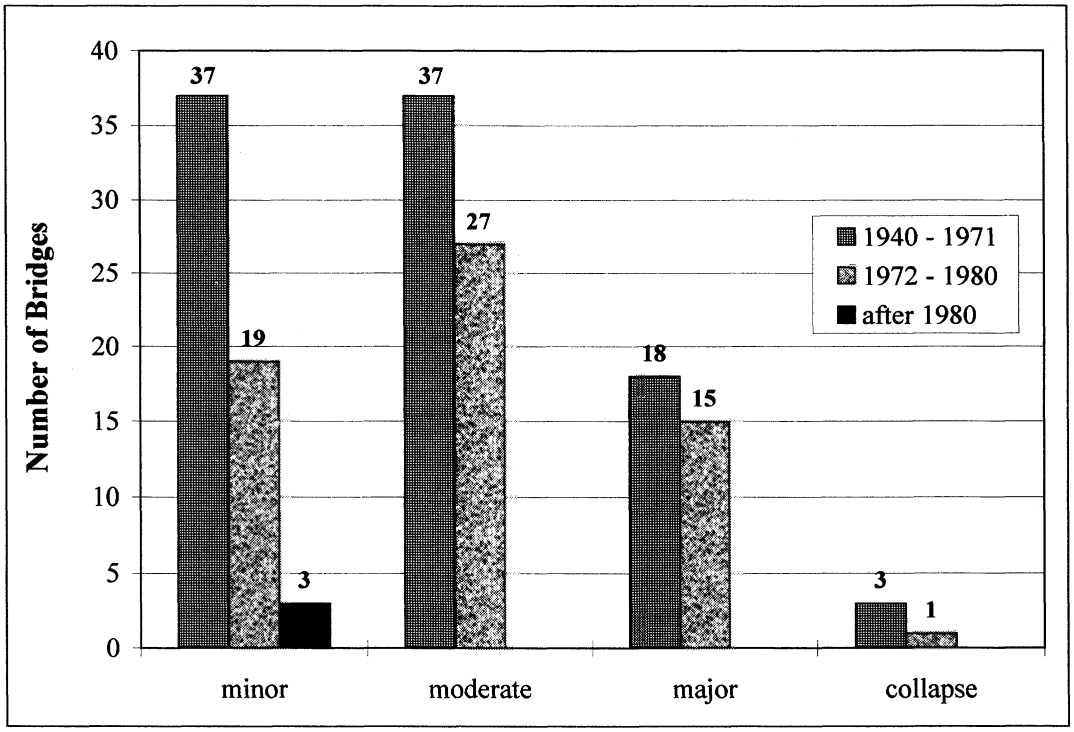


FIGURE 4-13 Distribution of Damaged Bridges by Design Year and Damage State (*correlation data set*)

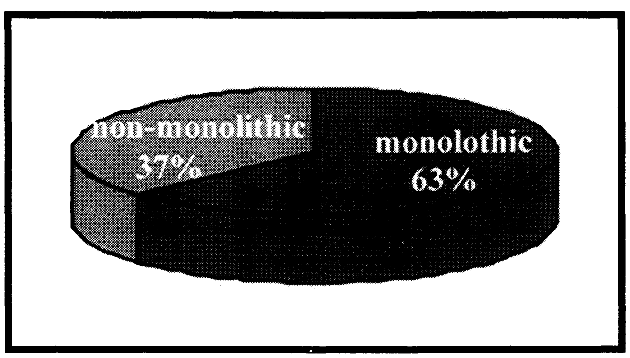


FIGURE 4-14 Distribution of Damaged Single Span Bridges by Abutment Type (*correlation data set*)

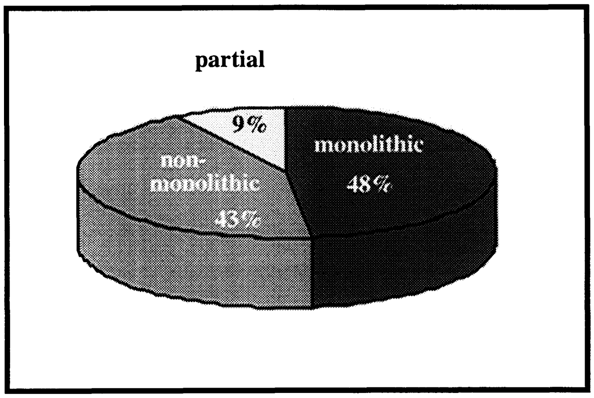


FIGURE 4-15.a Distribution of Damaged Multiple Span Bridges by Abutment Type (*correlation data set*)

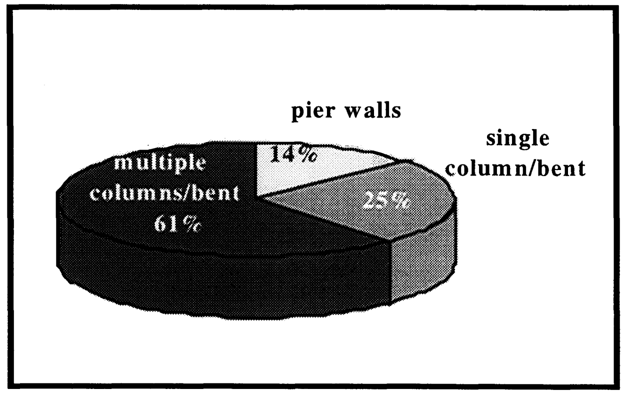


FIGURE 4-15.b Distribution of Damaged Multiple Span Bridges by Column Bent Type (*correlation data set*)

bridges. None of the single span bridges with bin or cellular closure type abutments were reported as damaged. Multiple span bridges with pier walls (bridge sub-categories C1M13 through C1M16) experienced no damage except when the bridge had bin or cellular closure type of abutments (C1M17 and C1M18). Thirteen percent of the multiple span continuous bridges with *monolithic* abutment types (C1M1) were reported as damaged. Similarly, 15 percent of all the multiple span continuous bridges with *non-monolithic* abutment types (C1M7) were reported as damaged.

Figures 4-15.a and 4-15.b show the distribution of damaged bridges by abutment and column bent types. It could be inferred from figure 4-15.a that fewer bridges with non-monolithic abutment types sustained damaged than the ones with monolithic type abutments. However, this is misleading since in the *correlation data set* the number of bridges with monolithic abutments was higher than that of bridges with non-monolithic abutments. Instead, the damage ratio (i.e., the ratio of number of damaged bridges to the number of bridges in the inventory should be used for comparison. Figure 4-16.a shows the ratio of total number of damaged bridges with monolithic abutments to the total number of bridges with monolithic abutments at a given PGA level.

For example, according to figures 4-16.a and 4-16.b, all bridges with non-monolithic abutments were damaged at PGA levels between 0.5g and 0.6g compared to 70 percent for bridges with monolithic abutment types. Figures 4-16.b and 4-16.c depict a similar ratio for multiple span bridges with non-monolithic abutments and bin or closure type abutments, respectively. Similar explanations are valid for column bent types as demonstrated in figures 4-17.a, 4-17.b and 4-17.c.

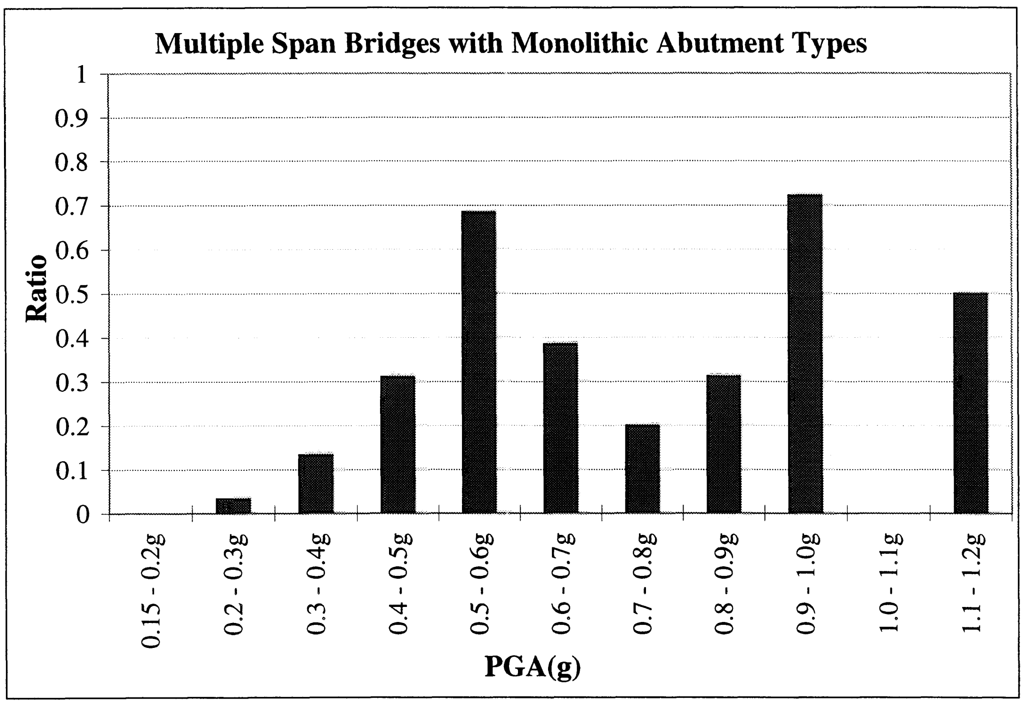


FIGURE 4-16.a Comparison of Damaged Bridges with Monolithic Abutment Types to Total Number of Bridges with Monolithic Abutment Types

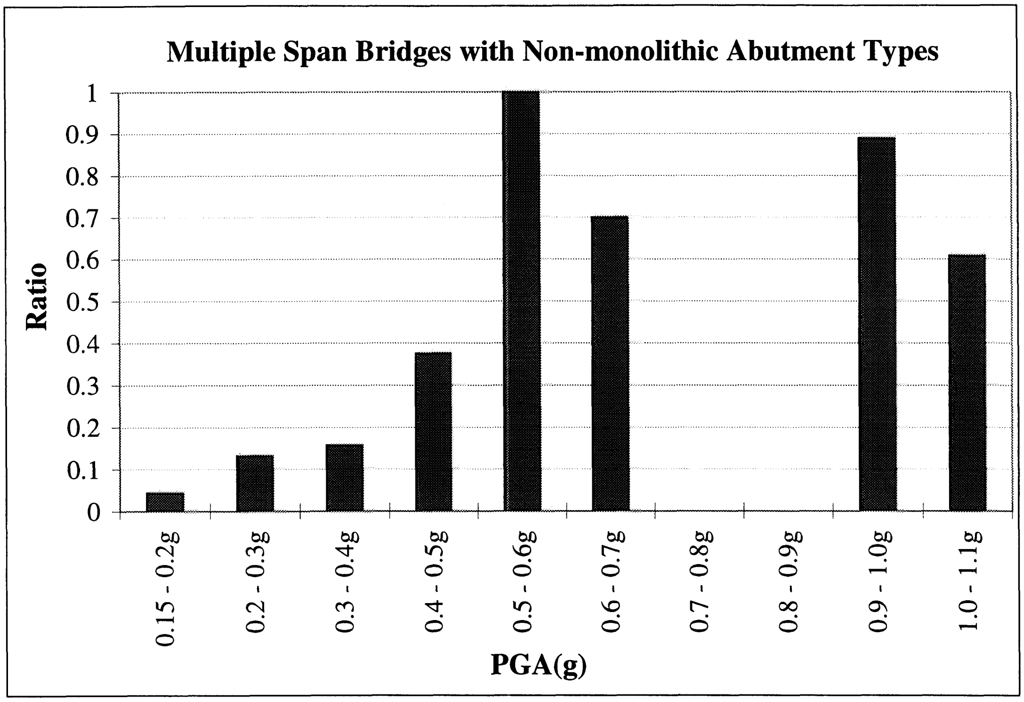


FIGURE 4-16.b Comparison of Damaged Bridges with Non-monolithic Abutment Types to Total Number of Bridges with Non-monolithic Abutment Types

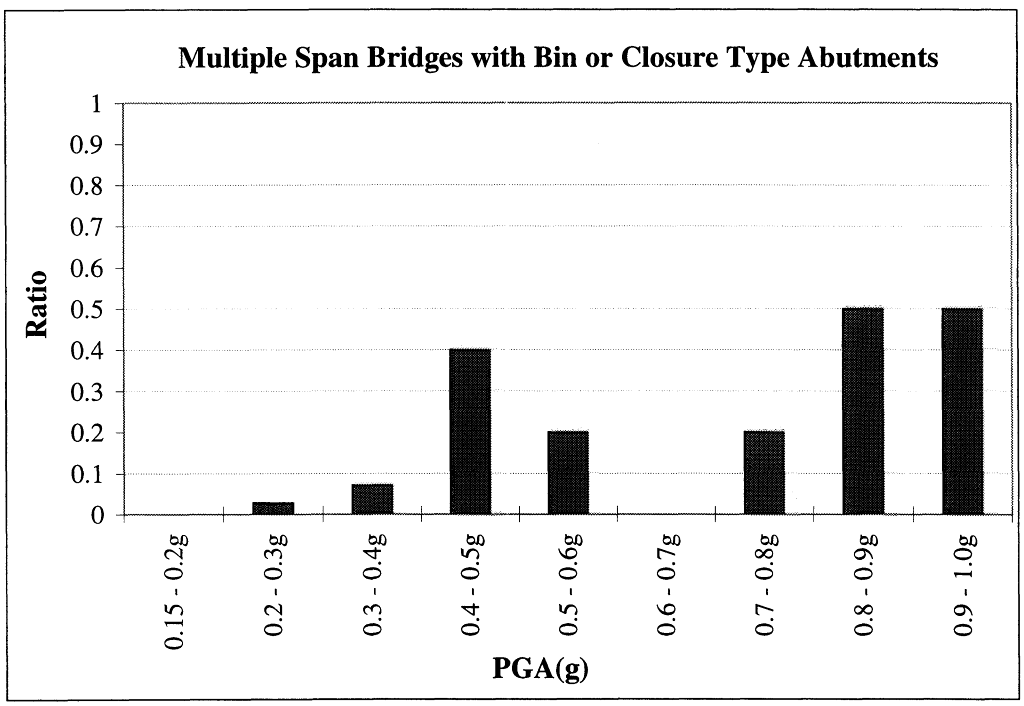


FIGURE 4-16.c Comparison of Damaged Bridges with Partial Abutment Types to Total Number of Bridges with Partial Abutment Types

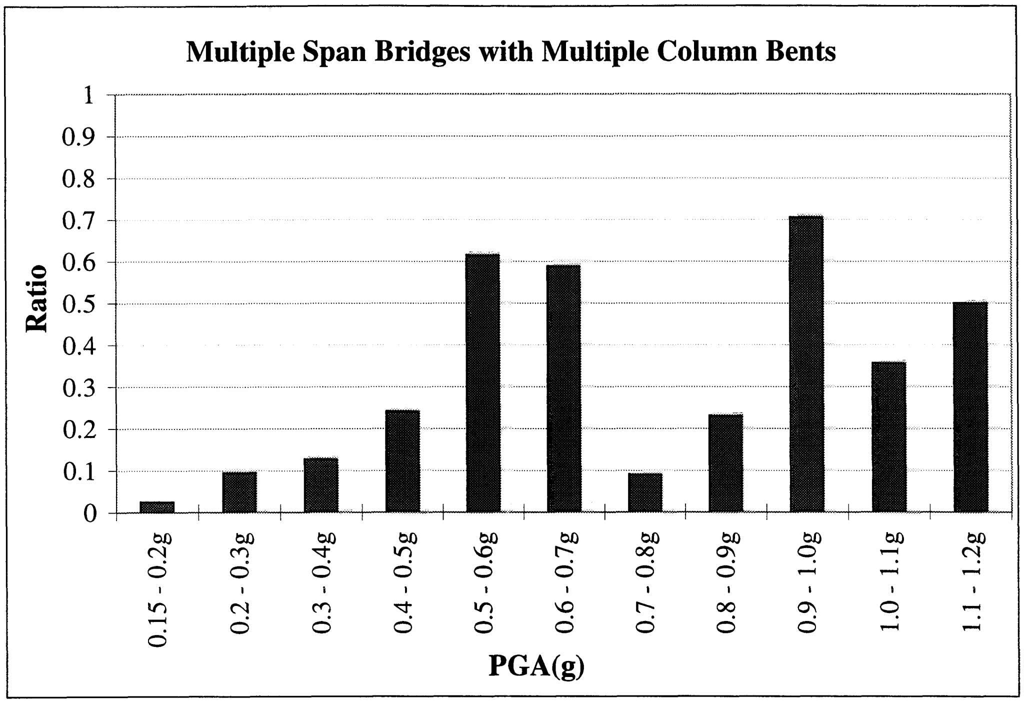


FIGURE 4-17.a Comparison of Damaged Bridges with Multiple Column Bents to Total Number of Bridges with Multiple Column Bents

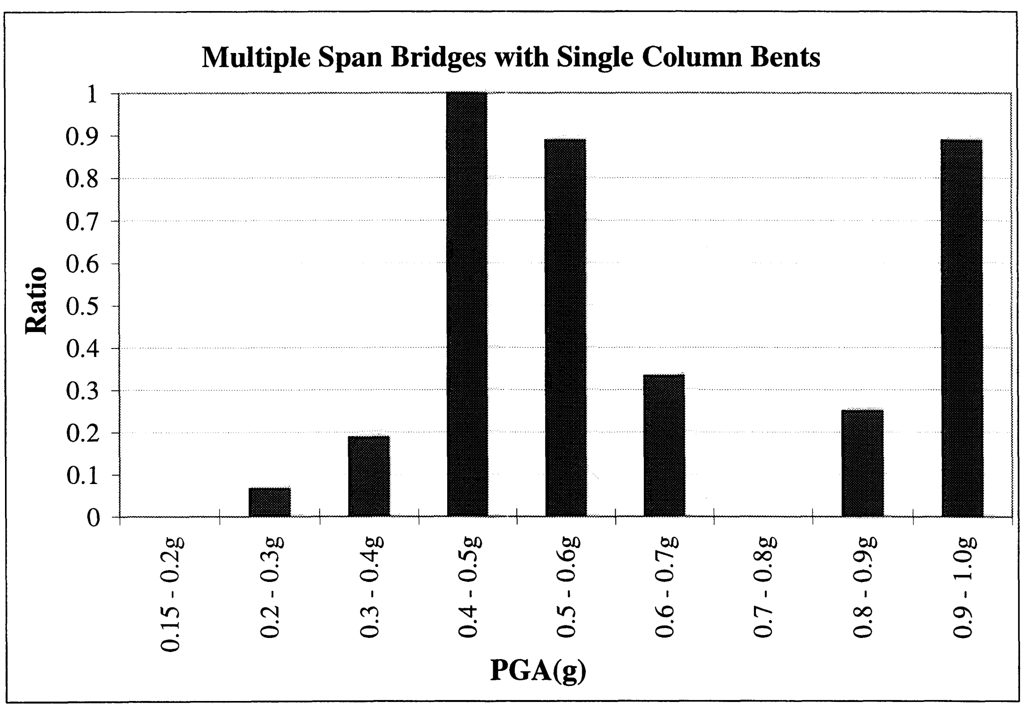


FIGURE 4-17.b Comparison of Damaged Bridges with Single Column Bents to Total Number of Bridges with Single Column Bents

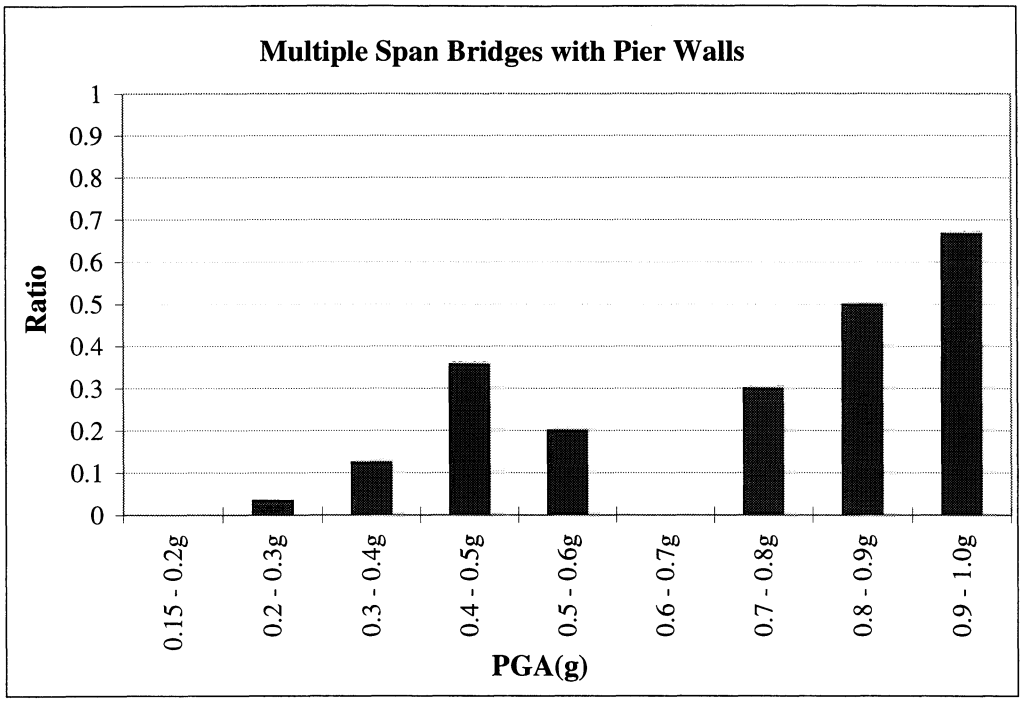


FIGURE 4-17.c Comparison of Damaged Bridges with Pier Walls to Total Number of Bridges with Pier Walls

Damage to Retrofitted Bridges

Several of the retrofitted bridges were impacted in the Northridge earthquake. A detailed description of performance of retrofitted bridges is given by Yashinsky [1995]. In most cases the retrofitted bridges, especially with jacketed columns, performed well. There were cases in which the hinge restrainers did not perform adequately. Based on the list of retrofitted bridges provided by Yashinsky [1995], 189 of the bridges in the *highway data set* were retrofitted. One of the three types of retrofits was used for these bridges: (*i*) hinge restrainers (phase I), (*ii*) column retrofitting (phase II), and (*iii*) phase I and II. The statistical analyses performed for retrofitted and unretrofitted bridges are presented in Section 4.4.

4.3.3 Relative Importance of Attributes that Most Contribute to Damage

A multivariate logistic regression analysis was conducted to identify the characteristics that contribute most to bridge damage. In the analyses, peak ground acceleration, skew, abutment type, column bent type, span length, design year, span length, number of spans, span continuity and soil type were included. Peak ground acceleration, span length and skew were defined as continuous variables. The analyses were conducted for: (*i*) multiple span bridges in the *correlation data set*, (*ii*) all bridges in the *correlation data set*, and (*iii*) multiple span damaged bridges in the *correlation data set.* Peak ground acceleration, abutment type, skew, span length and span continuity were found to be the most important characteristics in all cases. In addition, for the multiple span bridge data set, column bent type was found to be important. In the analyses, high correlation between span length and damage was observed. When the span length is defined as a binary variable, i.e., less than 500 feet or greater than or equal to 500 feet as used in the HAZUS [1997] and in ATC-13 [1985] bridge classifications, poor correlation was obtained between span length and bridge damage.

4.4 Correlation Studies for Bridge Damage

4.4.1 Empirical Fragility Curves

Correlation studies were performed to identify the structural characteristics that are most frequently associated with damage. In addition, empirical fragility curves were developed for bridges in the *correlation data set.* Bridges in this data set were first grouped to obtain damage frequency matrices, i.e., number of bridges in a given damage state at different ground motion levels, and to obtain damage probability matrices. Empirical fragility curves were developed from the damage frequency matrices using logistic regression analysis. In several cases, the data or the distribution of data by damage and PGA levels were not sufficient to obtain a statistically satisfactory fit for the fragility curves. Table 4-7 lists the different types of attributes considered in grouping bridges to obtain different sets of empirical fragility curves. The empirical fragility curves for various groups of bridges are presented in the Appendices listed in table 4-7. In addition, empirical fragility curves were developed by using only the groups of damaged bridges, i.e., excluding the undamaged bridges. These fragility curves (i.e., conditional fragility curves) would provide information on the damage state of a damaged bridge. However, for most of the bridge sub-categories considered in this study, the number of damaged bridges was insufficient to provide reliable statistics. Therefore, these empirical fragility curves are not presented in this report except for an example in figure 4-20.

TABLE 4-7 Characteristics of Data Groups Used in Correlation Analyses

Dependent Variable	Independent Variable	Data Set Grouped by	Results are Presented in
damage state	PGA values reported by USGS [1994]	• abutment type • number of spans • column bent type • sub-categories given in table 2-1 • design year • HAZUS classes • retrofit history	Appendix E
	PGA values reported by WCFS [1995]	• number of spans • sub-categories given in table 2-1 • design year • HAZUS classes	Appendix F

As discussed previously, the number of steel bridges in the inventory and among the bridges damaged in the Northridge earthquake was small. An empirical fragility curve was developed for all steel bridges, which is shown in figure 4-18. Correlation studies were not performed for different sub-categories of steel bridges due to lack of data.

Tables 4-8 and 4-9 show the damage frequency matrix and the damage probability matrix for the multiple span bridges included in the *correlation data set.* Figures 4-19 and 4-20 show the empirical fragility curves for all the multiple span bridges in this data set, unconditional and conditional on damage, respectively. The comparison of the empirical fragility curve for multiple span bridges with the observed data is shown in figure 4-21 for different damage states. Other examples of fragility curves are shown here for bridges grouped by the criteria listed in table 4-10. Figures 4-22 through 4-26 show these fragility curves. The PGA values shown on the horizontal axis of these figures are the observed values reported by USGS [1994]. Each figure consists of two graphs:

- ***GRAPH A*** shows the probability of exceeding a given damage state at different PGA levels,
- ***GRAPH B*** shows the probability of being in a damage state at different PGA levels.

TABLE 4-8 Damage Frequency Matrix for Multiple Span Bridges

	USGS Peak Ground Acceleration (g)						
Observed damage	0.15 - 0.2	0.2 - 0.3	0.3 - 0.4	0.4 - 0.5	0.5 - 0.6	0.6 - 0.7	0.7 - 0.8
None	194	262	150	31	10	15	17
Minor	2	8	16	2	6	4	2
Moderate	1	8	8	9	6	5	1
Major	0	6	0	5	5	3	1
Collapse	0	0	0	0	0	0	0
Total	197	284	174	47	27	27	21

	USGS Peak Ground Acceleration (g)						
Observed damage	0.8 - 0.9	0.9 - 1.0	1.0 - 1.1	1.1 - 1.2	1.2 - 1.3	1.3 - 1.4	total
None	18	7	9	1	0	1	717
Minor	0	5	1	1	0	0	47
Moderate	4	7	3	0	0	0	52
Major	1	9	0	0	0	0	30
Collapse	2	1	1	0	0	0	4
Total	25	29	14	2	0	1	848

TABLE 4-9 Damage Probability Matrix for Multiple Span Bridges (%)

	USGS Peak Ground Acceleration (g)						
Observed damage	0.15 - 0.2	0.2 - 0.3	0.3 - 0.4	0.4 - 0.5	0.5 - 0.6	0.6 - 0.7	0.7 - 0.8
None	98.48	92.25	86.21	65.96	37.04	55.56	80.95
Minor	1.02	2.82	9.20	4.26	22.22	14.81	9.52
Moderate	0.51	2.82	4.60	19.15	22.22	18.52	4.76
Major	0	2.11	0	10.64	18.52	11.11	4.76
Collapse	0	0	0	0	0	0	0

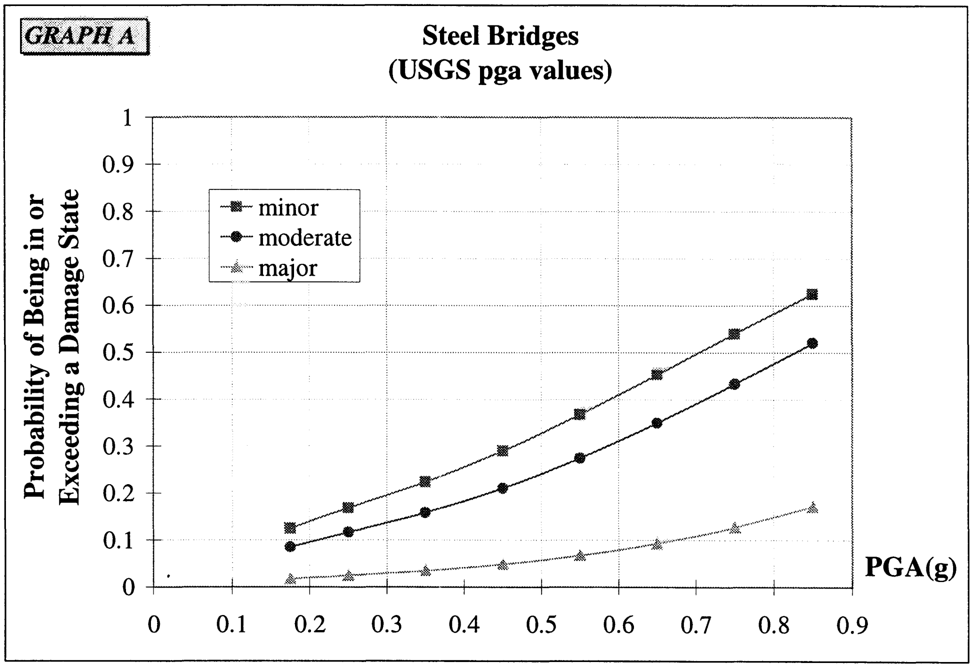
	USGS Peak Ground Acceleration (g)					
Observed damage	0.8 - 0.9	0.9 - 1.0	1.0 - 1.1	1.1 - 1.2	1.2 - 1.3	1.3 - 1.4
None	72.00	24.14	64.29	50	0	100
Minor	0	17.24	7.14	50	0	0
Moderate	16.00	24.14	21.43	0	0	0
Major	4.00	31.03	0	0	0	0
Collapse	8.00	3.45	7.14	0	0	0

The ratio of bridges in each damage state, and statistical significance test results are presented in these figures. The number of bridges in each damage state, i.e., *no damage*, *minor damage*, *moderate damage*, *major damage* and *collapse*, are listed. The statistical parameters are valid for both graphs and were briefly discussed in Section 2.3. In general, a model is accepted for a *p*-value of the Wald chi-square statistic at a significance level of less than or equal to 0.05. A chi-square test with (t - 2) degrees of freedom is used to test whether the parameter estimate on the model is zero. The variable t represents the number of response variable levels. When data exist only for two damage states at all ground motion levels, the chi-square statistics is not applicable and is denoted by N/A. The odds ratio reported in these figures is the exponentiated value of the corresponding parameter estimate.

TABLE 4-10 Description of Example Fragility Curves Obtained for Different Damage States

Dependent Variable	Independent Variable	Data Set Grouped by	Shown in figure
damage state	PGA values reported by USGS [1994]	• single span bridges	4-22
		• single span bridges with monolithic abutment types (C1S1)	4-23
		• multiple span bridges with monolithic abutment type and multiple columns/bent (C1M1)	4-24
		• bridges designed between 1940 and 1971	
		• bridges by retrofit history	4-25
			4-26

Number of bridges Chi-square with 1 DF	62, 5, 10, 3 6.865; p= 0.0088		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
3.5065	1.3656	0.0102	33.332



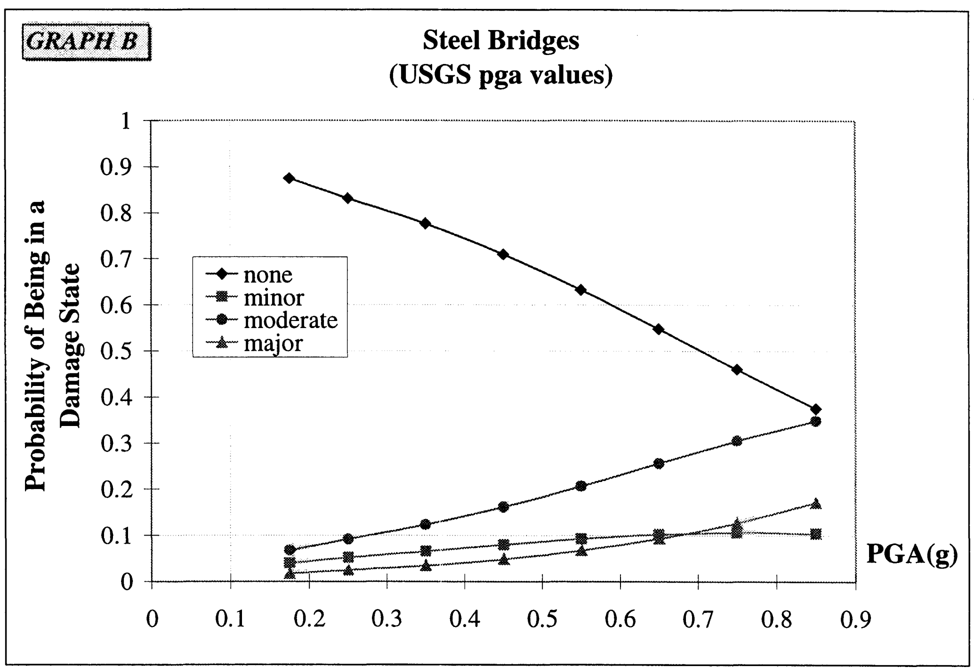
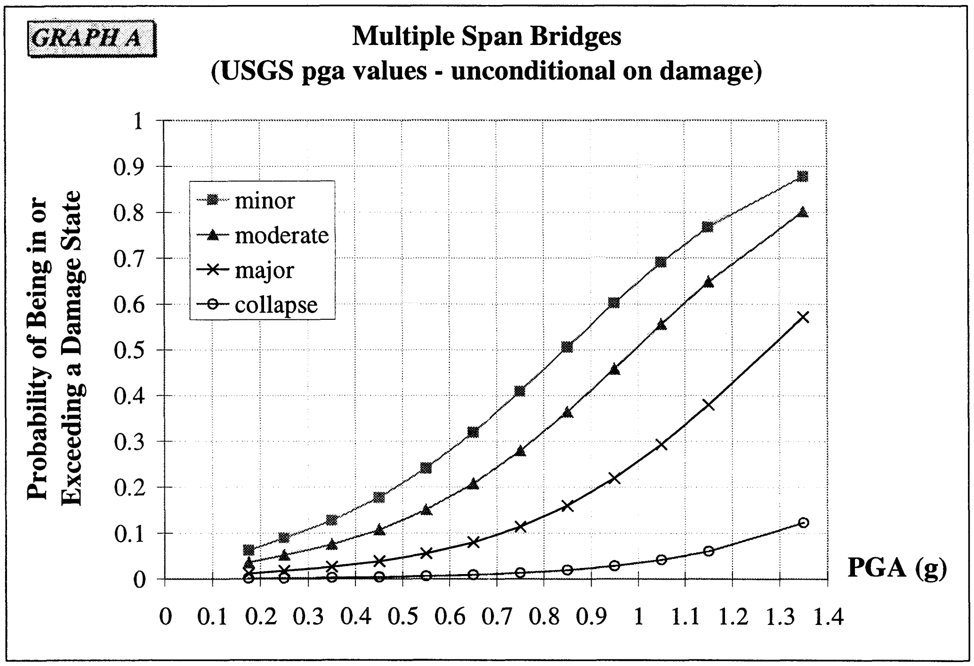


FIGURE 4-18 Empirical Fragility Curves for Steel Bridges

Number of bridges **Chi-square with 3 DF**	715, 47, 52, 30,4 4.5136; p= 0.2111		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
3.9028	0.3564	0.0001	49.543



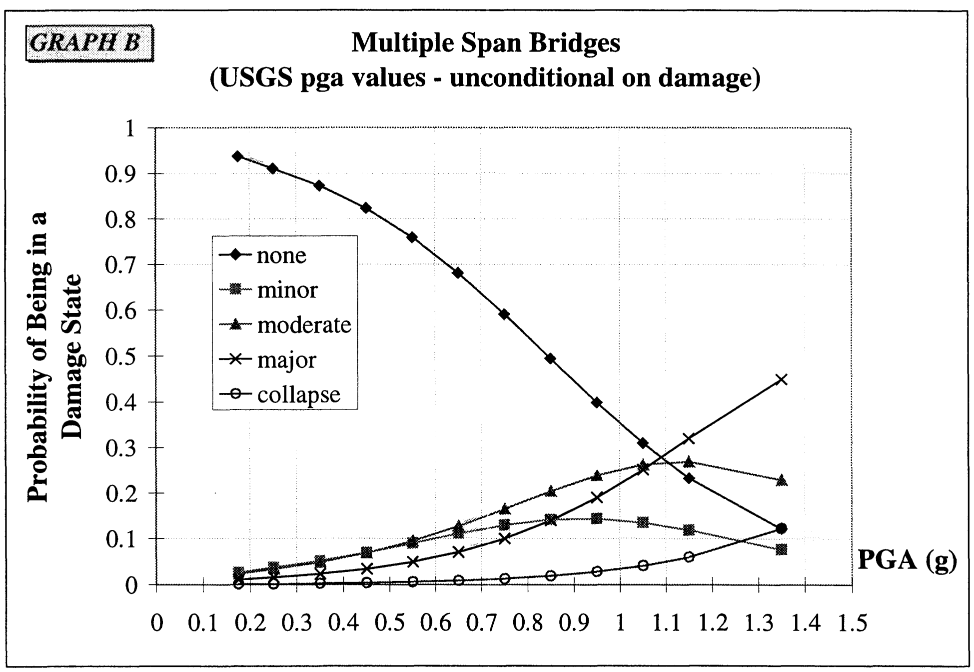
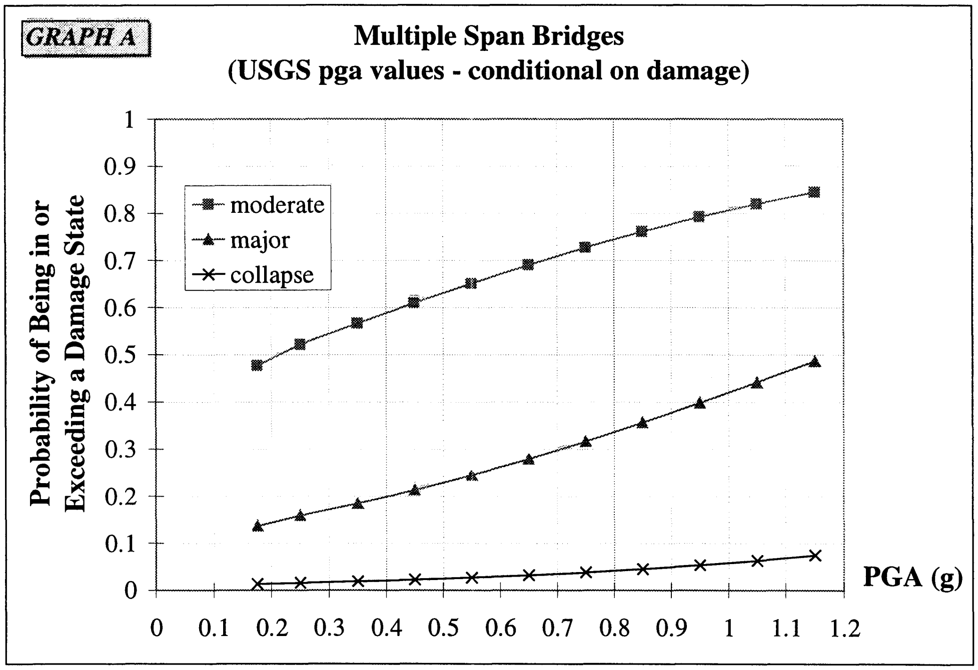


FIGURE 4-19 Empirical Fragility Curves for Multiple Span Bridges, Unconditional on Damage

Number of bridges Chi-square with 2 DF	47, 52, 30,4 3.80; p= 0.1496		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
1.7878	0.6144	0.0036	5.9778



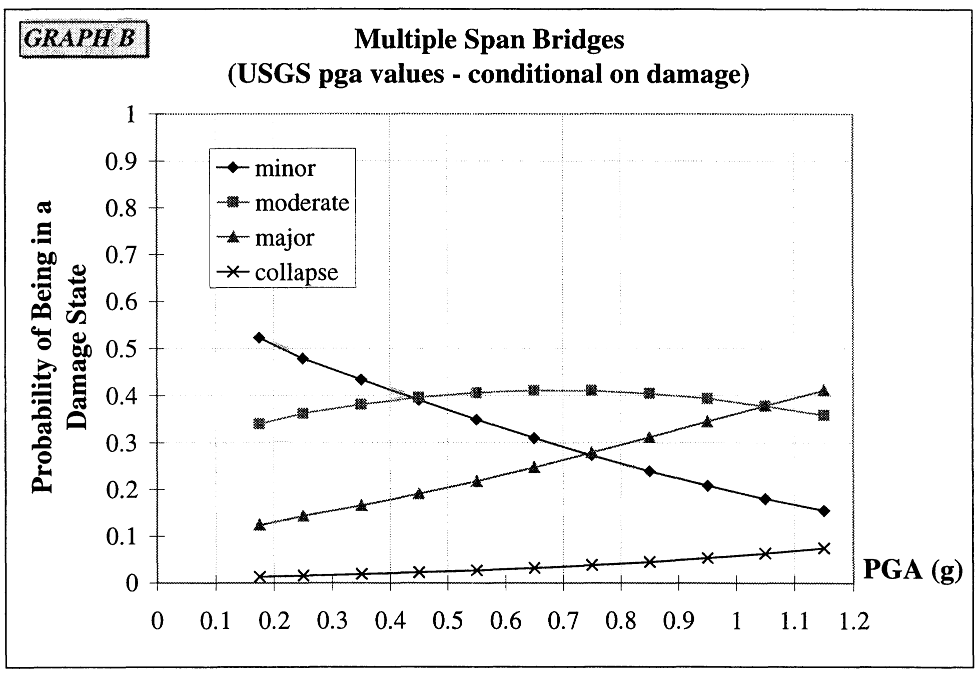


FIGURE 4-20 Empirical Fragility Curves for Multiple Span Bridges, Conditional on Damage

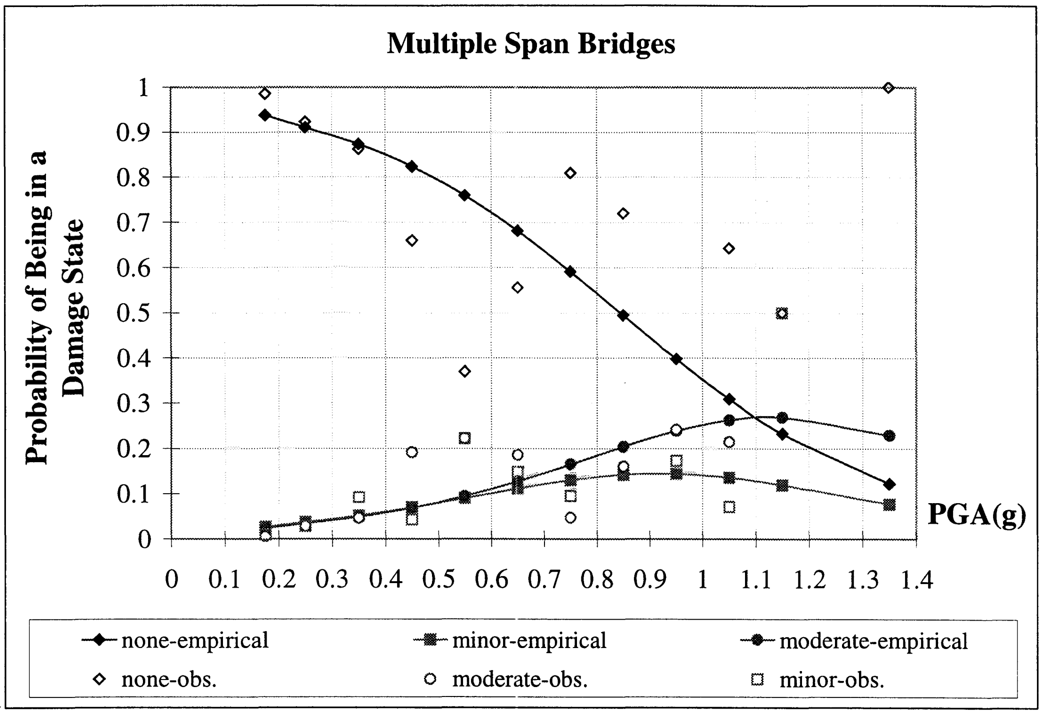


FIGURE 4-21.a Empirical Fragility Curves and Observed Data for Multiple Span Bridges – No Damage, Minor Damage and Moderate Damage

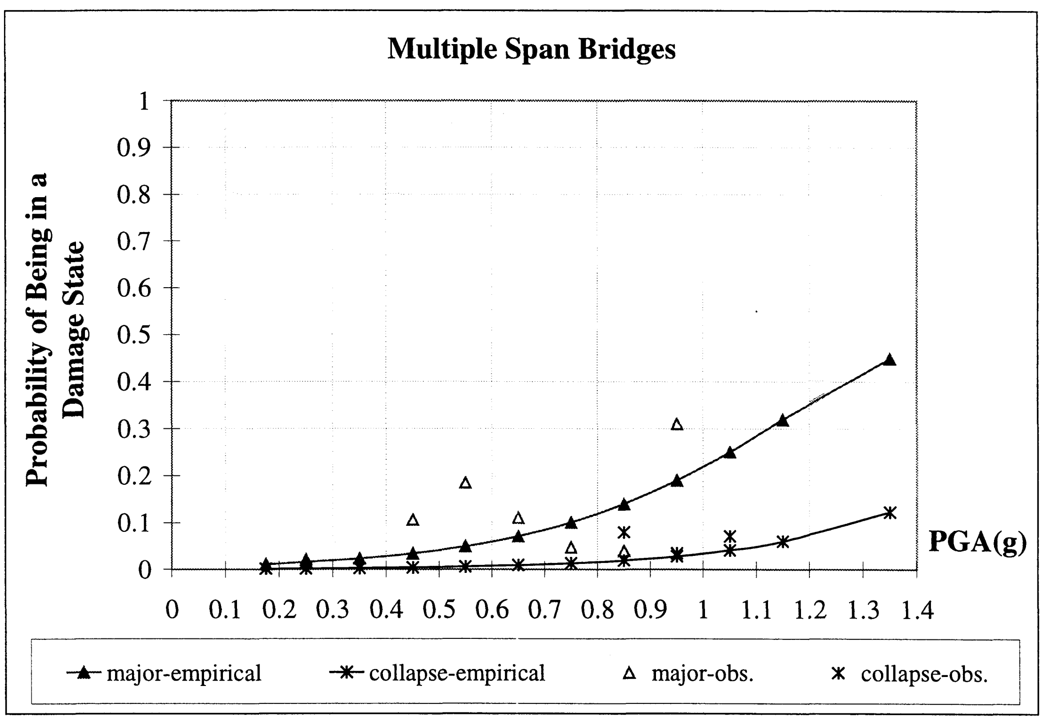
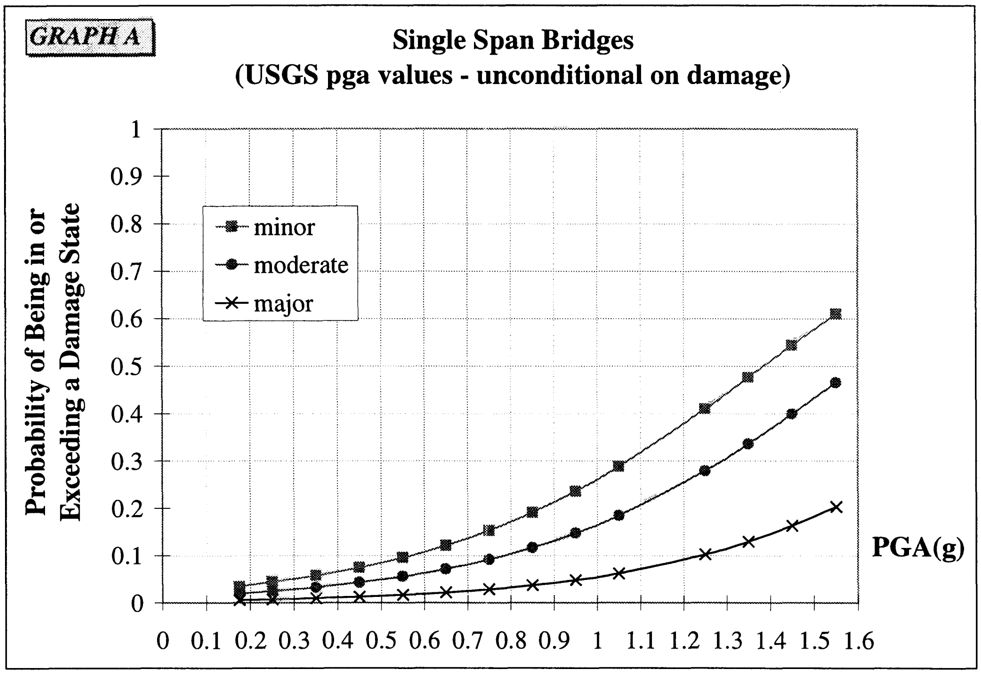


FIGURE 4-21.b Empirical Fragility Curves and Observed Data for Multiple Span Bridges - Major Damage and Collapse

Number of bridges	306, 11, 11, 5, 0		
Chi-square with 2DF	4.1333; p = 0.1266		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
2.6972	0.5393	0.0001	14.838



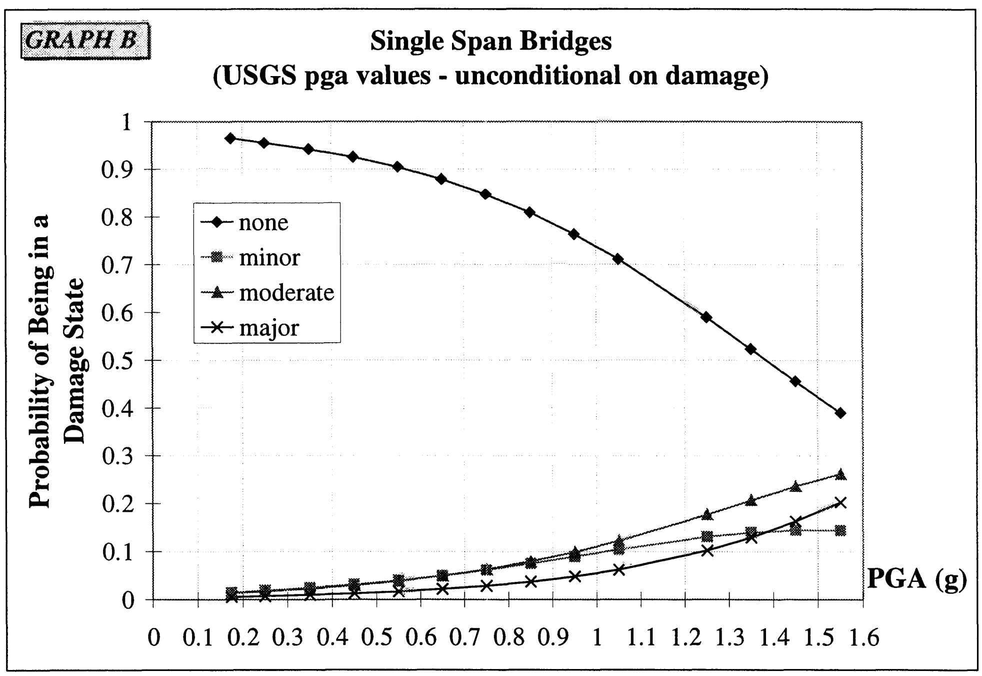
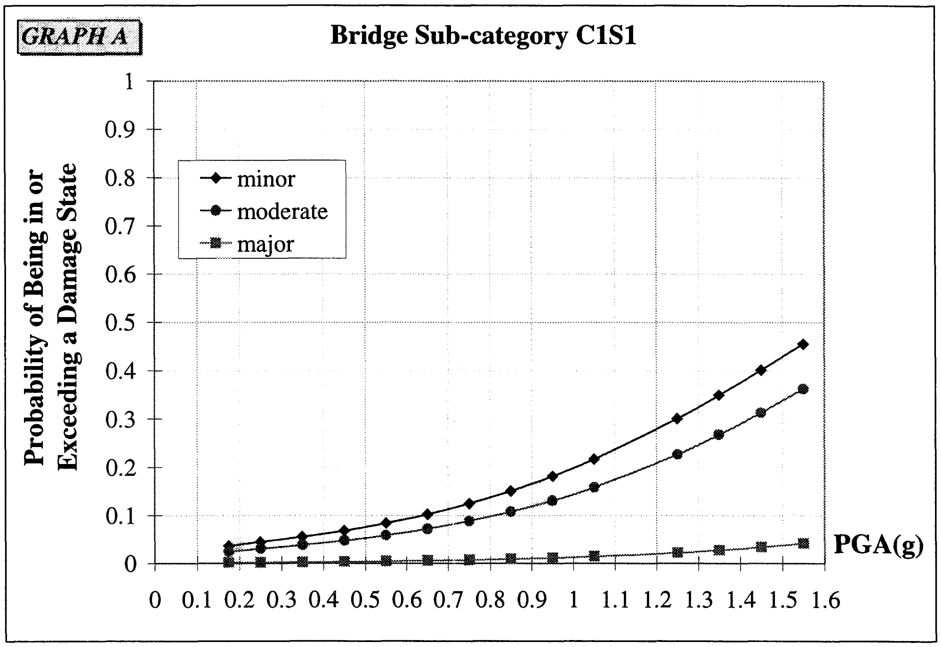


FIGURE 4-22 Empirical Fragility Curves for Single Span Bridges

Number of bridges **Chi-square with 1 DF**	197, 5, 11, 1, 0 5.9637; p = 0.0507		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
2.2107	0.6464	0.0006	9.1225



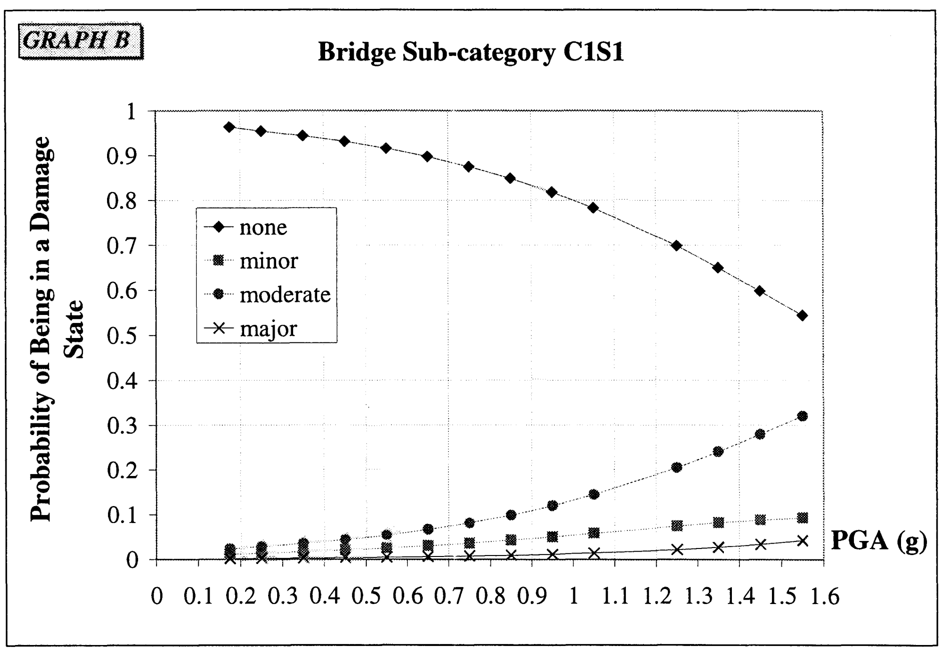
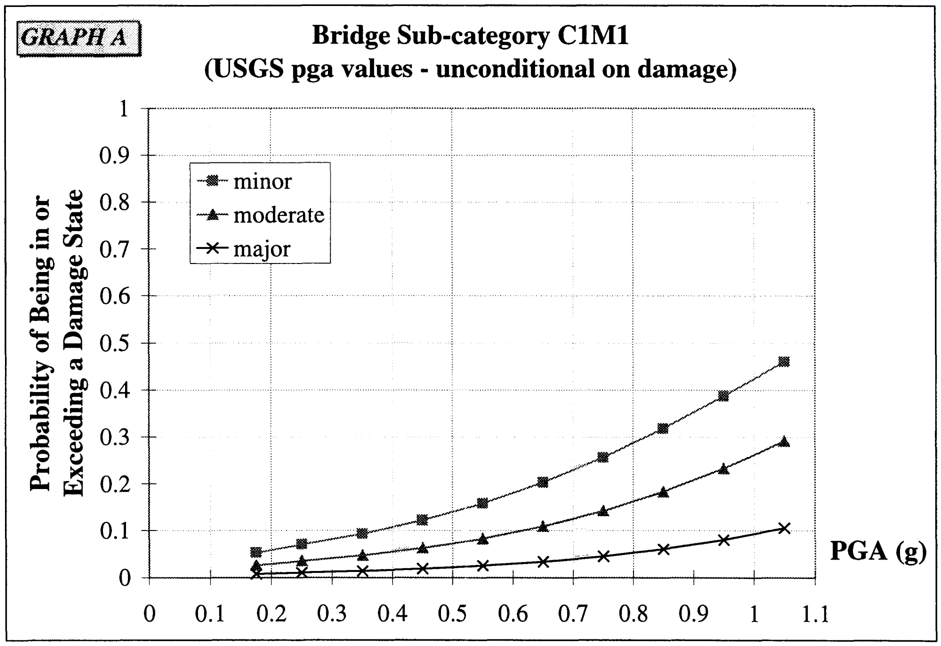


FIGURE 4-23 Empirical Fragility Curves for Single Span Bridges with Monolithic Type Abutments (Sub-category C1S1)

Number of bridges Chi-square with 2DF	228, 16, 13, 6, 0 0.2596; p = 0.8783		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
3.0251	0.6207	0.0001	20.597



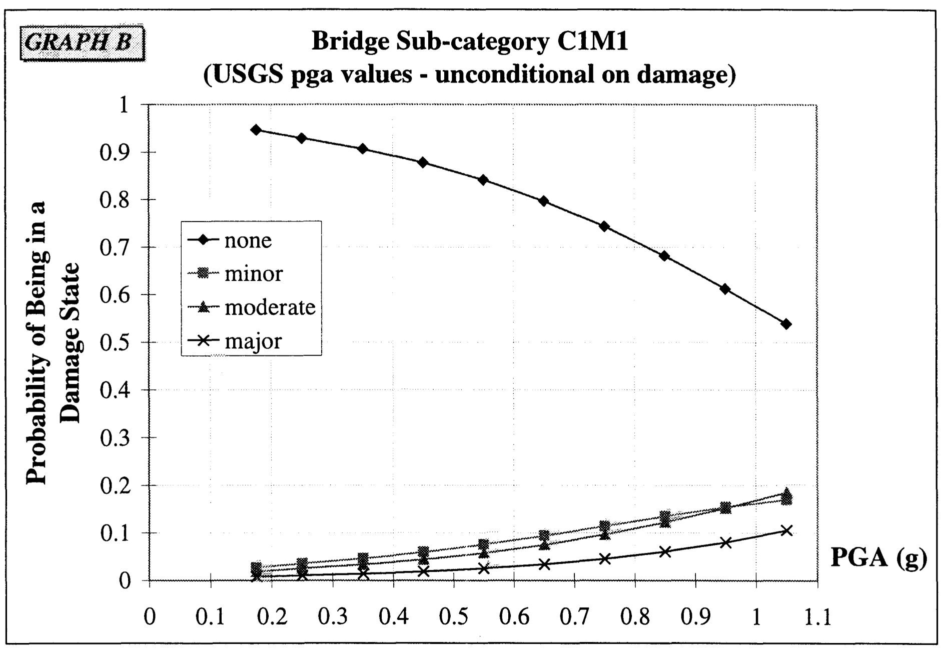
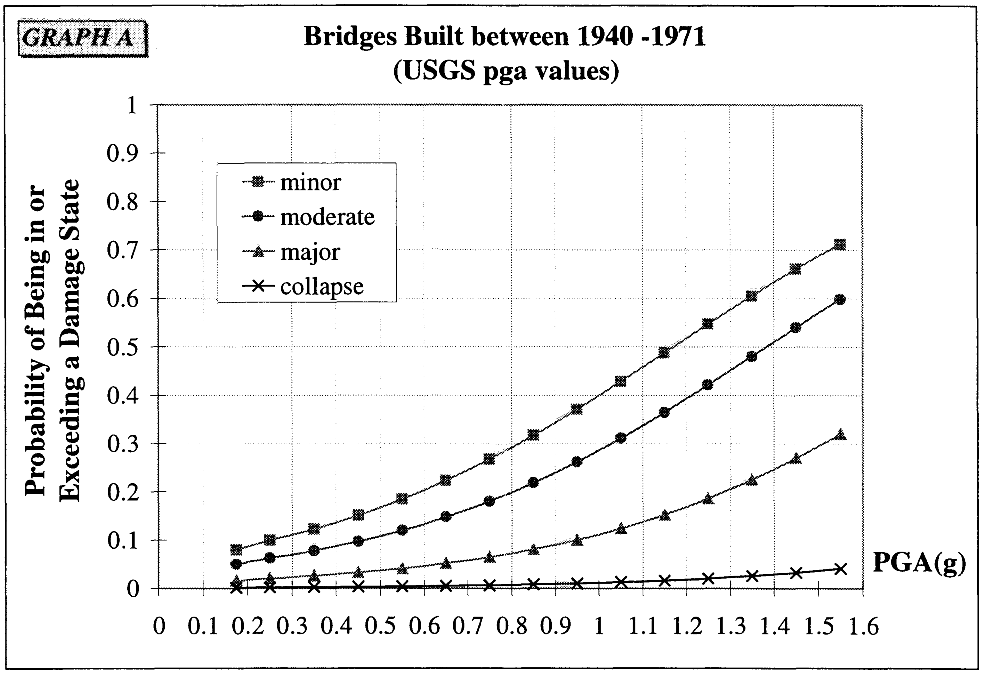


FIGURE 4-24 Empirical Fragility Curves for Multiple Span Bridges with Monolithic Type Abutments, Continuous Spans and Multiple Column Bents (Sub-category C1M1)

Number of bridges	887, 49, 59,28, 3		
Chi-square with 3 DF	9.5481; p = 0.0228		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
2.3905	0.3152	0.0001	10.9195



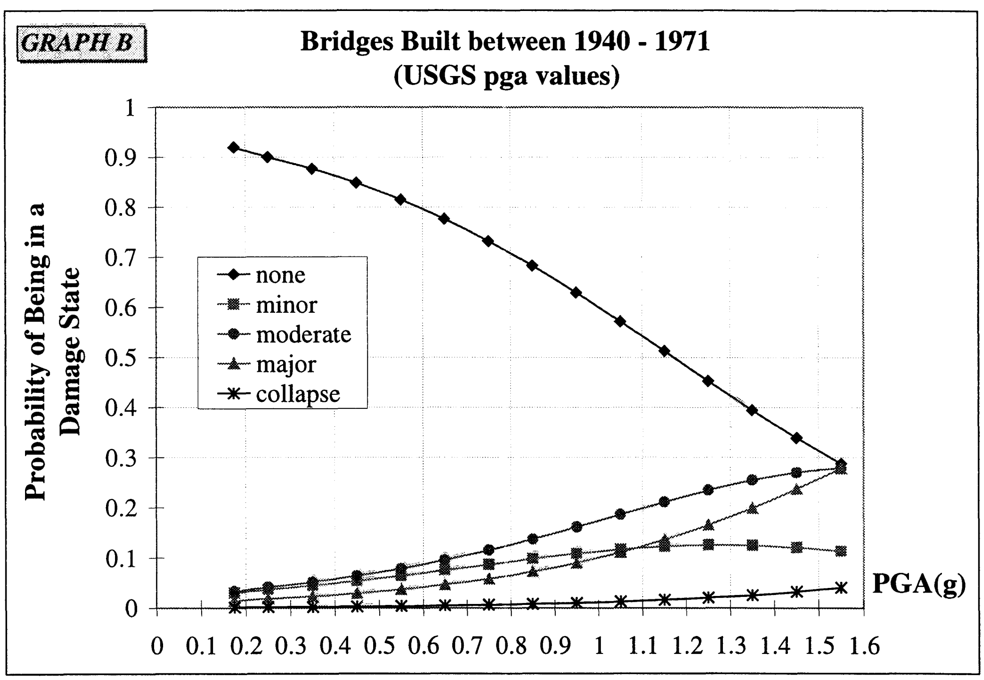


FIGURE 4-25 Empirical Fragility Curves for Bridges Built between 1940 and 1971

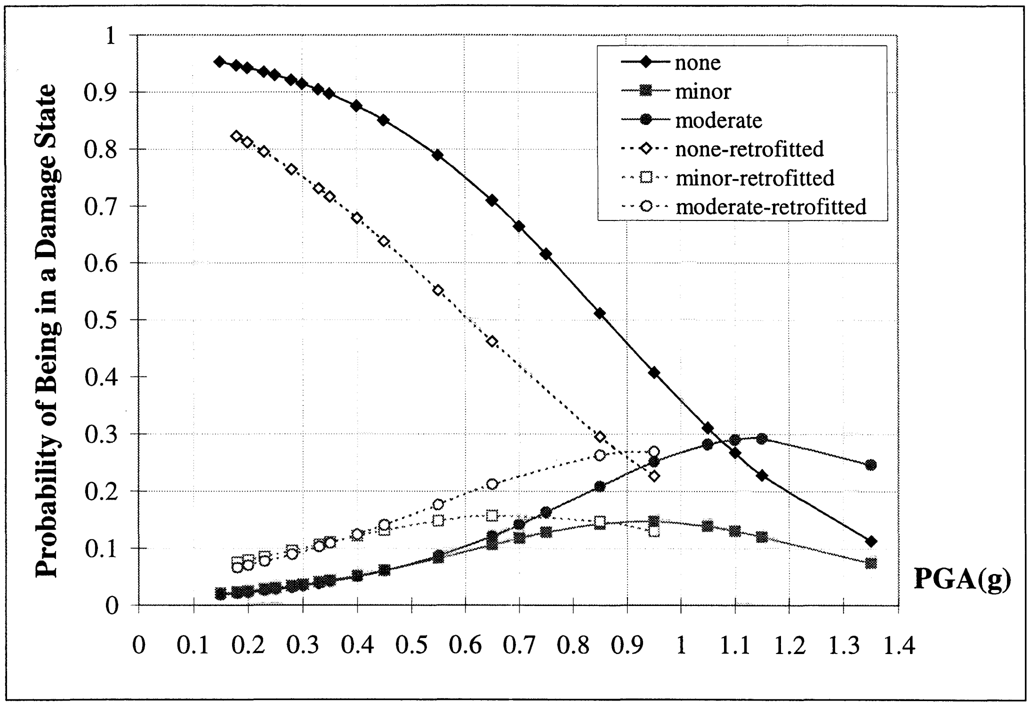


FIGURE 4-26.a Comparison of Empirical Fragility Curves for Retrofitted and Unretrofitted Bridges - No Damage, Minor Damage and Moderate Damage

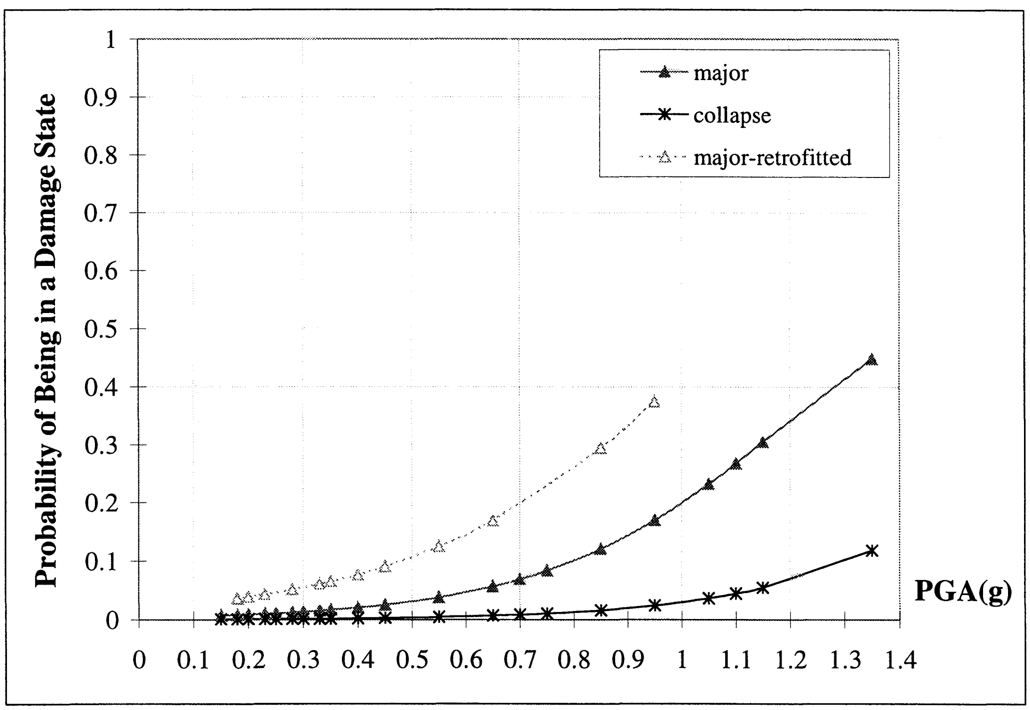


FIGURE 4-26.b Comparison of Empirical Fragility Curves for Retrofitted and Unretrofitted Bridges - Major Damage and Collapse

4.4.2 Comparison of HAZUS Fragility Curves and Observed Damage

In order to compare the observed damage with the fragility curves provided in HAZUS [1997], empirical fragility curves were developed for bridges grouped by the HAZUS bridge classification. The bridges in the *highway data set* were grouped into HAZUS bridge classes HBR-3, 5, 9 and 11, and damage frequency matrices were obtained for each of the classes. Then empirical fragility curves were obtained for each of these groups using logistic regression analysis.

The HAZUS fragility curves were compared both with observed data and the empirical curves for each of these classes. Figures 4-27 through 4-30 show the probability of being in different damage states as functions of peak ground acceleration for HAZUS bridge classes HBR-3, 5, 9 and 11. In these figures, the solid symbols show the probability of being in a damage state obtained from the observed damage and the lines with hollow symbols represent the fragility curves given in HAZUS. Figure 4-31 shows the comparisons for the exceedance probabilities and probabilities of being in a given damage state for class HBR5. Comparisons of HAZUS fragility curves with the empirical ones for the other classes are given in Appendix E. The observed damage data do not appear to agree well with the available ground motion-damage relationships in the majority of the cases investigated in this study. Similar to HBR5, the HAZUS fragility curves of other classes overestimated the exceedance probabilities for any given damage state. The curves for HBR5 overestimate the probability of being in major damage state at most of the PGA levels. They overestimate the probability of being in minor damage state at lower PGA levels and underestimate it at higher PGA levels for the given data set.

As discussed in Section 4.2, the PGA levels reported by USGS and WCFS vary considerably. Therefore, the observed damage was compared to the HAZUS fragility curves using both sets of ground motion data. Figure 4-32 compares the HAZUS fragility curves with the empirical fragility curves using the PGA values reported by WCFS [1995]. The difference between the predicted and the empirical exceedance probabilities is smaller when the WCFS PGA values are used. However, the HAZUS fragility curves still overestimate the exceedance probabilities for a given damage state at all PGA levels for all bridge classes.

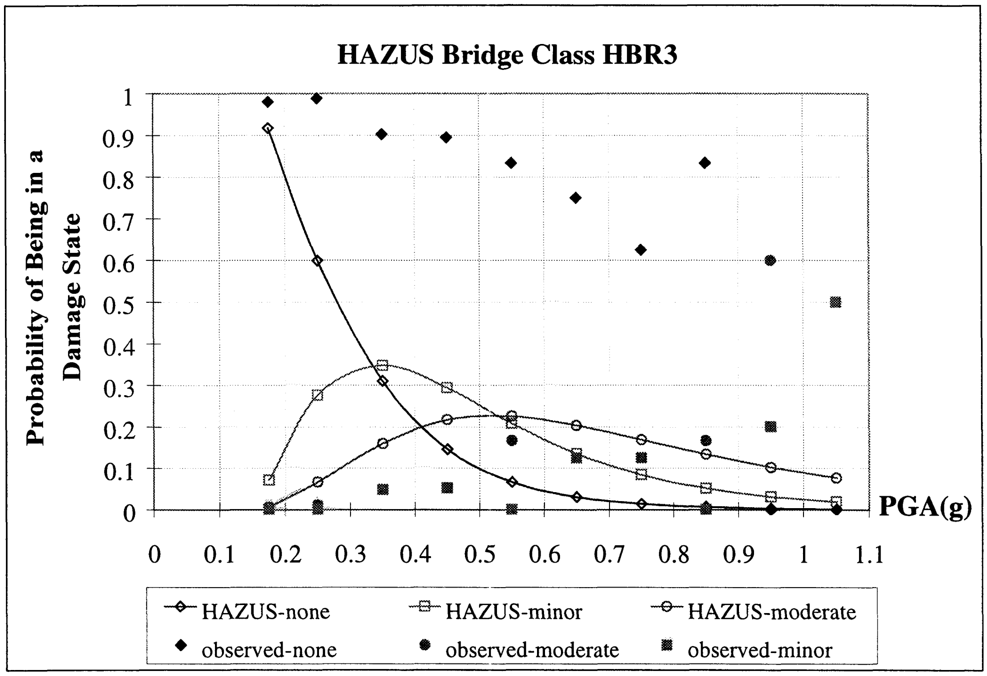


FIGURE 4-27.a Comparison of Damage Probabilities from Data to HAZUS Fragility Functions, No Damage, Minor Damage and Moderate Damage.

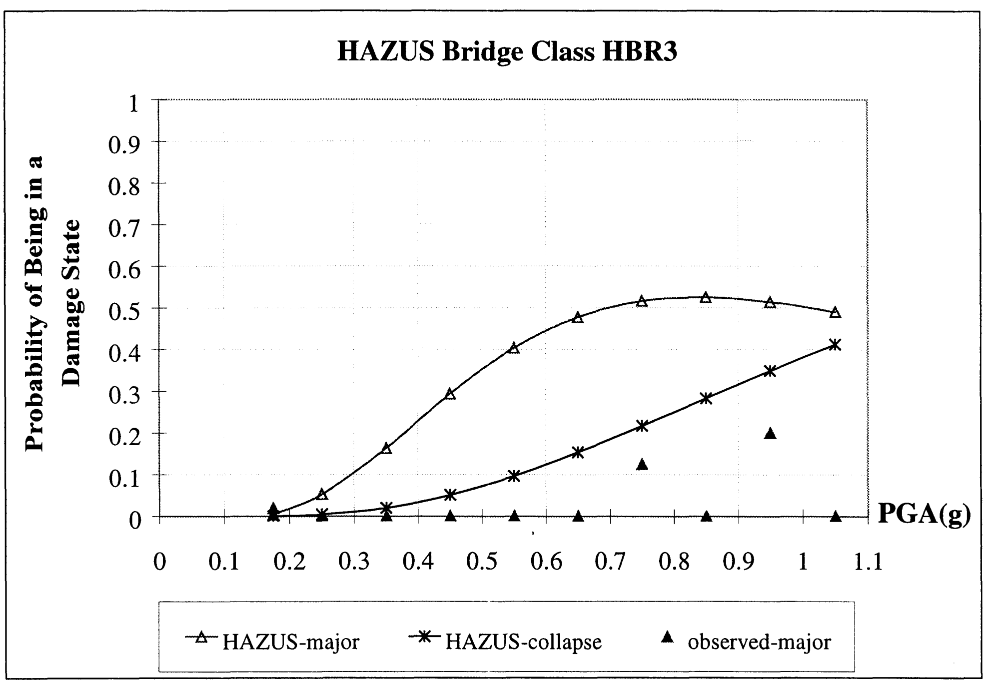


FIGURE 4-27.b Comparison of Damage Probabilities from Data to HAZUS Fragility Functions, Major damage and Collapse

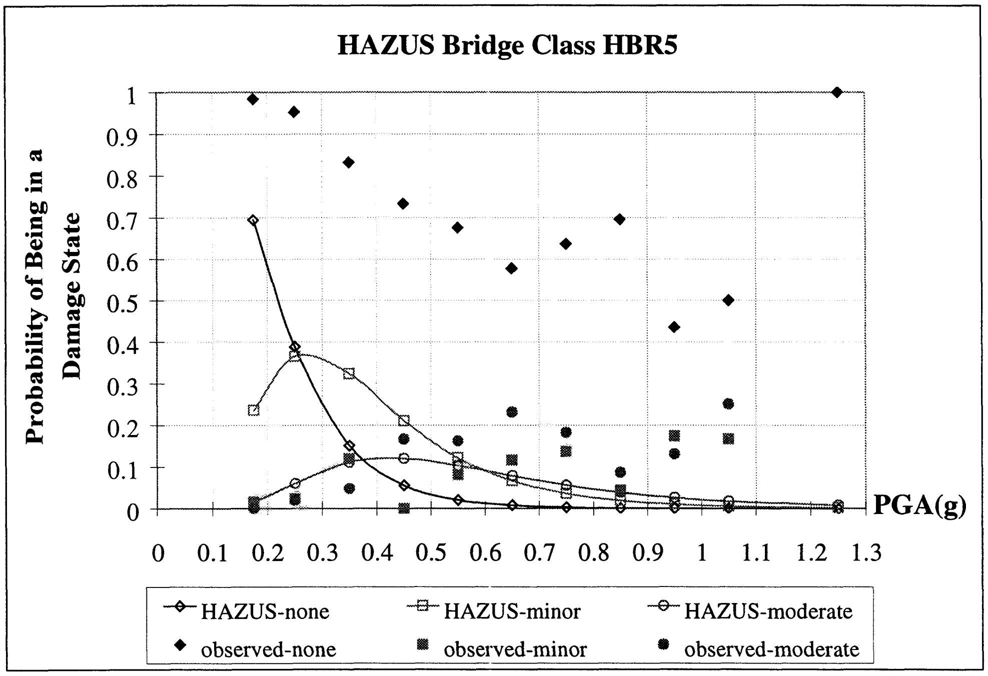


FIGURE 4-28.a Comparison of Damage Probabilities from Data to HAZUS Fragility Functions, No Damage, Minor Damage and Moderate Damage

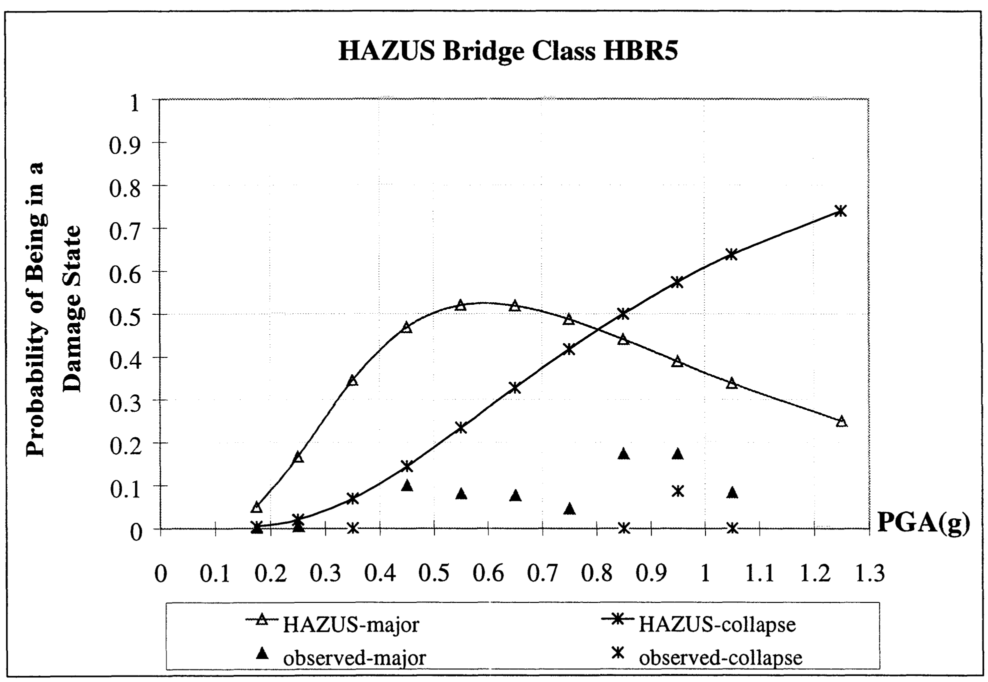


FIGURE 4-28.b Comparison of Damage Probabilities from Data to HAZUS Fragility Functions, Major Damage and Collapse

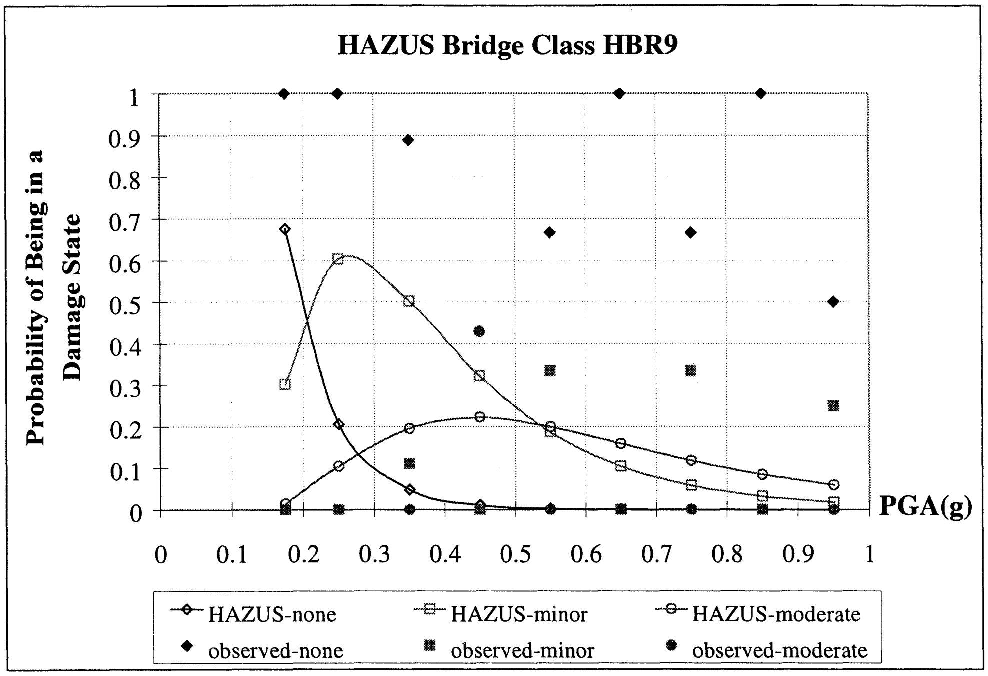


FIGURE 4-29.a Comparison of Damage Probabilities from Data to HAZUS Fragility Functions, No Damage, Minor Damage and Moderate Damage

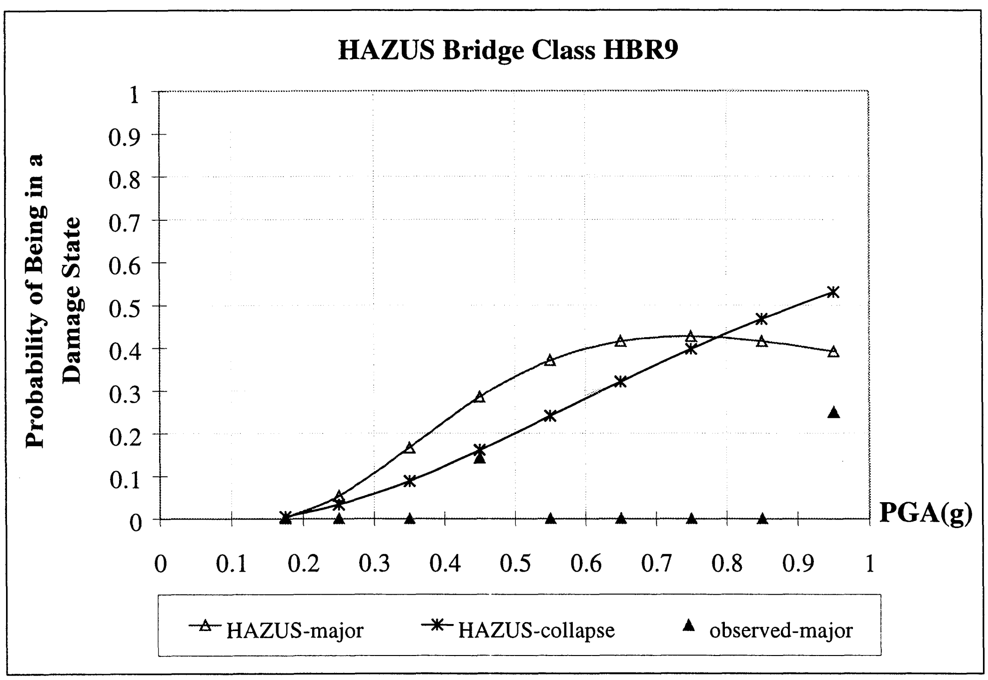


FIGURE 4-29.b Comparison of Damage Probabilities from Data to HAZUS Fragility Functions, Major Damage and Collapse

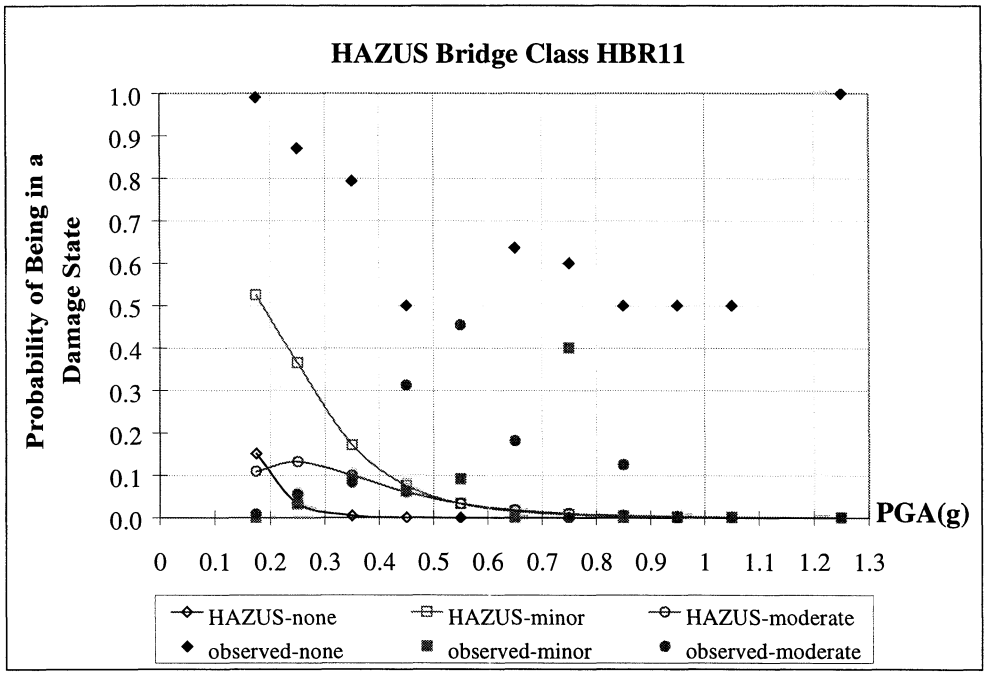


FIGURE 4-30.a Comparison of Damage Probabilities from Data to HAZUS Fragility Functions, No Damage, Minor Damage and Moderate Damage

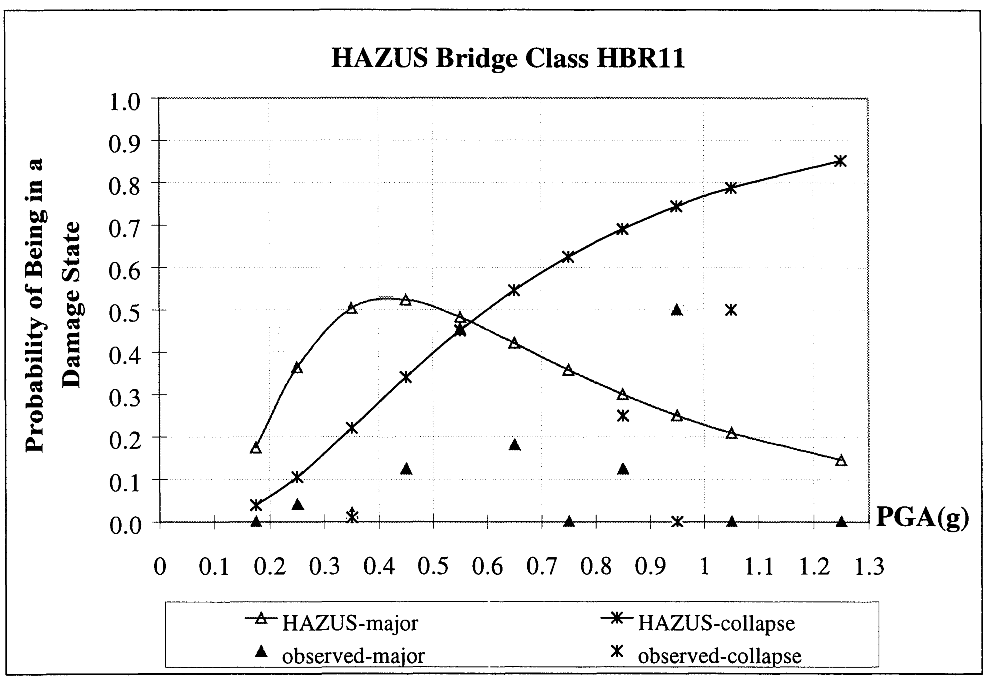
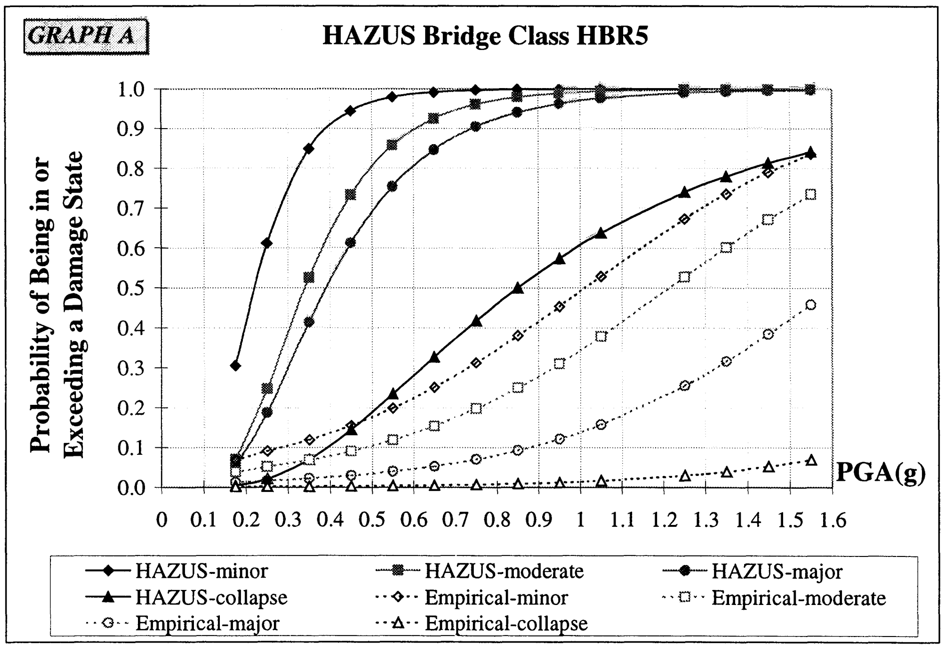


FIGURE 4-30.b Comparison of Damage Probabilities from Data to HAZUS Fragility Functions, Major Damage and Collapse

Number of bridges Chi-square with 3DF	539, 38, 39, 19, 2 7.0884, p = 0.0691		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
3.0228	0.3627	0.0001	20.5485



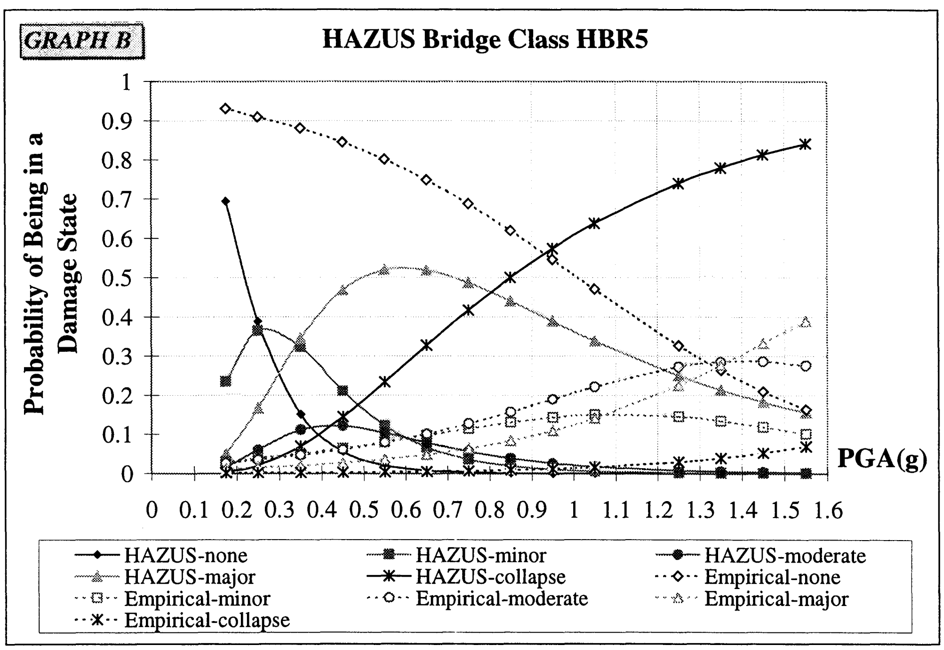
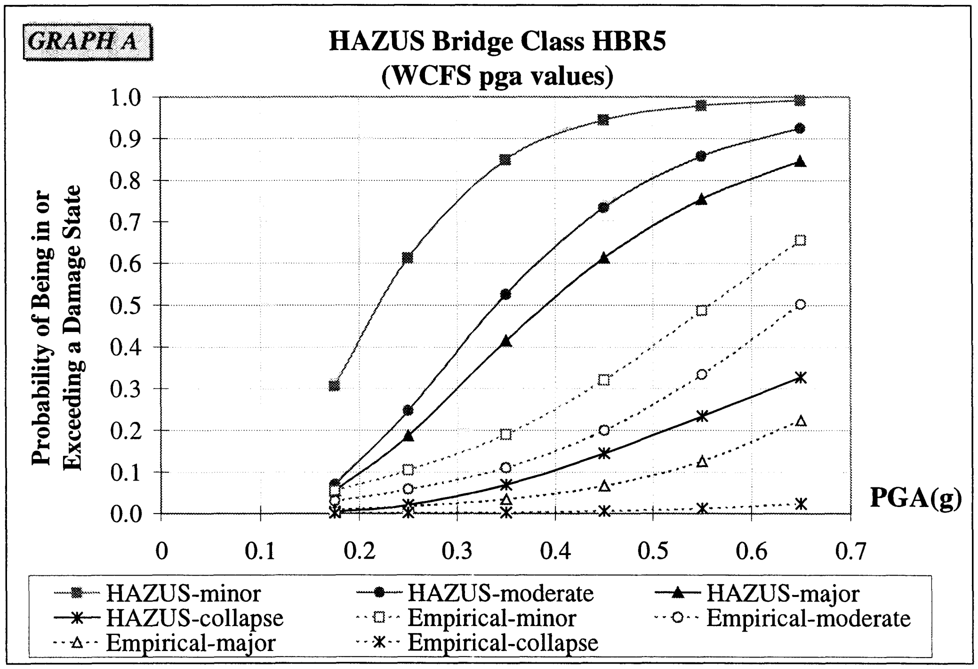


FIGURE 4-31 Comparison of Empirical and the HAZUS Fragility Curves for HAZUS Bridge Class HBR5, USGS PGA values

Number of bridges Chi-square with 3DF	463, 36, 39, 19, 2 7.3775, p = 0.0608		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
6.9848	0.7831	0.0001	999



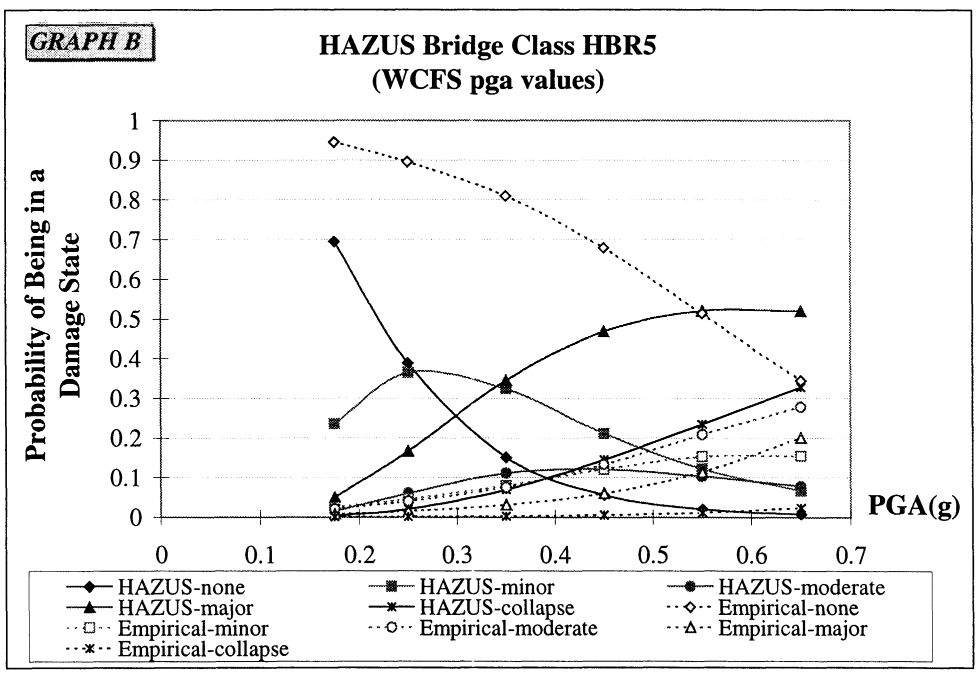


FIGURE 4-32 Comparison of Empirical and the HAZUS Fragility Curves for HAZUS Bridge Class HBR5, WCFS PGA values

4.4.3 Data Error in the Inventory

As discussed in Section 2.1, some of the attribute values for abutment type, column bent type and joint location were found to be incorrect in the database used in this study. For all the bridges in the Northridge earthquake, the SMS and BIRIS databases maintained by Caltrans were compared and attributes with discrepancies were identified. Fifteen percent of the abutment information was found to have errors and were corrected based on the as-built plans. Due to time constraints, only the error in abutment type attribute was corrected. In only a few cases (for 2 to 3 percent of the damaged bridge data set) were the values of the design year and skew attributes found to be incorrect. Statistical analyses were carried out to determine the potential effect of data error in the inventory on the correlation studies. The two main consequences of data error were the following:

- The total number of homogeneous bridges included in the *correlation data set* changed. This was due to the fact that bridges that were listed with heterogeneous abutment types in the SMS database were found to have homogeneous characteristics or vice versa.
- The number of bridges in a given bridge sub-category changed. Therefore, the probability of being in a damage state for a given bridge sub-category changed. This was also because of the changes in the abutment types during the correction process.

For example, for bridge sub-category C1M1 (multiple span continuous bridges with monolithic abutments and multiple column bents) the number of undamaged bridges was large. Thus, a change in the number of damaged bridges did not affect the probability values significantly. However, for the bridge sub-category C1M4 (multiple span discontinuous bridges with monolithic abutments and single column bents) the effect of data error on the results was more noticeable. Examples that illustrate the change in the results due to data correction are shown in tables 4.11.a and 4.11.b for all multiple span bridges with monolithic abutments. The shaded cells in table 4.11.b represent the cells with changes in number of bridges.

Unconditional and conditional fragility curves were obtained for multiple span bridges with monolithic abutments and non-monolithic abutments using both the corrected and uncorrected data. The effects of data error with respect to data size are illustrated in these fragility curves shown in figures 4-33 through 4-36. The dotted lines in the figures show the fragility curves obtained from the corrected data set. The solid lines show the fragility curves for the uncorrected data. The empirical fragility curves indicate that the effect of data error was more emphasized for smaller data sets. The change in the probabilities of being at or exceeding a damage state after the correction of data was higher for the conditional fragility curves as expected.

TABLE 4-11.a Number of Multiple Span Bridges with Monolithic Abutments for Different Damage States at Different PGA Values [USGS, 1994], - uncorrected data

	0.15 - 0.2	0.2 - 0.3	0.3 - 0.4	0.4 - 0.5	0.5 - 0.6	0.6 - 0.7	0.7 - 0.8	0.8 - 0.9	0.9 - 1.0	1.0 - 1.1	1.1 - 1.2	total
no	79	116	78	20	6	8	8	11	5	9	1	341
minor	0	3	9	1	6	2	1	0	5	0	1	28
moderate	0	0	3	5	6	2	0	3	4	0	0	23
major	0	1	0	3	1	1	1	0	4	0	0	11
collapse	0	0	0	0	0	0	0	2	0	0	0	2
total	79	120	90	29	19	13	10	16	18	9	2	405

TABLE 4-11.b Number of Multiple Span Bridges with Monolithic Abutments for Different Damage States at Different PGA Values [USGS, 1994], - corrected data

	.15 - .2	.2 - .3	.3 - .4	.4 - .5	.5 - .6	.6 - .7	.7 - .8	.8 - .9	.9 - 1.0	1.0 - 1.1	total
no	79	116	78	20	6	8	8	11	5	9	340
minor	0	3	6	1	5	2	1	0	4	1	23
moderate	0	1	3	4	6	2	0	4	4	0	24
major	0	1	0	3	2	1	0	0	3	0	10
collapse	0	0	0	0	0	0	0	2	1	0	3
total	79	121	87	28	19	13	9	17	17	10	400

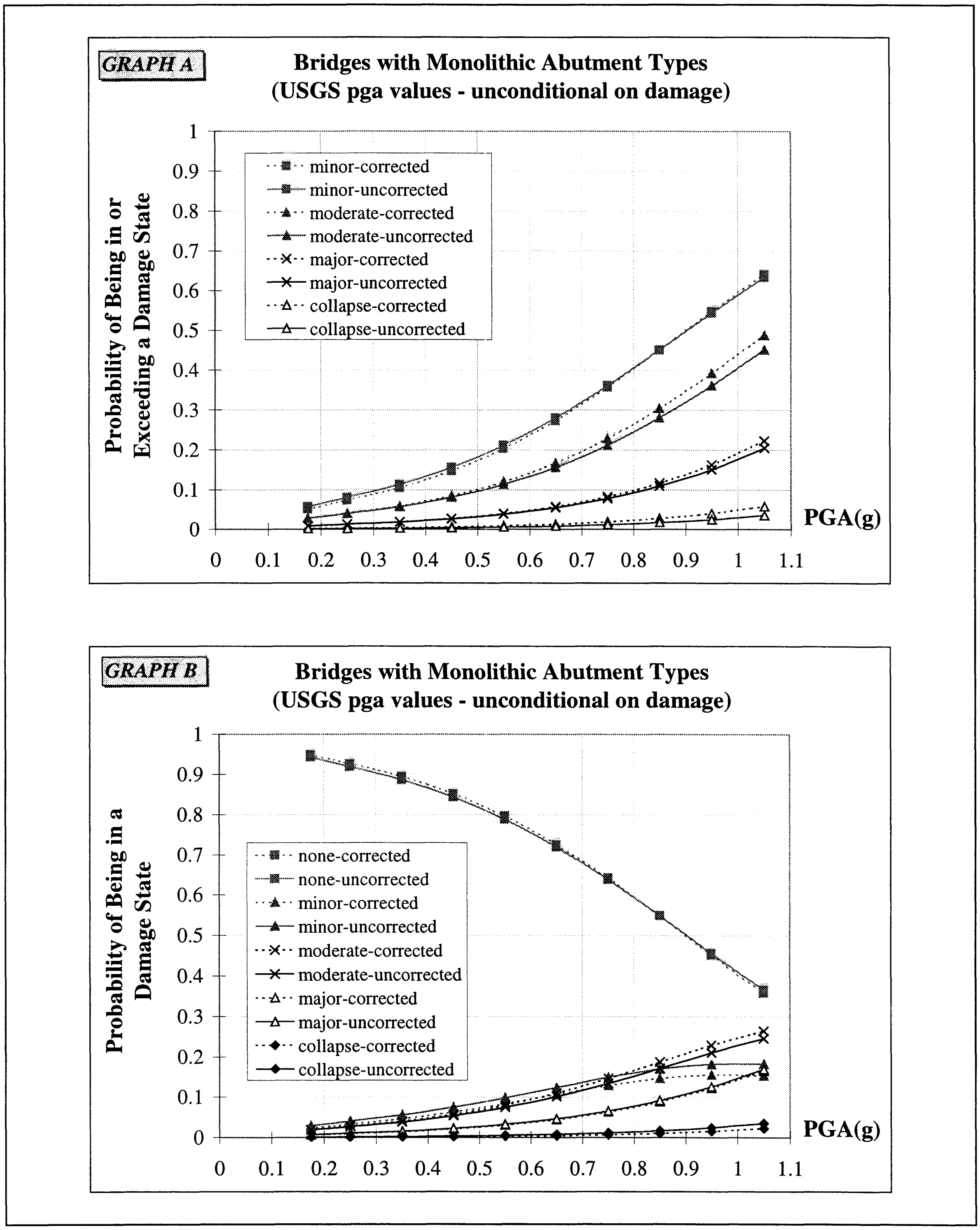


FIGURE 4-33 Comparison of Empirical Fragility Curves for Multiple Span Bridges with Monolithic Abutments based on Corrected and Uncorrected Data

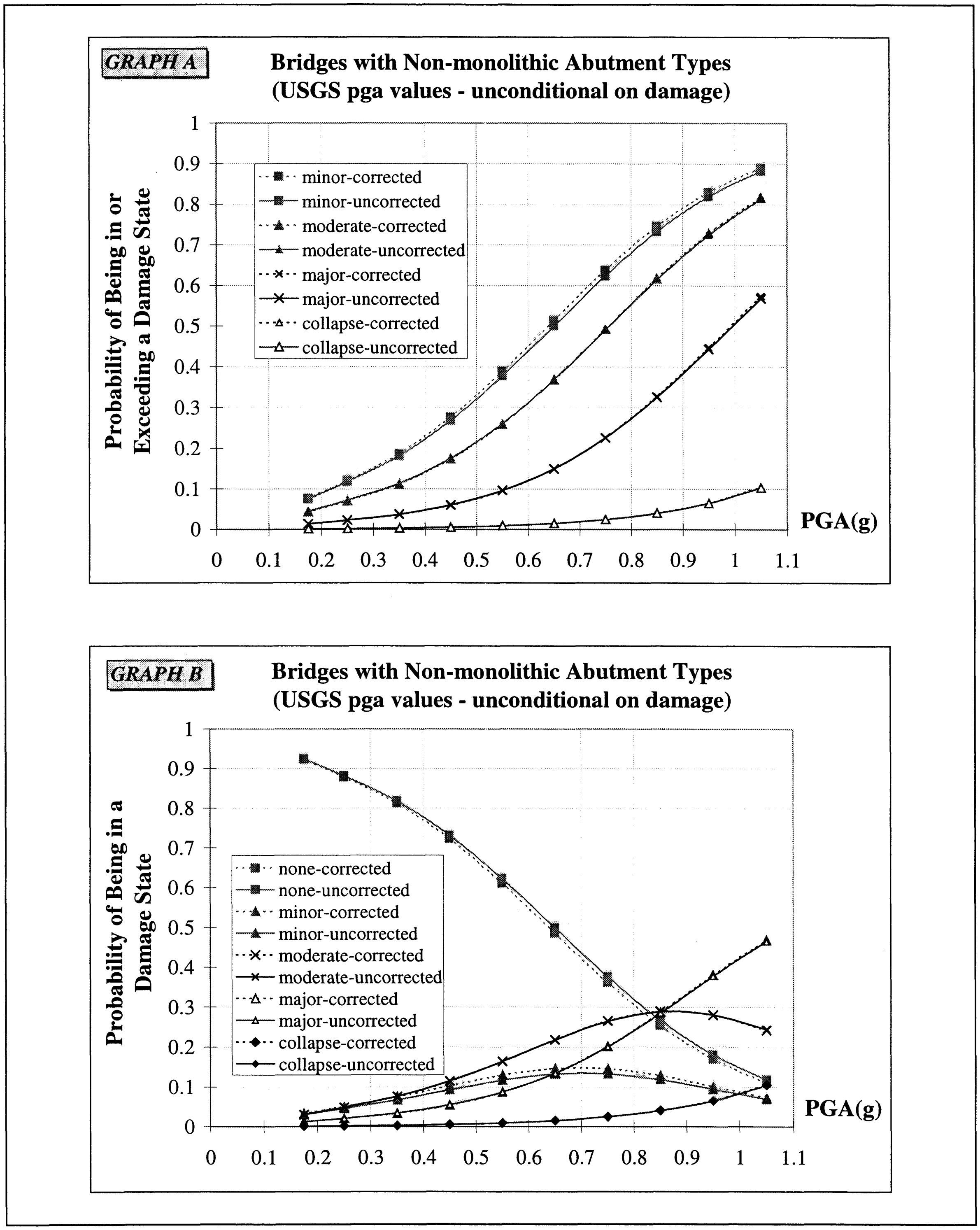


FIGURE 4-34 Comparison of Empirical Fragility Curves for Multiple Span Bridges with Non-monolithic Abutments based on Corrected and Uncorrected Data

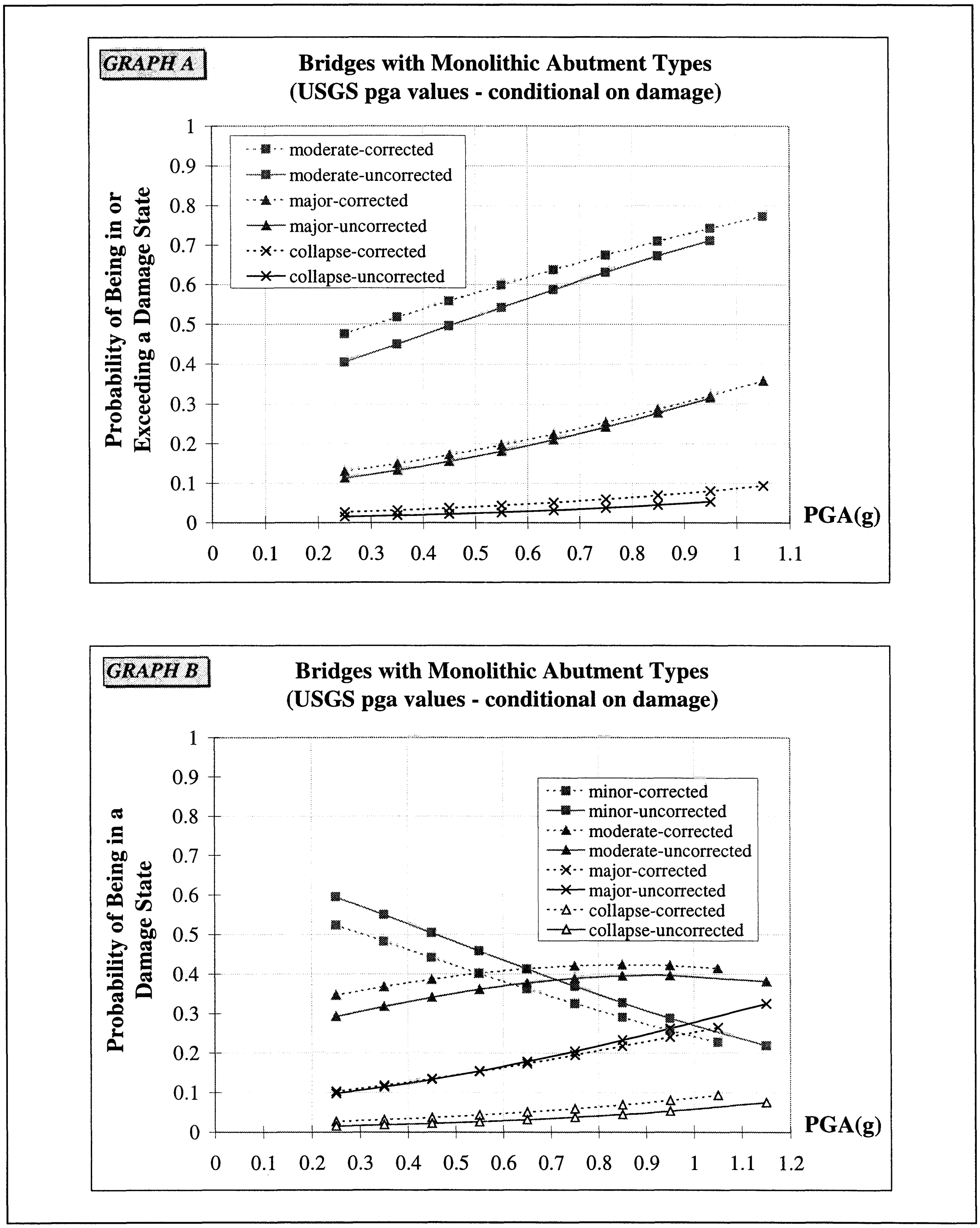


FIGURE 4-35 Comparison of Conditional Empirical Fragility Curves for Multiple Span Bridges with Monolithic Abutments based on Corrected and Uncorrected Data

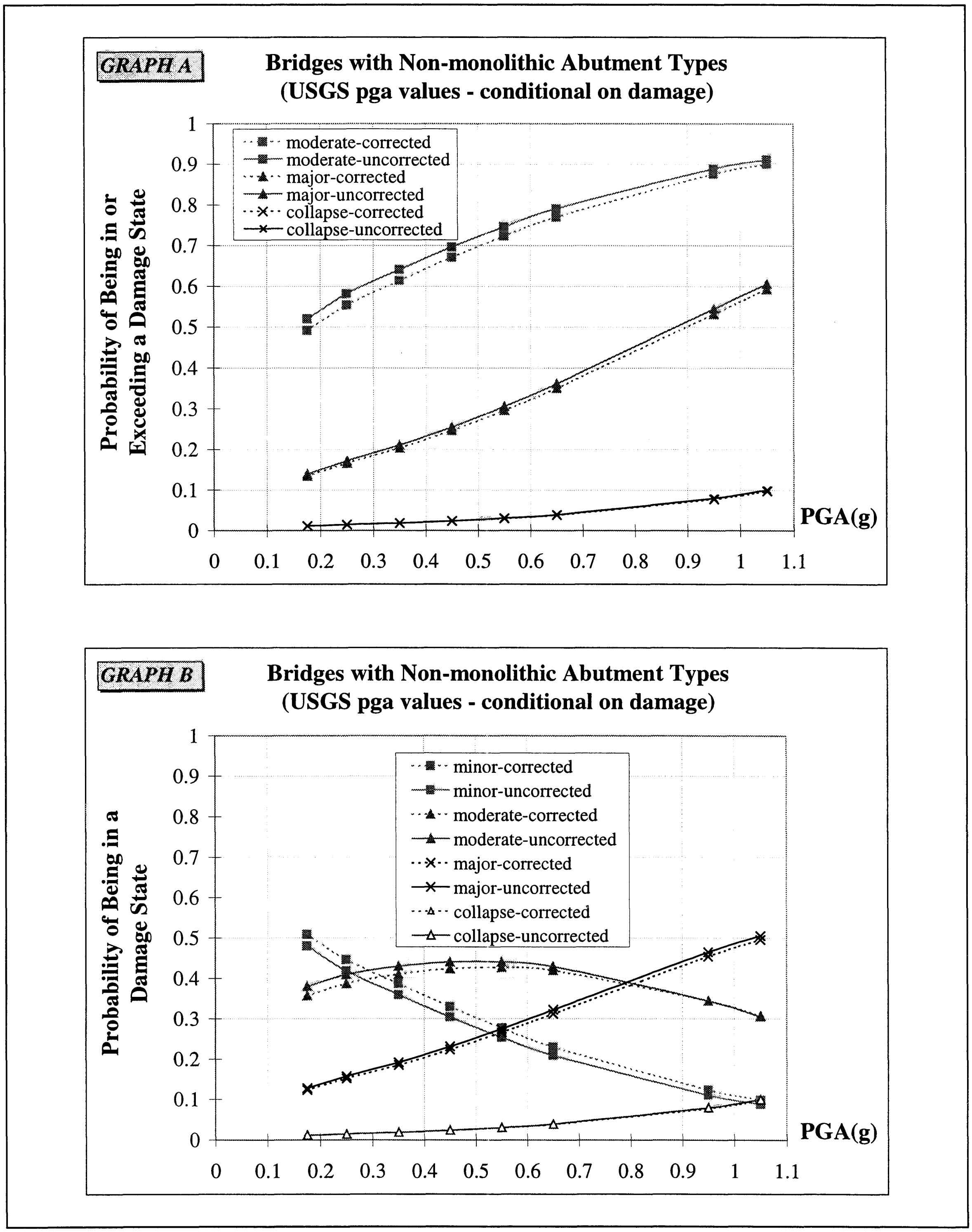


FIGURE 4-36 Comparison of Conditional Empirical Fragility Curves for Multiple Span Bridges with Non-monolithic Abutments based on Corrected and Uncorrected Data

4.5 ESTIMATED REPAIR COST

4.5.1 Description of Repair Cost Data

Estimated repair cost data were obtained from the supplementary bridge damage reports compiled by Caltrans following the Northridge earthquake. A total of about $190 million was reported as the repair cost in these reports. The total repair cost for the six collapsed bridges corresponds to seventy five percent of the reported repair cost of all damaged bridges. Table 4-12 shows the estimated repair cost of damaged bridges by damage state.

The distribution of bridges by estimated repair cost and damage state is shown in figure 4-37. Only 77 of the 160 damaged bridges in the *correlation data set* were reported to have some repair cost. No repair cost was reported for the remaining 83 bridges in the *correlation data set.* Repair costs were reported for 12 bridges, which were not reported as damaged. Figure 4-38 shows the damage state distribution for bridges with no reported repair cost.

TABLE 4-12 Distribution of Estimated Repair Cost by Damage State

Damage State	Estimated Repair Cost ($)
Collapse	152,845,000
Major	30,876,272
Moderate	5,660,094
Minor	536,640

An extensive database on repair cost was compiled from the supplementary bridge reports obtained from Caltrans [1994b]. The database included total estimated repair cost and more detailed information on repair work and cost for 130 bridges in Los Angeles and Ventura Counties. In addition, repair costs for various components were gathered. Data on component repair costs were not reported for all bridges. For forty-three of the 77 bridges, a breakdown of repair cost by component was provided while for the others only the total repair cost and a general description of the repairs were given. The forty-three bridges with detailed component repair cost data were further analyzed. Figure 4-39 shows the distribution of repair cost by bridge components for different damage states. The collapsed bridges were not included in this

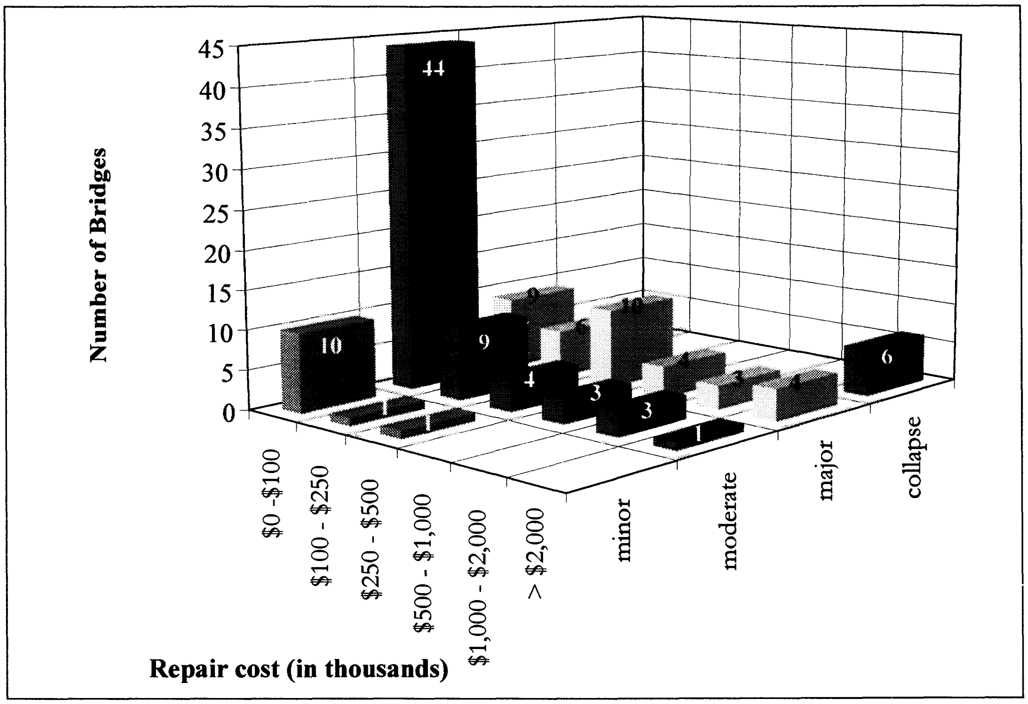


FIGURE 4-37 Distribution of All Damaged Bridges by Repair Cost and Damage State

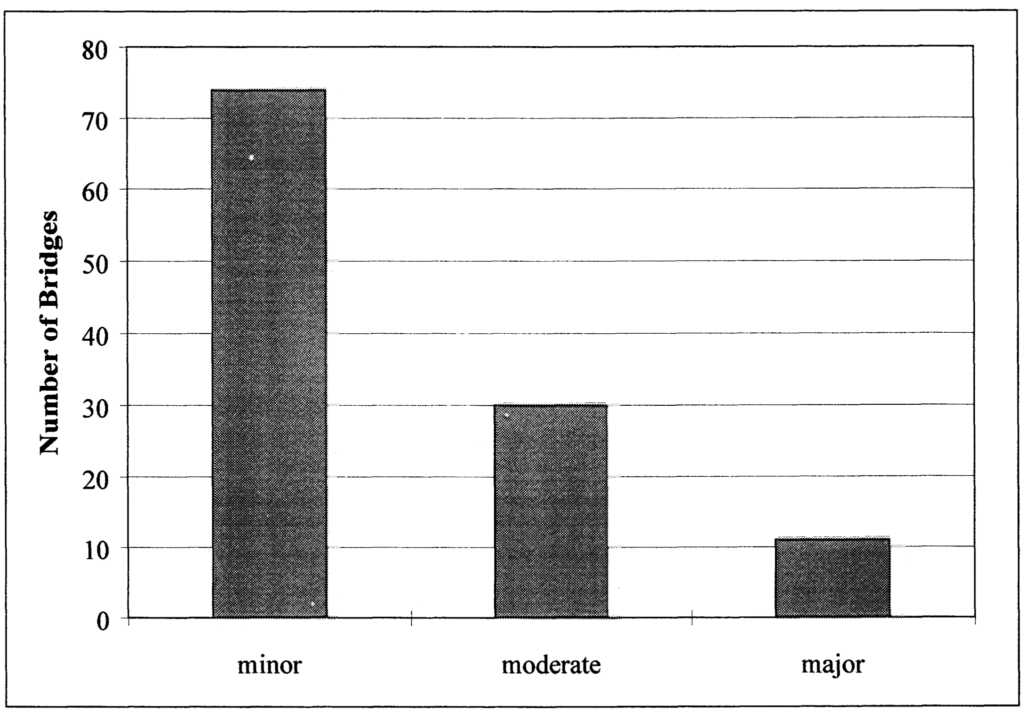


FIGURE 4-38 Number of Damaged Bridges with No Reported Repair Costs

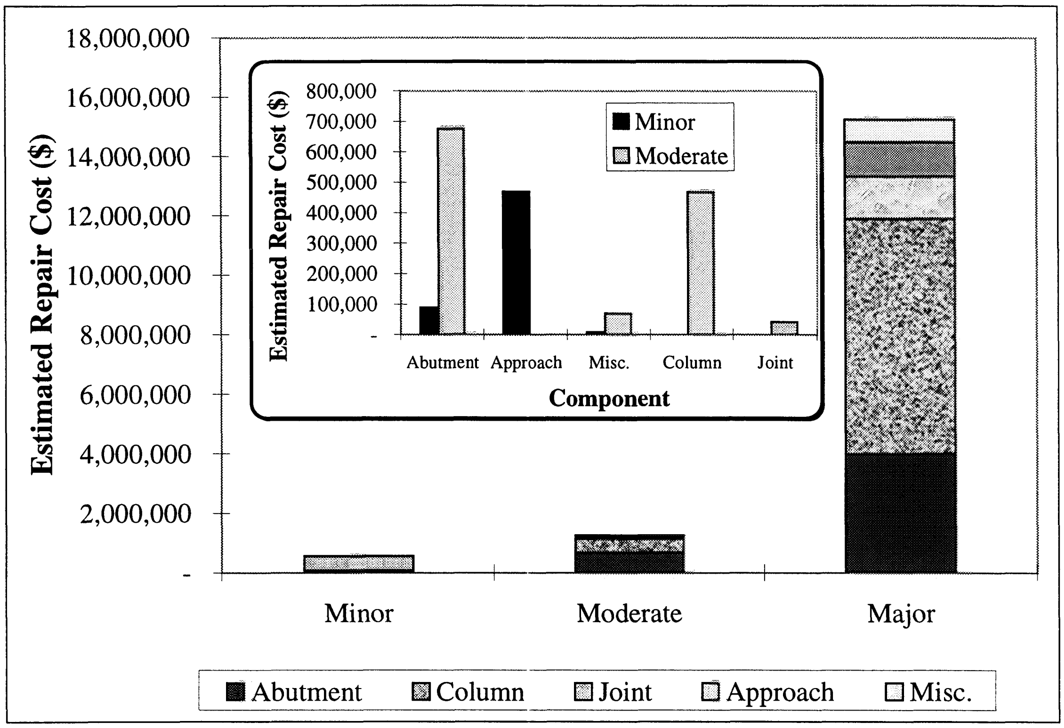


FIGURE 4-39 Distribution of Estimated Repair Cost by Bridge Components for Different Damage States

figure since a detailed description of repair by component type was not available for those bridges. According to figure 4-39, most of the repair costs for bridges with *minor* damage were for approach settlement. Bridges with *minor* damage did not suffer damage to the columns or joints. While approach settlement was not reported for bridges that suffered *moderate* damage, the damage to abutments was the most significant type of damage for these bridges. Although abutment damage contributed significantly to repair cost for bridges with *major* damage, the highest repair cost for bridges with *major* damage was due to column damage. Damage to joints was observed mostly in bridges with *major* damage. The average repair cost for bridges that suffered *major* damage (about $500,000/bridge) was about 5 times that for bridges with moderate damage (about 110,000/bridge). Under *miscellaneous* repairs, repair to railings, curbs, sidewalks and electrical conduits were included.

In order to investigate the distribution of repair cost by structural type, bridges were also grouped by the bridge sub-categories given in table 2-1. Figure 4-40 shows the distribution of estimated repair cost by bridge sub-categories. Only nine of the sub-categories were included in this figure. Data were not available for the other sub-categories. The number of bridges in each bridge sub-category ranged from 2 to 13 as shown in figure 4-40. Costs due to traffic closures were not included in the analysis. Figure 4-40 shows that bridge sub-category C1M10, identified as the most vulnerable, accounted for about 40 percent of the total repair cost. For bridge sub-categories C1M7 through C1M10, which had non-monolithic type abutments, most of the repair cost was due to abutment damage. Estimated repair cost for joints was noticeable for sub-categories with discontinuous spans, i.e., sub-categories C1M4, C1M7 and C1M10. Bridge sub-category C1M1 was defined by Basöz and Kiremidjian [1996] as the least vulnerable multiple span bridge sub-categories for concrete bridges. In figure 4-40, the repair cost for 9 bridges in this class was shown to be one of the smallest repair cost values.

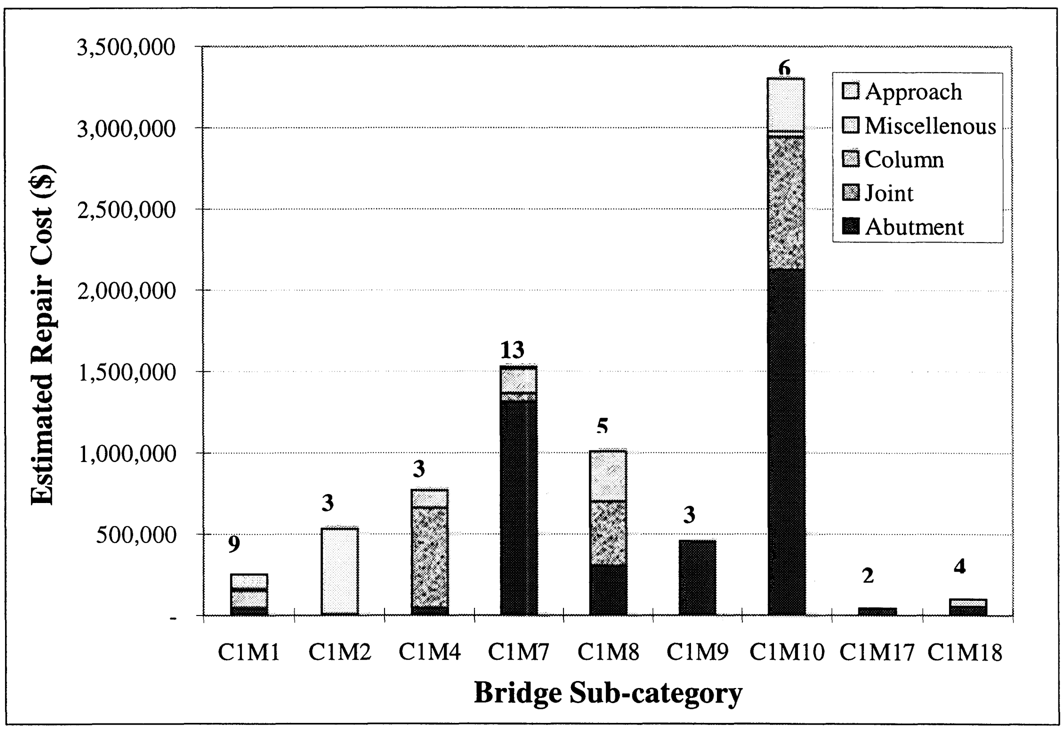


FIGURE 4-40 Distribution of Estimated Repair Cost by Bridge Components for Different Bridge Sub-categories

Estimates on costs associated with traffic closure were reported for only 13 bridges. Therefore, due to lack of data, estimated repair costs used in this study did not include costs due to traffic control. Figures 4-41 and 4-42 show the estimated costs due to traffic closure and its ratio to the total repair cost for bridges with *moderate* and *major* damage, respectively. The ratio of traffic closure cost to total estimated repair cost showed significant variation for these thirteen bridges while the high percentages were associated with low repair costs.

The repair cost ratio was defined as the ratio of repair cost to replacement cost. Seven repair cost ratio intervals were used in the correlation analyses: 0; 0-10%; 10-20%; 20-30%; 30-40%; 40-50%; >50%. According to ATC-13, one-to-one correspondence exists between the repair cost ratio and the damage states. Furthermore, repair cost ratios less than 1 percent are expected to correspond to only *minor* damage state. The repair cost ratios for different damage states observed in the Northridge earthquake are plotted in figure 4-41. As illustrated in this figure, in the Northridge earthquake low repair cost ratios less than 1 percent were observed for *minor*, *moderate* and *major* damage states. However, the mean repair cost ratios do increase with severity of damage states.

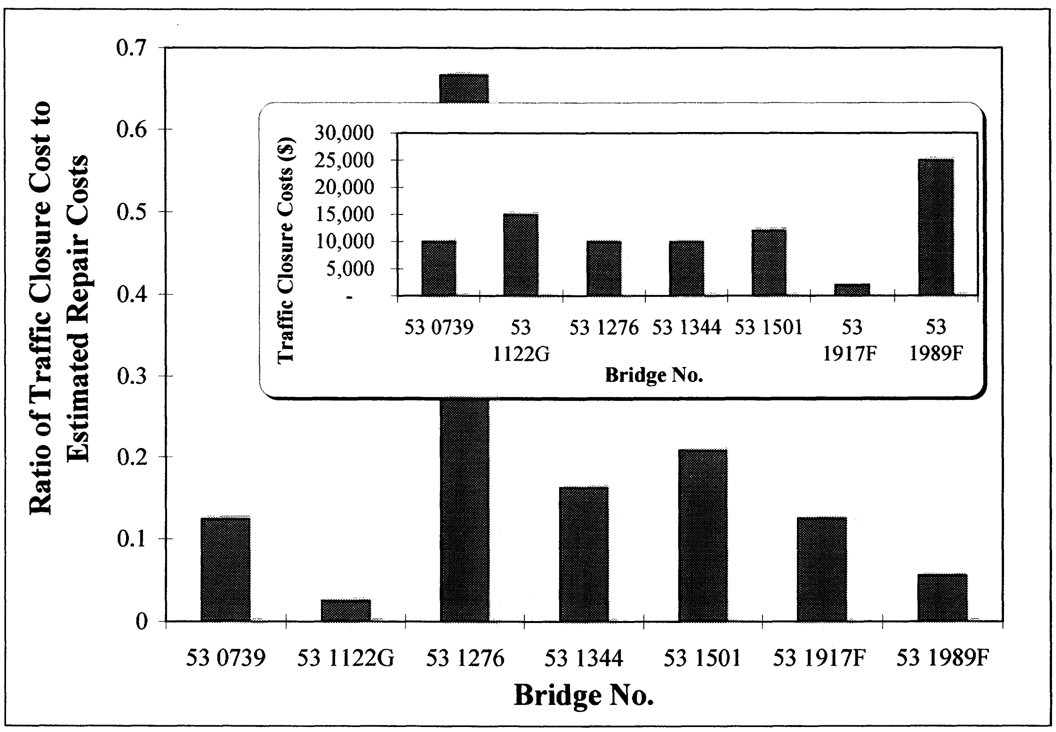


FIGURE 4-41 Ratio of Traffic Closure Cost to Estimated Repair Cost - Moderate Damage

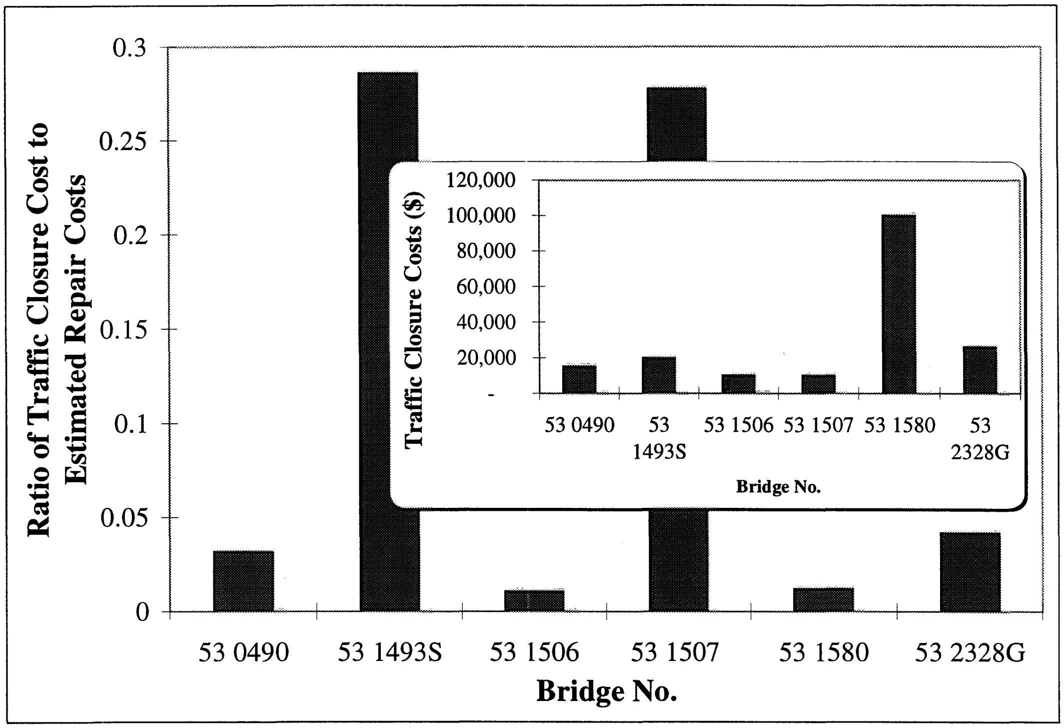


FIGURE 4-42 Ratio of Traffic Closure Cost to Estimated Repair Cost - Major Damage

The repair cost ratio was defined as the ratio of repair cost to replacement cost. The replacement cost was obtained as the product of the unit cost per square feet ($\$90/ft^2$) by bridge width and length. Seven repair cost ratio intervals were used in the correlation analyses: 0; 0-10%; 10-20%; 20-30%; 30-40%; 40-50%; >50%. According to ATC-13, one-to-one correspondence exists between the repair cost ratio and the damage states. Furthermore, repair cost ratios less than 1 percent are expected to correspond to only *minor* damage state. The repair cost ratios for different damage states observed in the Northridge earthquake are plotted in figure 4-43. As illustrated in this figure, in the Northridge earthquake low repair cost ratios less than 1 percent were observed for *minor*, *moderate* and *major* damage states. However, the mean repair cost ratios do increase with severity of damage states.

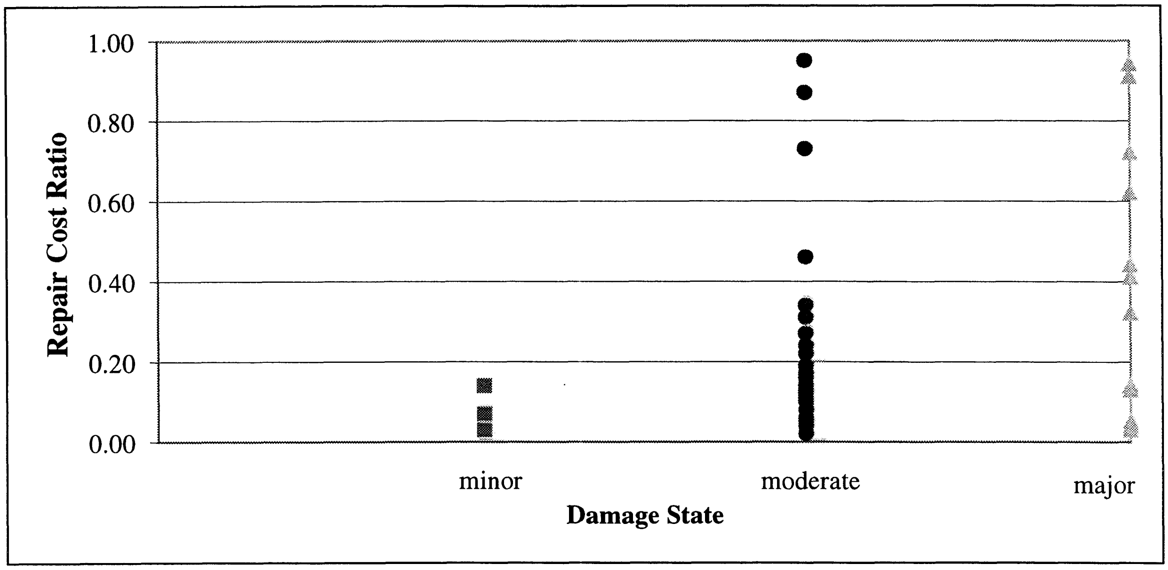


FIGURE 4-43 Distribution of Repair Cost Ratio by Damage State

4.5.2 Correlation Studies for Repair Cost Ratio

Similar to ground motion-damage relationships, empirical ground motion-repair cost ratio relationships were developed. For most of the bridge sub-categories listed in table 2-1, data were available for only two repair cost ratio levels: 0% and 0.1 to 10%. The empirical fragility curves for ground motion-repair cost ratios are presented in the Appendices listed in table 4-13. Example fragility curves listed in table 4-14 are presented here. In figures 4-44 to 4-48 two graphs are presented: (*i*) ***GRAPH A*** shows the probability of being in or exceeding a repair cost ratio at different PGA levels, and (*ii*) ***GRAPH B*** shows the probability of being in a repair cost ratio at different PGA levels. The ratio of bridges in each repair cost ratio interval, and statistical significance test results are presented in these figures. The number of bridges in each repair cost ratio interval, i.e., 0; 0-10%; 10-20%; 20-30%; 30-40%; 40-50%; >50%, is listed.

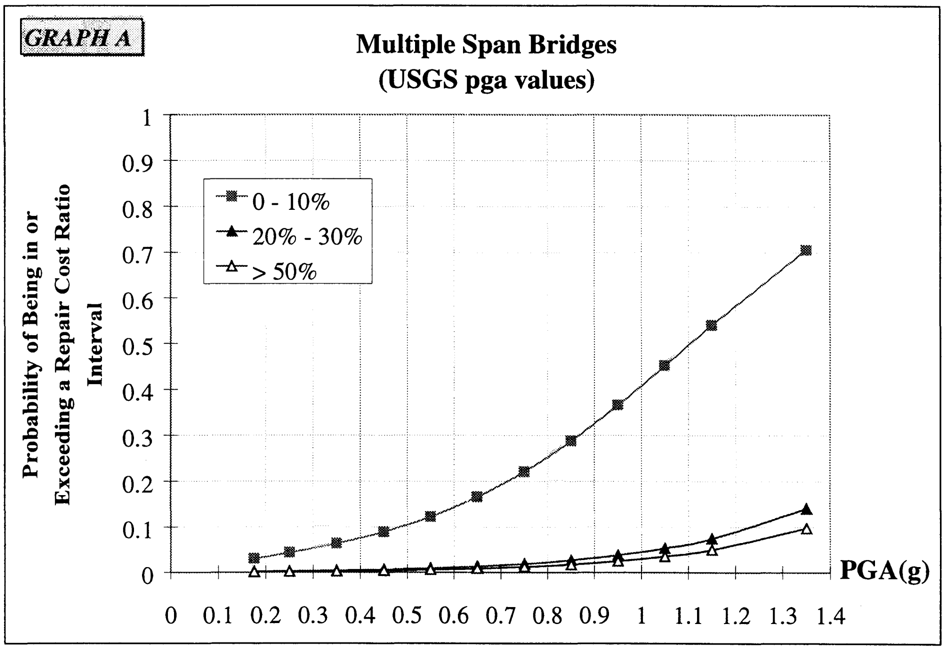
TABLE 4-13 Characteristics of Data Groups Used in Correlation Analyses

Dependent Variable	Independent Variable	Data Set Grouped by	Results are Presented in
repair cost ratio	PGA values reported by USGS [1994]	• number of spans • bridge sub-categories given in table 2-1 • design year	Appendix G
	PGA values reported by WCFS [1995]	• number of spans • bridge sub-categories given in table 2-1 • design year	Appendix H

TABLE 4-14 Description of Example Fragility Curves for Repair Cost Ratio

Dependent Variable	Independent Variable	Data Set Grouped by	Shown in figure
repair cost ratio	PGA values reported by USGS [1994]	• multiple span bridges	4-44
		• single span bridges	4-45
		• single span bridges with monolithic abutment types (C1S1)	4-46
		• multiple span bridges with monolithic abutment type and multiple columns/bent (C1M1)	4-47
		• bridges built between 1940 and 1971	4-48

Number of bridges Chi-square with 2 DF	777, 65, 0, 2, 0, 0, 4 22.306; p= 0.0001		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
3.6441	0.4301	0.0001	38.248



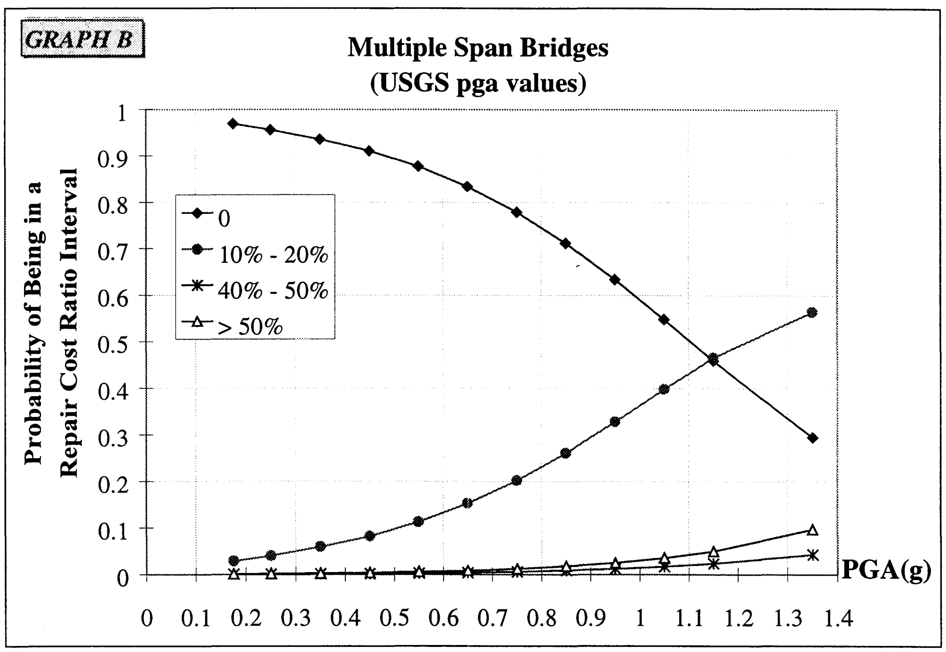
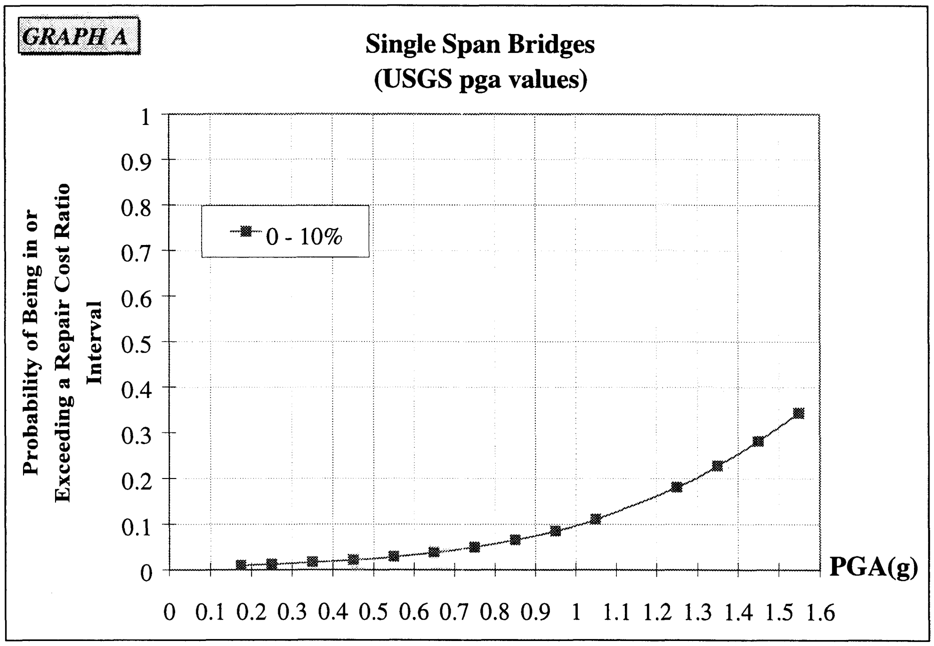


FIGURE 4-44 Empirical Fragility Curves for Multiple Span Bridges, Repair Cost Ratio

Number of bridges **Chi-square**	315, 9, 0, 0, 0, 0, 0 N/A		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
2.9131	0.8059	0.0003	18.414



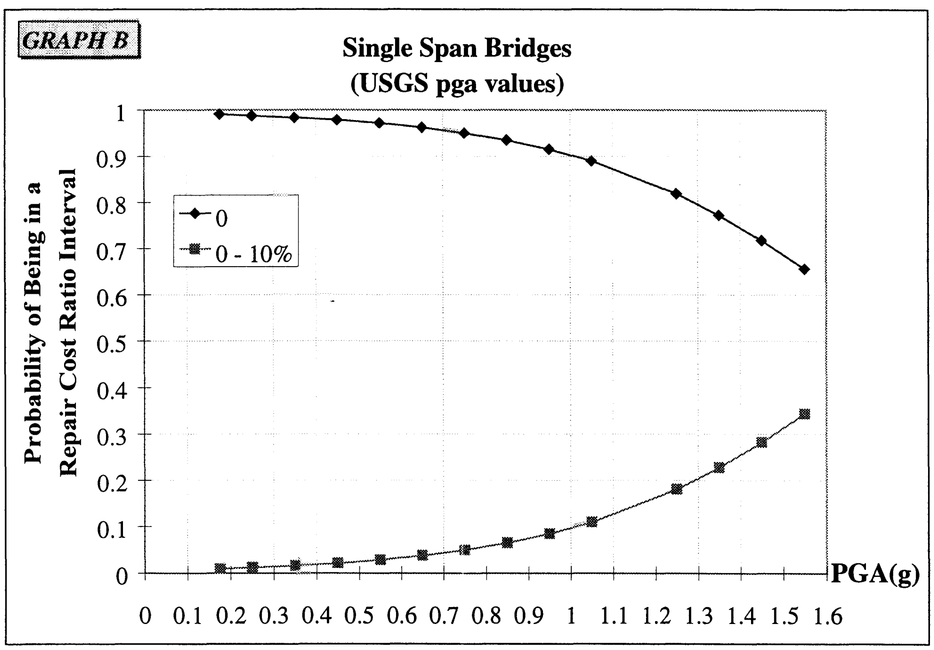
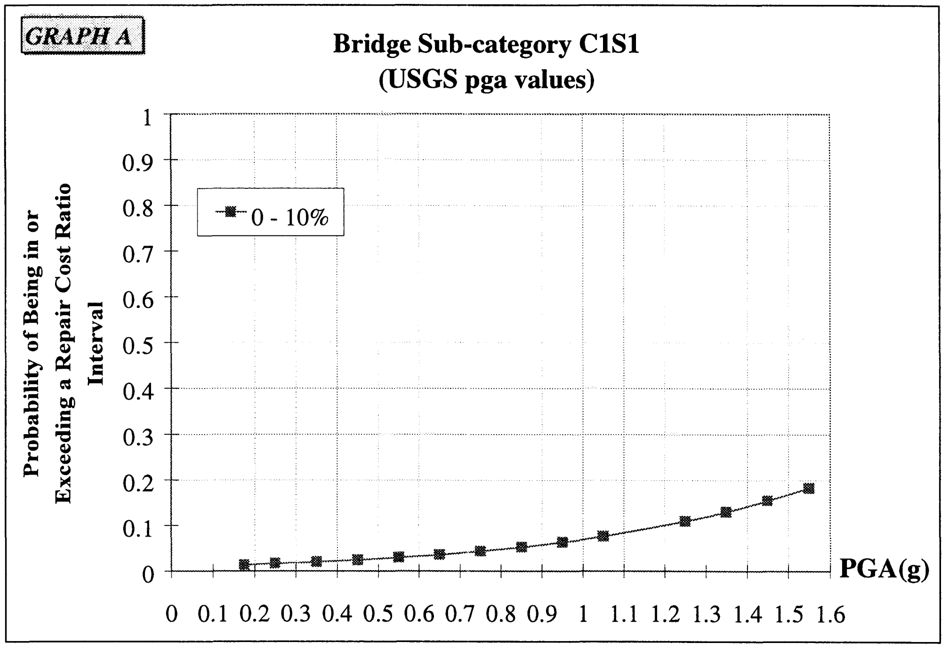


FIGURE 4-45 Empirical Fragility Curves for Single Span Bridges, Repair Cost Ratio

Number of bridges Chi-square	199, 6, 0, 0, 0, 0, 0 N/A		
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
1.993	0.9765	0.0413	7.337



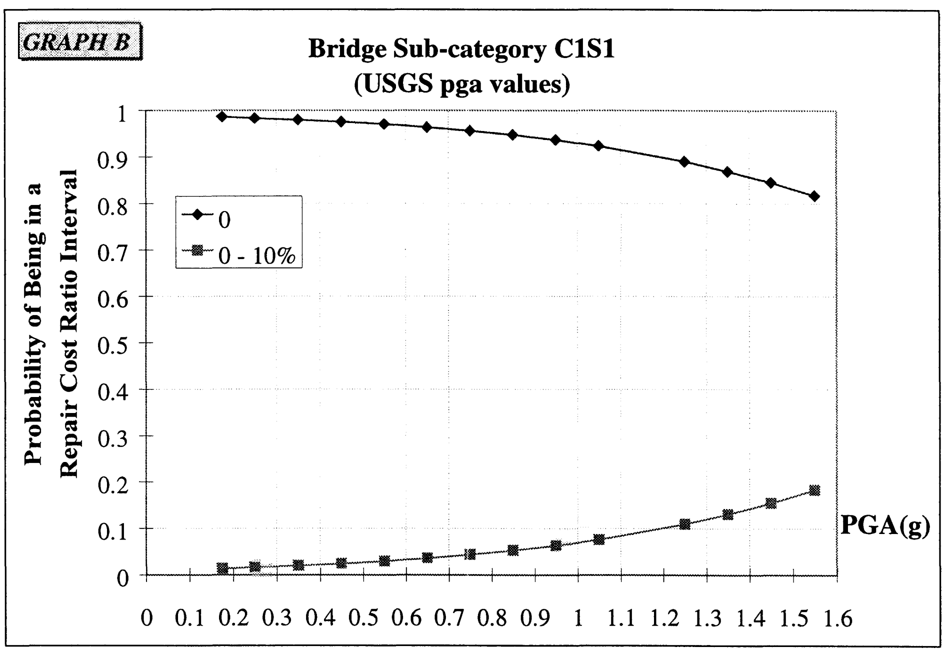
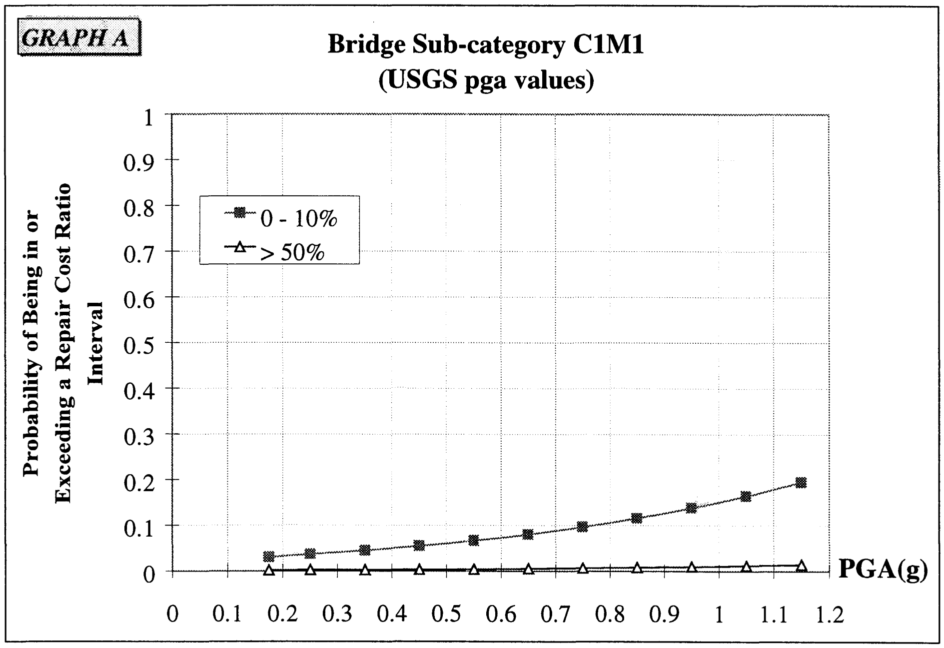


FIGURE 4-46 Empirical Fragility Curves for Single Span Bridges with Monolithic Type Abutments (Sub-Category C1S1), Repair Cost Ratio

Number of bridges Chi-square with 1DF	248, 14, 0, 0, 0, 0, 1 4.2087; p = 0.0402		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
2.0366	0.8679	0.019	7.666



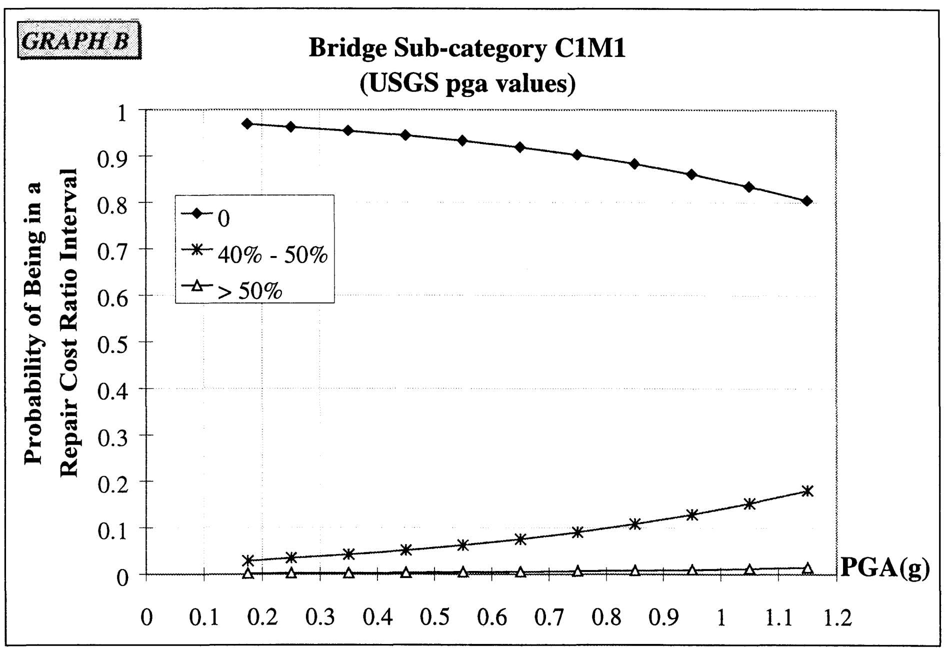
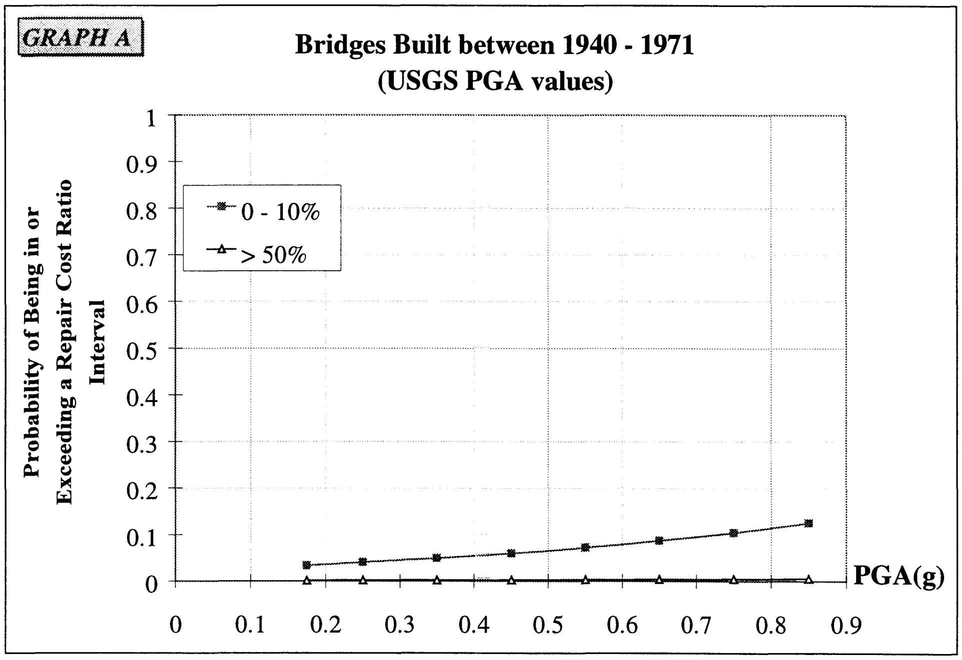


FIGURE 4-47 Empirical Fragility Curves for Multiple Span Bridges with Monolithic Type Abutments, Continuous Spans and Multiple Column Bents (Sub-category C1M1), Repair Cost Ratio

Number of bridges Chi-square with 2DF	743, 41, 0, 2, 0, 0, 2 346.4903; p = 0.0001		
Parameter Estimate	**Standard Error**	**Prob. > Chi-square**	**Odds Ratio**
2.0205	0.4602	0.0001	7.542



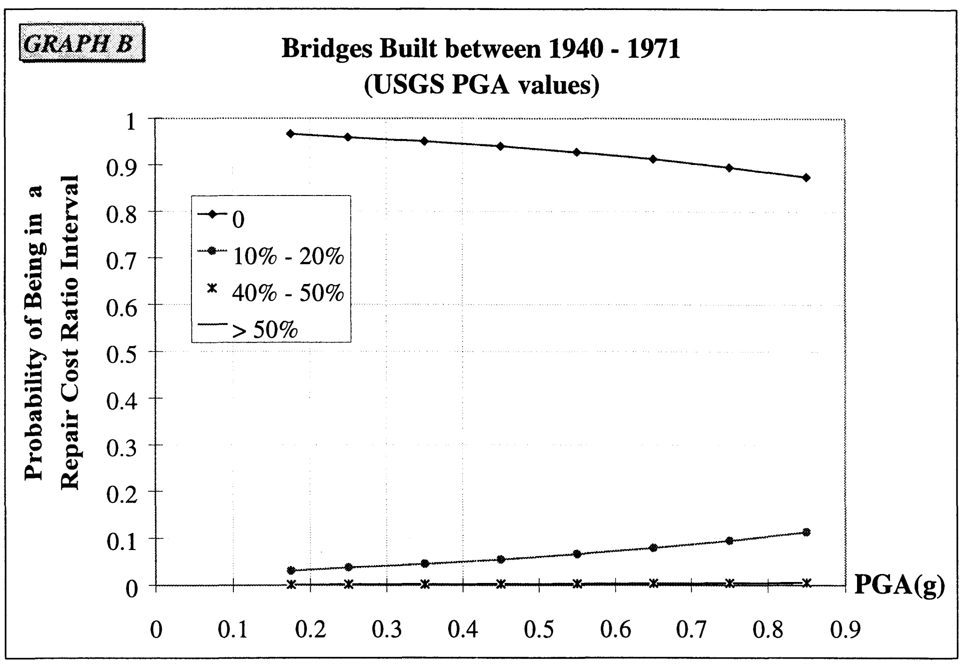


FIGURE 4-48 Empirical Fragility Curves for Bridges Built between 1940 and 1971

4.5.3 Comparison of ATC-13 Damage Probability Matrices and Repair Cost Ratio Data from the Northridge Earthquake

In ATC-13 [1985], ground motion-damage relationships are expressed in terms of damage probability matrices (DPMs). A damage probability matrix gives the probability of being in a damage state at a given MMI level. Damage states in ATC-13 are based on damage factor, which is defined as the ratio of repair cost of a structure to its replacement value. The damage states in ATC-13 are related to ranges of damage factor as listed in Table 4-15, where damage factor is defined as the ratio of repair cost of a structure to its replacement value. In order to distinguish between the definitions of damage states used earlier in the report and those used in ATC-13, the ATC-13 damage states are referred as the repair cost ratio states. Thus, the damage states listed in Table 4-15 also refer to the repair cost ratio states.

TABLE 4-15 ATC-13 Damage States and Damage Factors

Repair Cost Ratio State (Damage State – ATC-13)	Damage Factor (%)
None	0
Slight	0-1
Light	1-10
Moderate	10-30
Heavy	30-60
Major	60-100
Destroyed	100

The ATC-13 DPMs were compared to the repair cost ratio of the damaged bridges that were affected in the Northridge earthquake. The isoseismal map by Dewey et al. [1994] was used to obtain the MMI levels at each bridge site. According to this map, the maximum MMI level experienced at any bridge site was IX. Repair cost ratios corresponding to heavy and major damage states defined by ATC-13 were not observed in the Northridge earthquake data set.

Figures 4-49 and 4-50 show the probabilities of being in a given damage state at different MMI levels for ATC bridge classes 24 and 25, respectively. The results showed that the ATC-13 DPMs agree quite well with the observed damage and the repair cost ratios for low MMI values. The results show that the ATC-13 DPMs overestimate the expected repair cost ratio for both classes, especially for moderate and collapse states. The ATC-13 DPMs provide a non-zero probability for only two repair cost ratio states for most of the MMI levels. In contrast, for several MMI levels damage was observed in more than two or three repair cost ratio states. For example according to ATC-13, the probabilities of being in light and moderate damage states at MMI level IX are respectively 56.5 and 43.5 percent for bridges in class 25. However, non-zero probabilities were obtained for repair cost ratio states none, slight, moderate and destroyed, as shown in Figure 4-50.e based on the damage data.

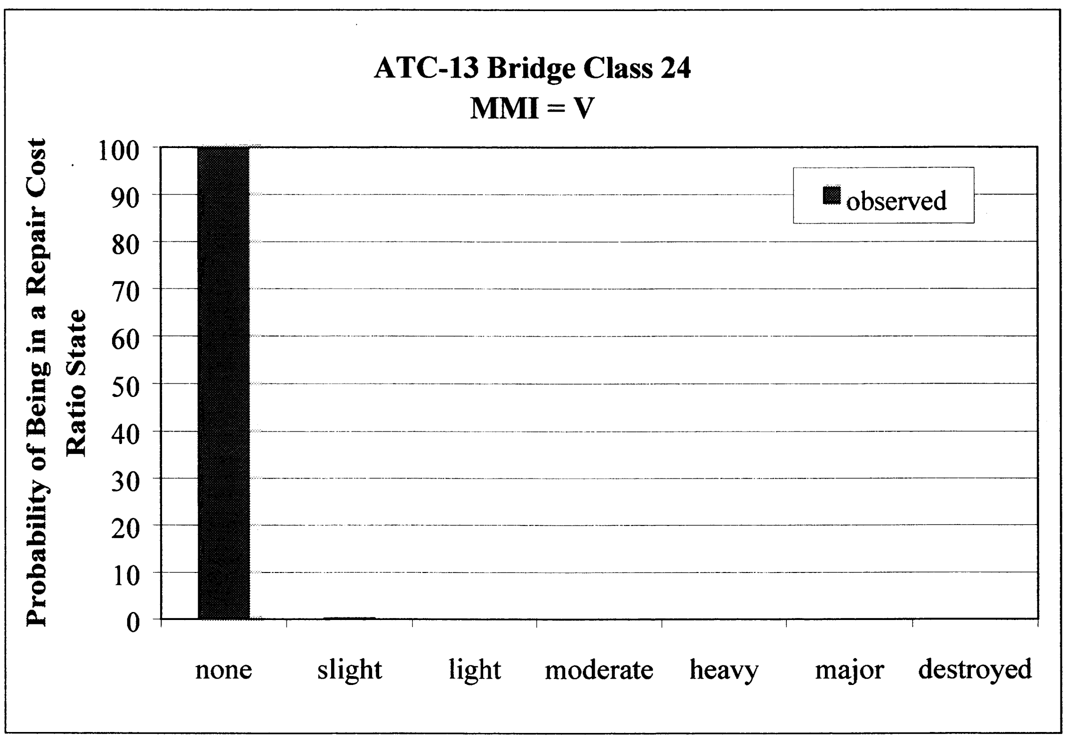


FIGURE 4-49.a Comparison of Observed and Predicted Probabilities of being in a given Repair Cost Ratio State, ATC-13 Bridge Class 24 MMI V

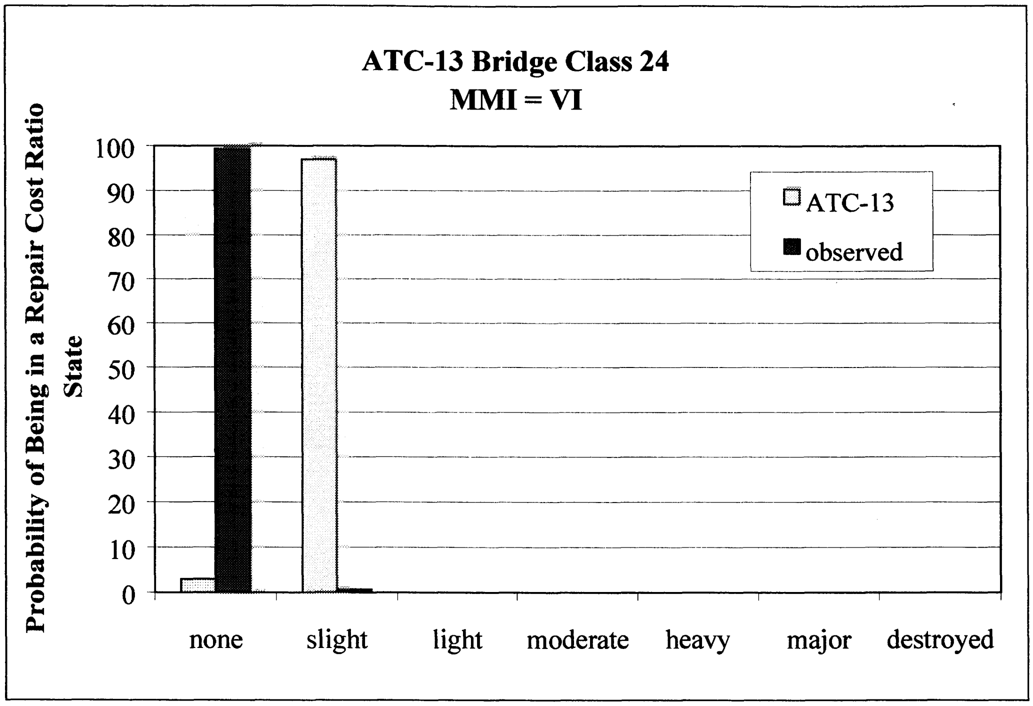


FIGURE 4-49.b Comparison of Observed and Predicted Probabilities of being in a given Repair Cost Ratio State, ATC-13 Bridge Class 24 MMI VI

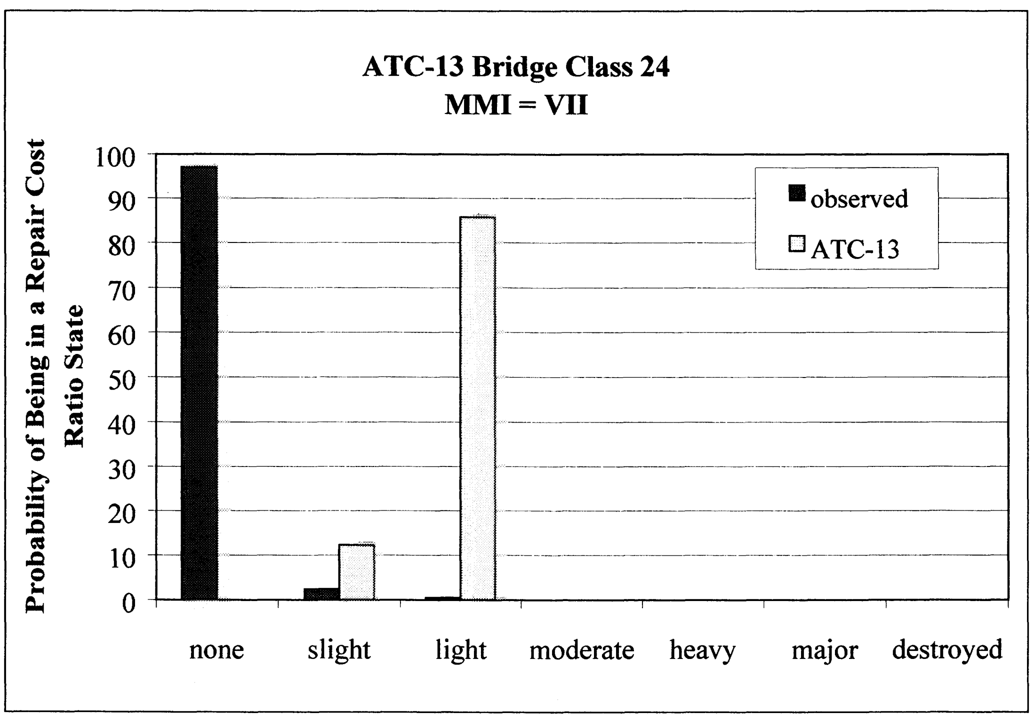


FIGURE 4-49.c Comparison of Observed and Predicted Probabilities of being in a given Repair Cost Ratio State, ATC-13 Bridge Class 24 MMI VII

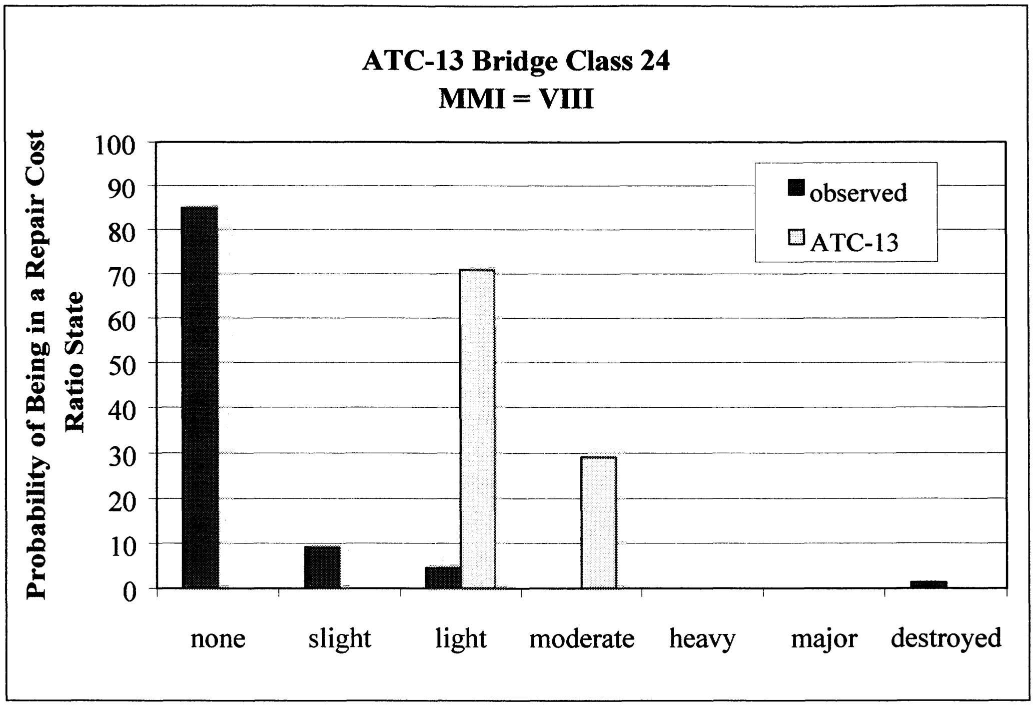


FIGURE 4-49.d Comparison of Observed and Predicted Probabilities of being in a given Repair Cost Ratio State, ATC-13 Bridge Class 24 MMI VIII

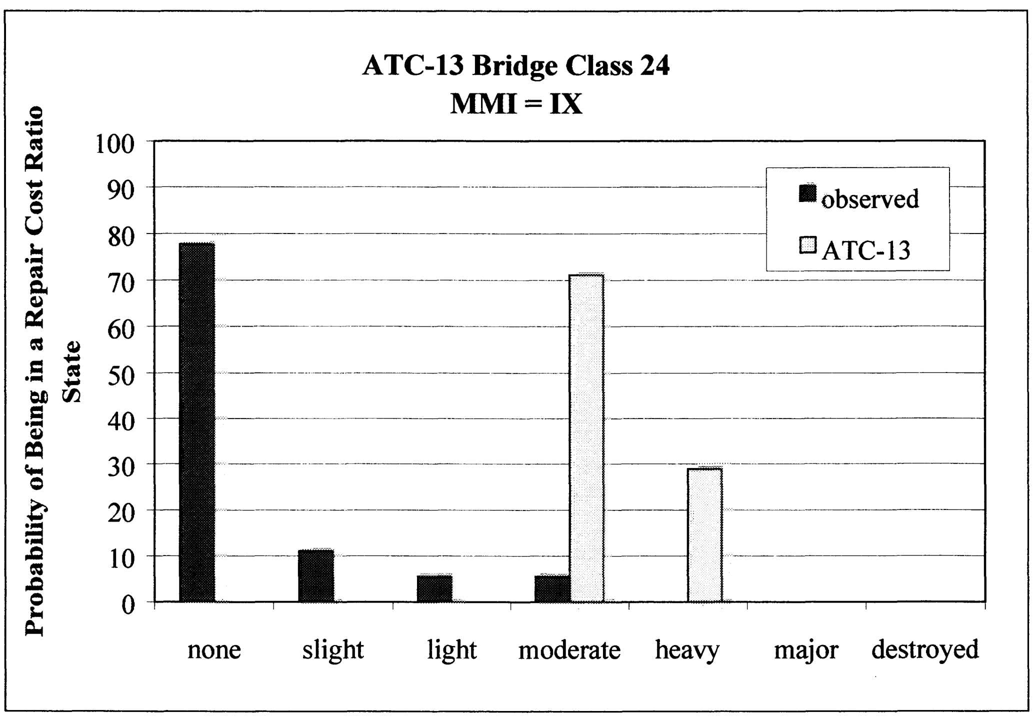


FIGURE 4-49.e Comparison of Observed and Predicted Probabilities of being in a given Repair Cost Ratio State, ATC-13 Bridge Class 24 MMI IX

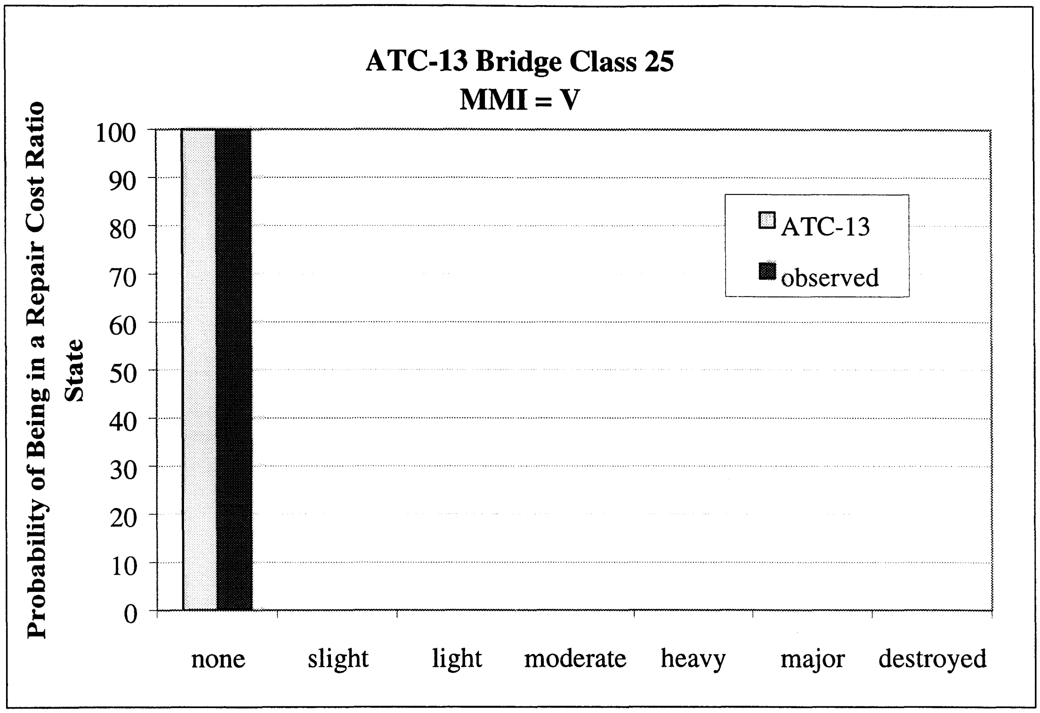


FIGURE 4-50.a Comparison of Observed and Predicted Probabilities of being in a given Repair Cost Ratio State, ATC-13 Bridge Class 25 MMI V

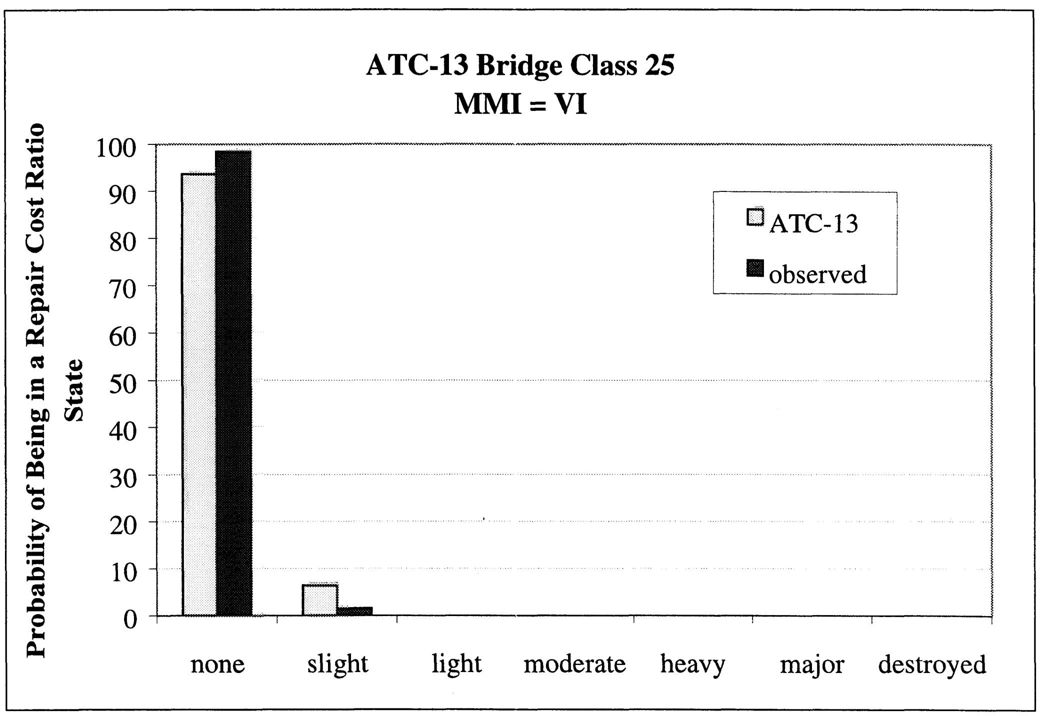


FIGURE 4-50.b Comparison of Observed and Predicted Probabilities of being in a given Repair Cost Ratio State, ATC-13 Bridge Class 25 MMI VI

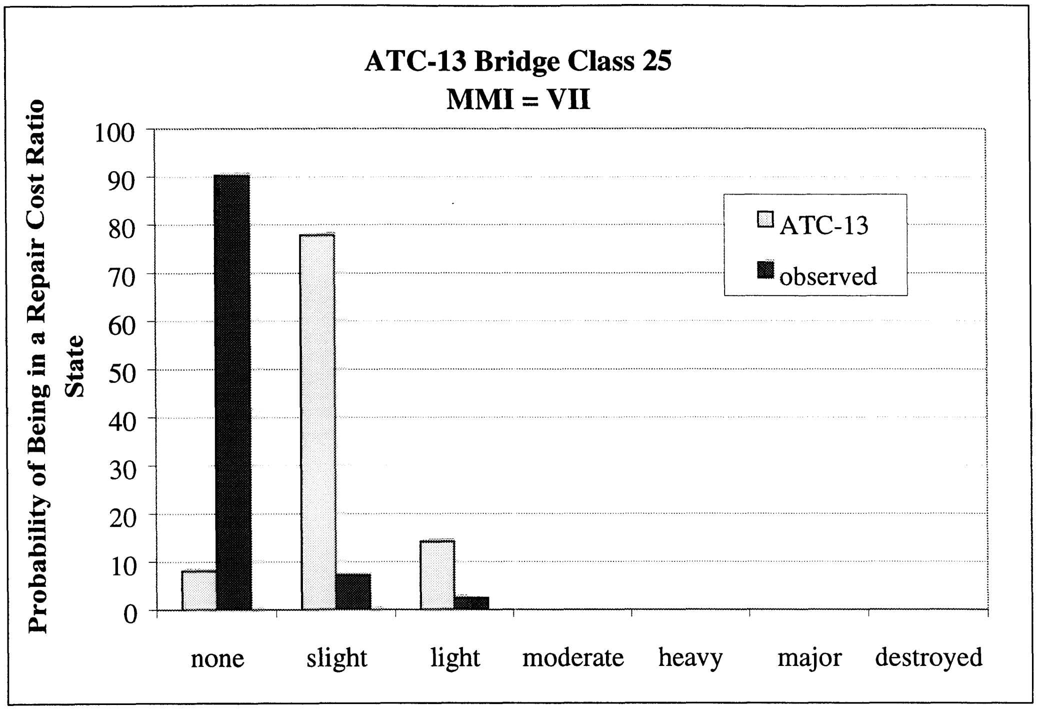


FIGURE 4-50.c Comparison of Observed and Predicted Probabilities of being in a given Repair Cost Ratio State, ATC-13 Bridge Class 25 MMI VII

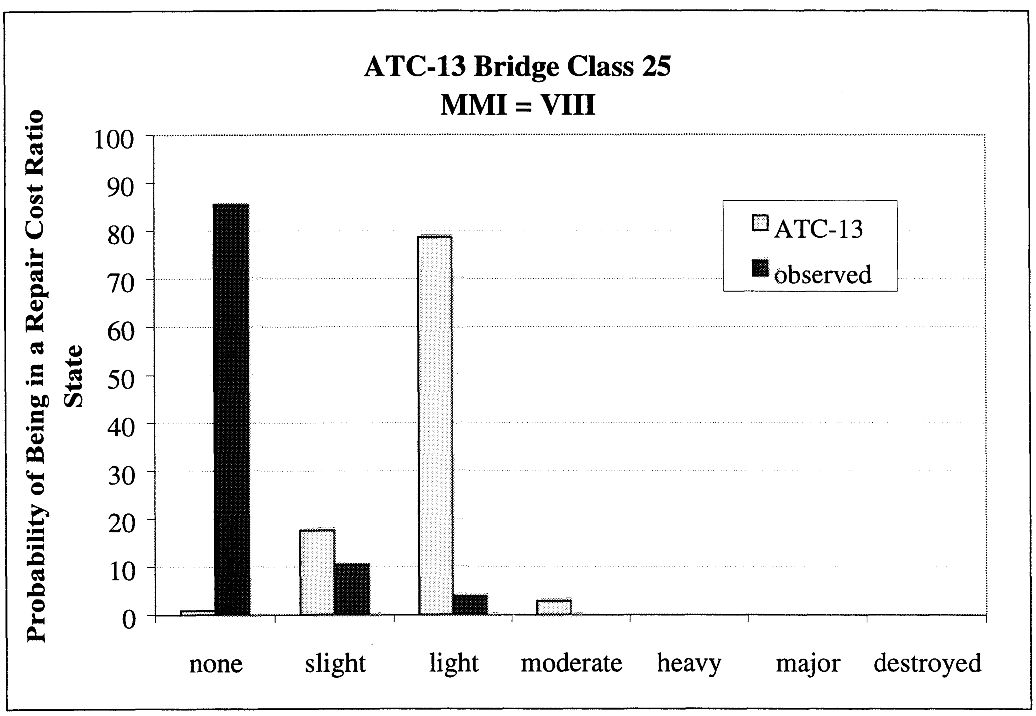


FIGURE 4-50.d Comparison of Observed and Predicted Probabilities of being in a given Repair Cost Ratio State, ATC-13 Bridge Class 25 MMI VIII

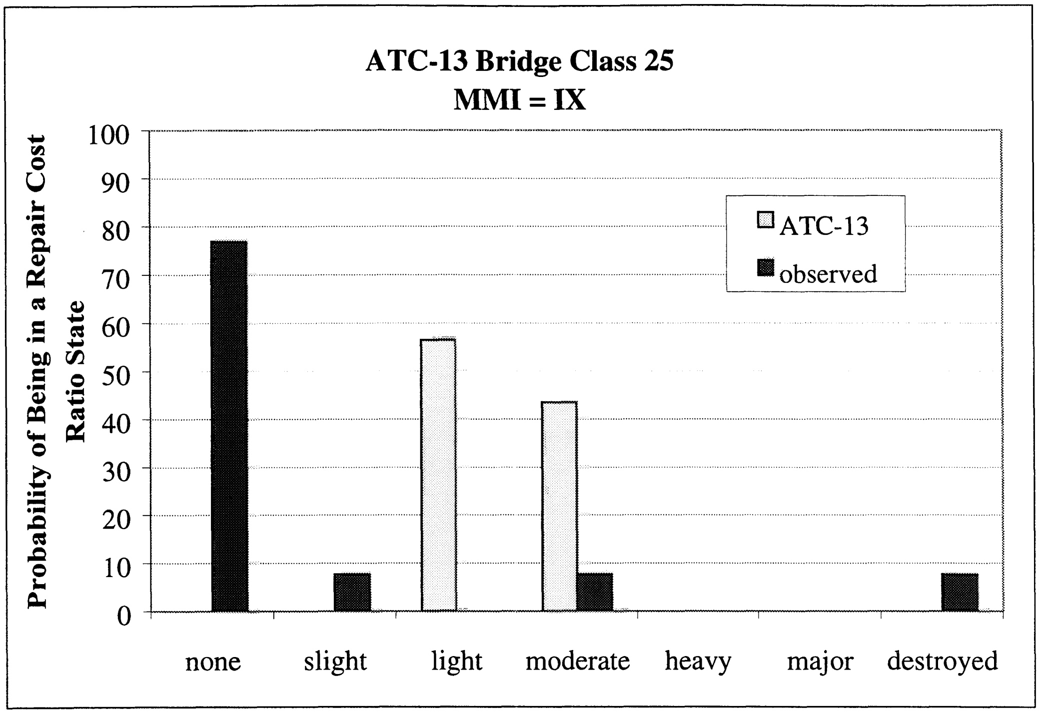


FIGURE 4-50.e Comparison of Observed and Predicted Probabilities of being in a given Repair Cost Ratio State, ATC-13 Bridge Class 25 MMI IX

4.6 DISCUSSION

Based on the analysis of data presented in this section, several important observations can be made. These are summarized as follows:

Ground motion levels: Peak ground acceleration was used to characterize the ground shaking. The two sets of ground motion levels available for the Northridge earthquake [USGS, 1994; WCFS, 1995] were used in the correlation studies. The highest PGA values at a bridge site obtained from the USGS and WCFS reports were 1.55g and 0.66g, respectively. A total of 1,181 and 1,003 bridges were included in the *correlation data set* using USGS [1994] and WCFS [1995] PGA values, respectively. Because the PGA levels were considerably different, empirical ground motion-damage and ground motion-repair cost ratio relationships were obtained for both data sets. Figure 4-51 shows comparison of PGA values reported by USGS and WCFS. According to the PGA values reported by USGS, a number of bridges experienced PGA levels higher than 1.0g.

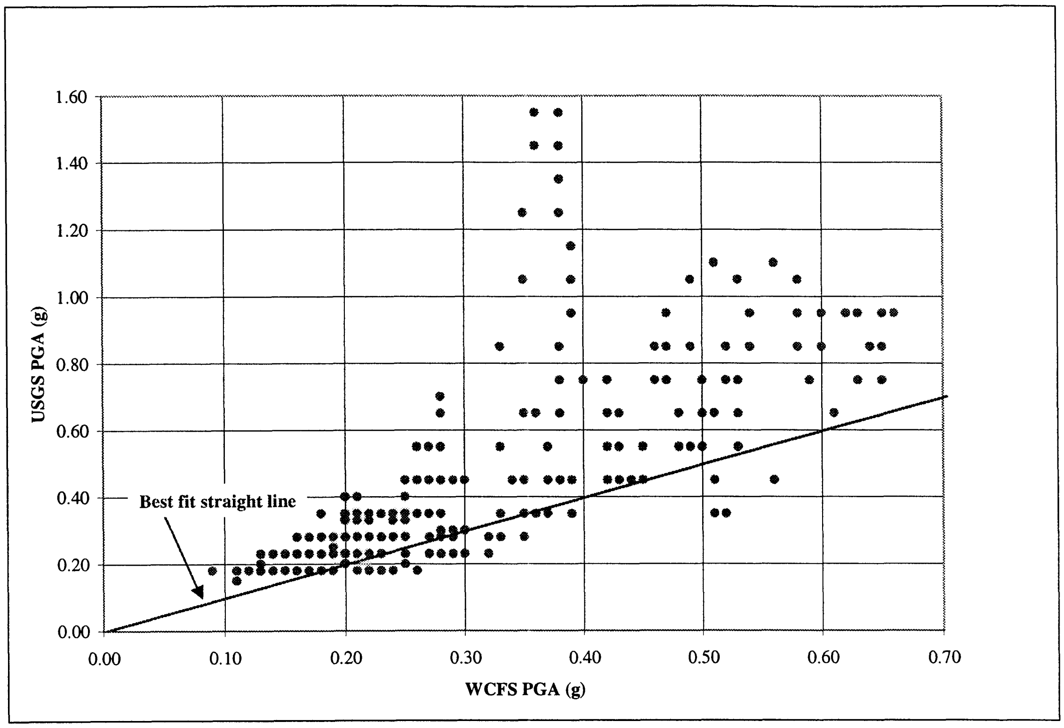
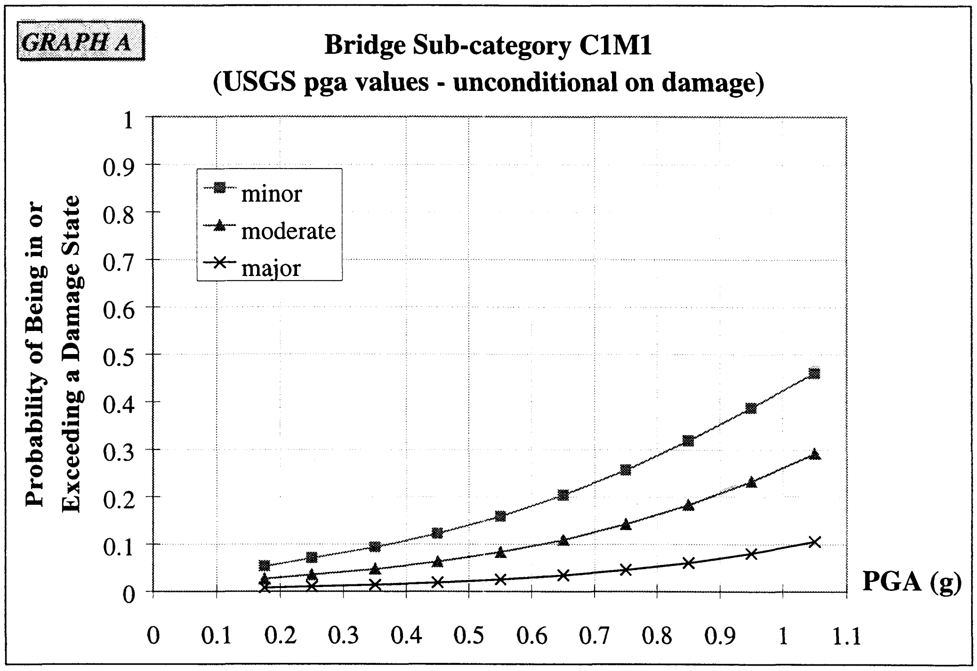


FIGURE 4-51 PGA Values Reported by USGS [1994] and WCFS [1995] – Northridge Earthquake

Figures 4-52 and 4-53 show the probabilities of being in or exceeding a damage state at different PGA values reported by USGS and WCFS respectively, for multiple span concrete bridges with monolithic abutment, continuous span and multiple column bents (sub-category C1M1) and multiple span bridges. The probability values at a given PGA level were quite different. In cases where the maximum USGS PGA value was around 1.0g, however, the shapes of the fragility curves based on the two ground motion data sets were similar. The probabilities of exceeding or being in a damage state were also quite similar at the respective maximum PGA levels. As can be inferred from figure 4-53, when bridges at higher USGS PGA values were included in the analysis, differences were observed both in the shapes of the curves and the maximum probability values.

In order to use a data set with consistent characteristics in the logistic regression analysis, only bridges subjected to 0.15g or higher PGA were included in the analysis. This criterion for data selection was adapted instead of selecting bridges in a geographical boundary.



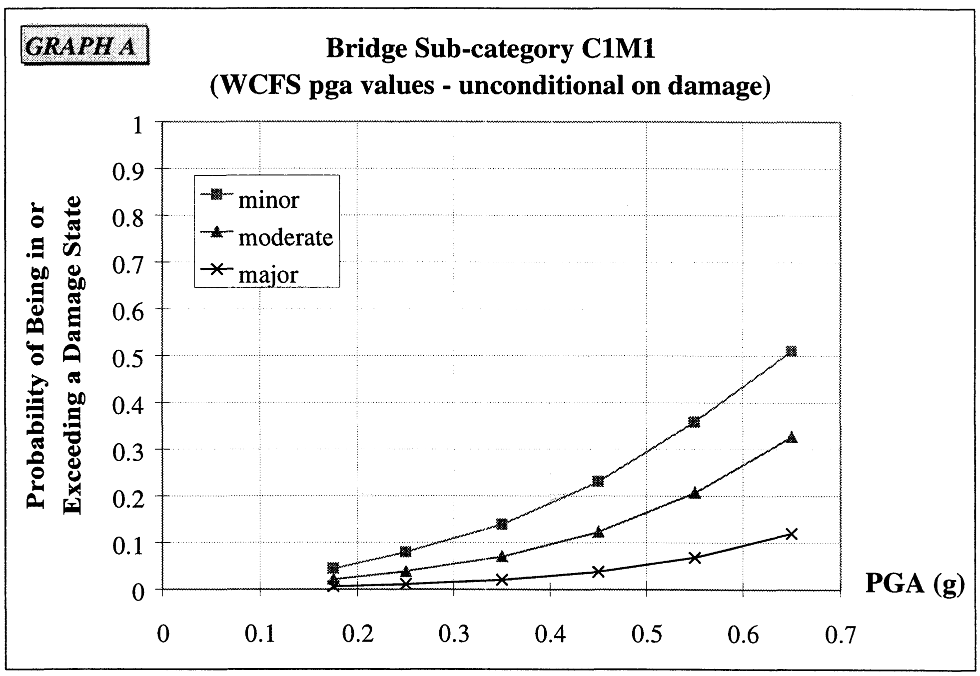
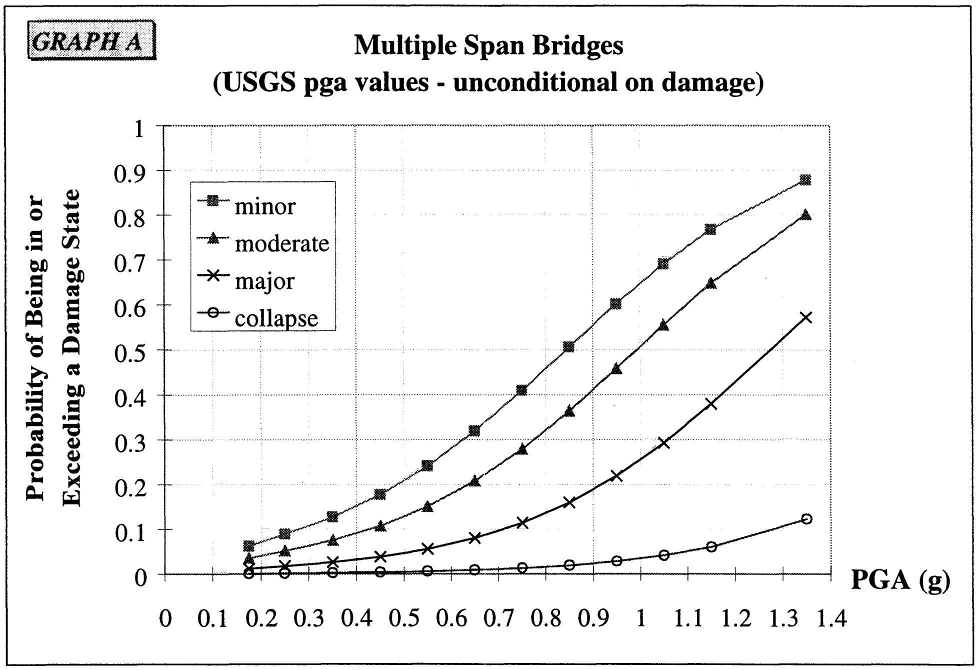


FIGURE 4-52 Empirical Fragility Curves for Multiple Span Bridges with Monolithic Type Abutments, Continuous Spans and Multiple Column Bents (Sub-category C1M1) based on USGS [1994] and WCFS [1995] PGA Values



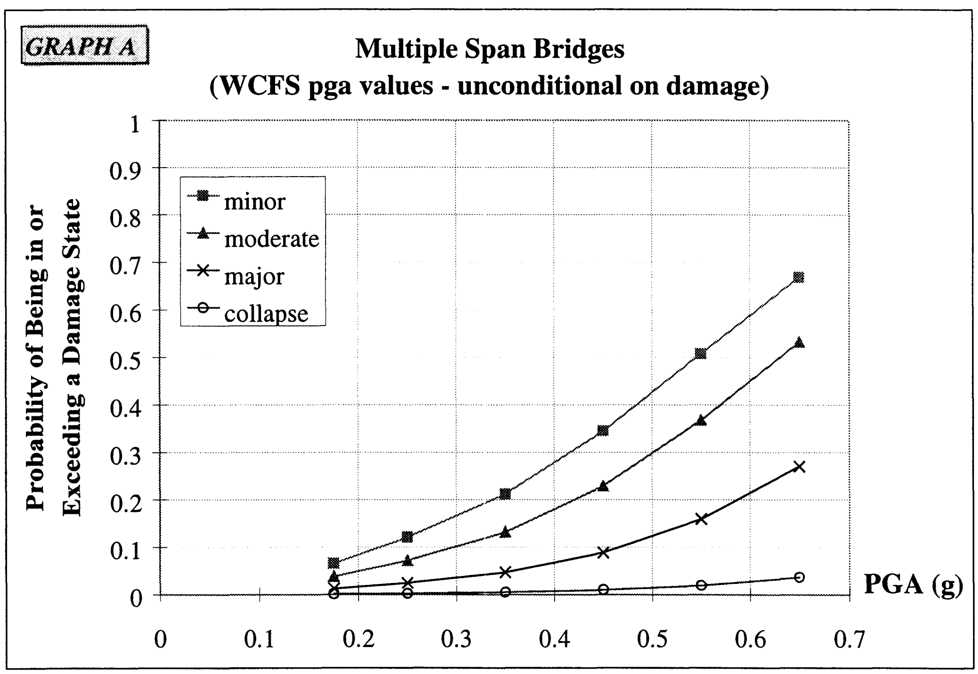


FIGURE 4-53 Empirical Fragility Curves for Multiple Span Bridges based on USGS [1994] and WCFS [1995] PGA Values

<u>Damage states:</u> In this study, the damage states reported during the post-earthquake bridge inspections were used in the analyses. A detailed investigation of the damage state definitions demonstrated the inconsistency and the subjectivity in bridge damage definitions among practicing engineers, and the need for a more systematic procedure to assess bridge damage. Based on the damage reports from the Northridge earthquake, component-based damage states were defined for concrete bridges.

<u>Structural characteristics and damage:</u> A total of 233 bridges were reported as damaged. Twelve of these bridges were pedestrian or railroad bridges. Twenty-one of the damaged bridges had steel superstructure. Because the majority of the bridges in the Greater Los Angeles area and the bridges damaged in the earthquake (200 out of 233 damaged bridges) were concrete bridges, analyses were carried out mainly for concrete highway bridges. However, empirical fragility curves were developed for all steel bridges that were exposed to PGA levels 0.15g or higher reported by USGS [1994]. These fragility curves are compared to empirical fragility curves developed for all concrete bridges in figure 4-54.

The largest PGA level experienced at the site of any steel bridge was 0.85g in comparison with 1.45g at the site of concrete bridges. None of the steel bridges collapsed, thus the probability of *collapse* for steel bridges was obtained as zero based on the data. Based on the damage data, the probability of *major* damage at a given PGA level below 0.85g was the same for concrete and steel bridges. On the other hand, steel bridges were more likely to experience *moderate* or *minor* damage than concrete bridges.

Bridges with homogeneous characteristics were used in the correlation studies. This criterion reduced the data set for damaged concrete bridges from 200 to 164. Although a small data set is not desirable, this criterion provided a database with more homogeneous bridge characteristics from which the effects of different abutment and column bent types on bridge damage susceptibility were determined. Due to lack of data, bridges with heterogeneous characteristics were not studied in detail.

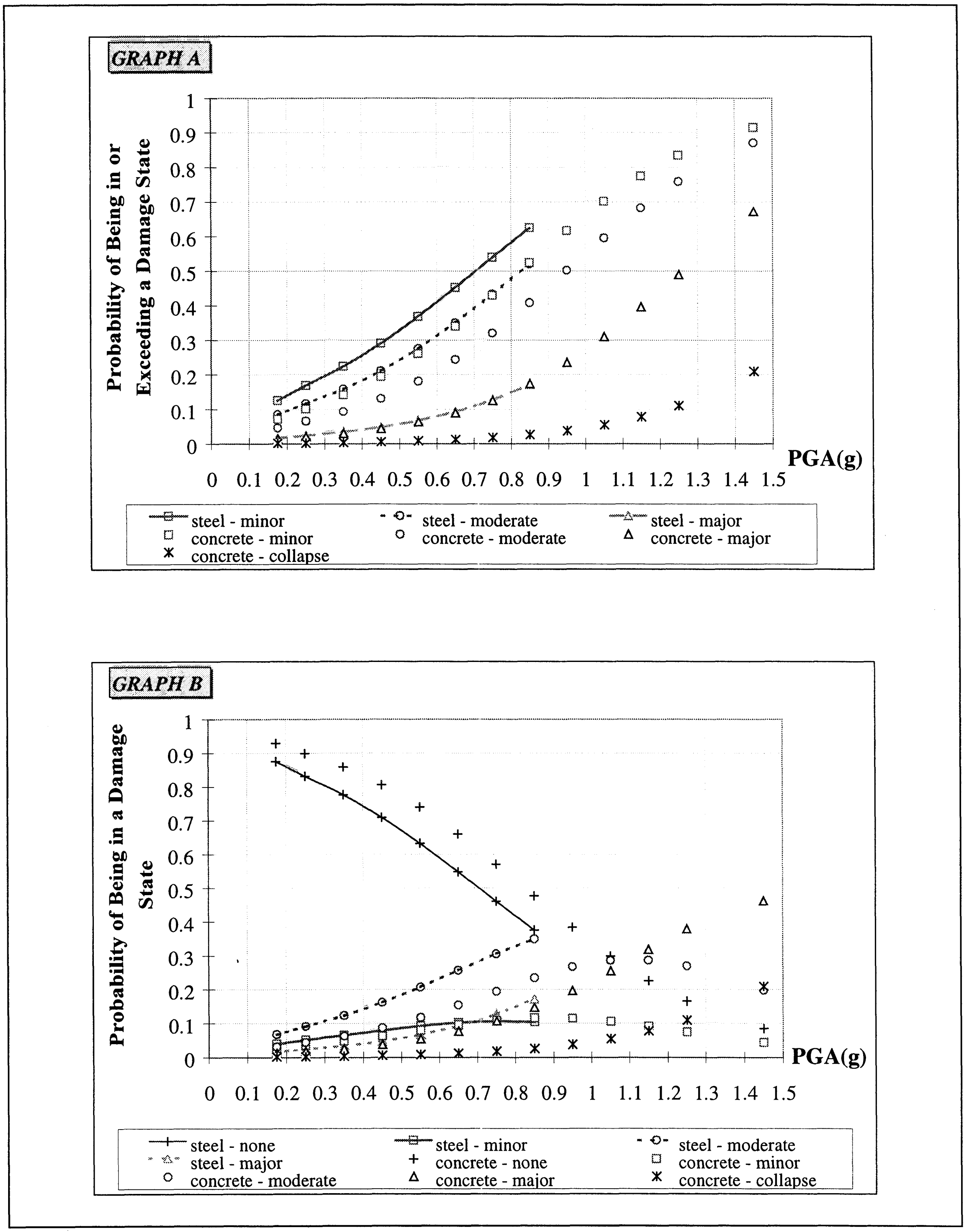


FIGURE 4-54 Empirical Fragility Curves for Steel and Concrete Bridges

The majority of the damaged bridges were designed according to pre-1971 standards, and were not retrofitted at the time of the earthquake. For analysis purposes, four design year periods were used: before 1940, between 1940 and 1971, between 1972 and 1980 and after 1980. Based on the results from logistic regression analysis, however, the performance of bridges did not show much variation with design year. One possible explanation is that the original design year recorded in the database was used in the statistical analyses. Some of the bridges, however, had been upgraded seismically or for daily traffic loads. These upgrades were not considered in this study.

The performance of retrofitted bridges was also studied and compared to that of unretrofitted bridges. The comparisons were based only on the PGA levels at bridge sites and some structural characteristics, such as number of spans, abutment type, column bent type and span continuity. In general, bridges with column retrofits performed well. In some cases, bridges with hinge restrainers suffered *major* damage. In order to determine the effect of specific retrofitting schemes more complete information on retrofit history is necessary.

As mentioned above, abutment type was among the attributes that significantly contributed to bridge damage. Bridges in the *correlation data set* were grouped into three categories according to their abutment types: (*i*) monolithic, (*ii*) non-monolithic, and (*iii*) partial abutment types, i.e., bin or cellular closure type abutment. Figure 4-55 shows empirical fragility curves for multiple span bridges with monolithic, non-monolithic and partial abutment types. As shown in figure 4-55, bridges with monolithic abutments were observed to perform better than those with non-monolithic abutments, while bridges with partial type abutments performed at least as well as the bridges with monolithic abutments.

Figure 4-56 shows a comparison of probabilities of exceeding different damage states for bridges with multiple column bents, single column bents and pier walls. Bridges with single column bents had the worst performance among the three column bent types.

<u>Bridge classification and ground motion-damage relationships</u>: In order to explore the effect of structural characteristics, such as number of spans, abutment type, column bent type and span continuity, the bridges were grouped into sub-categories. Basöz and Kiremidjian [1996] proposed bridge classes based on these structural characteristics. In that study, for a given bridge class defined by the superstructure type and substructure type and material, the authors defined the

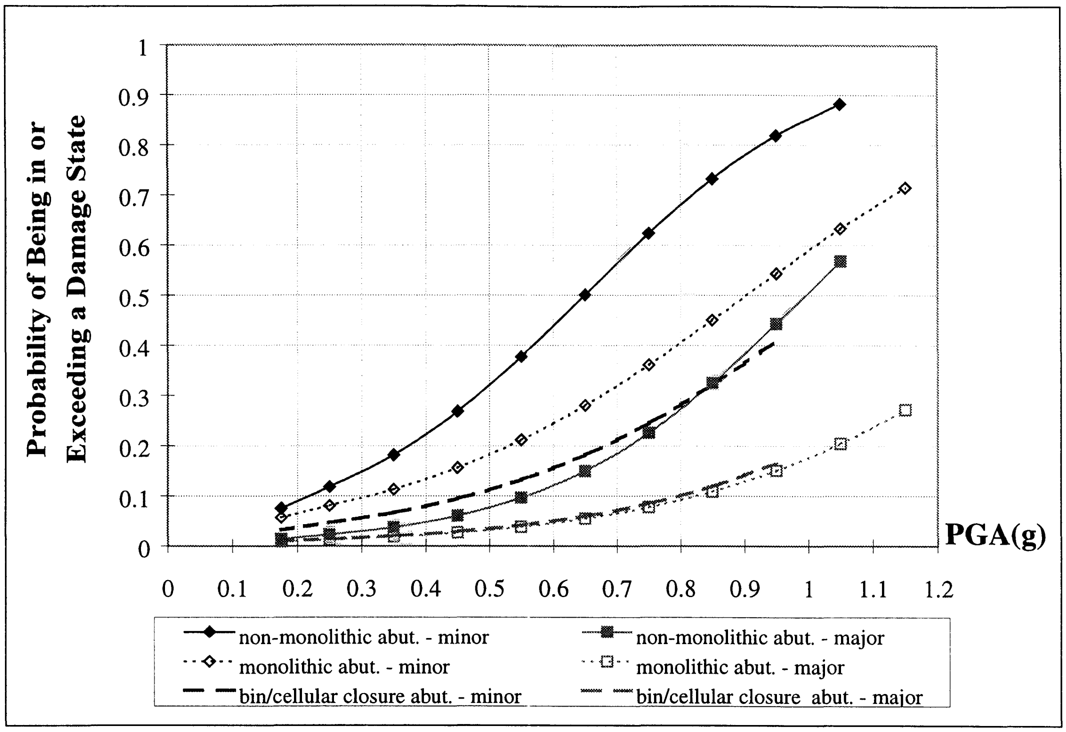


FIGURE 4-55 Performance Comparison of Different Types of Abutments - Minor Damage and Major Damage

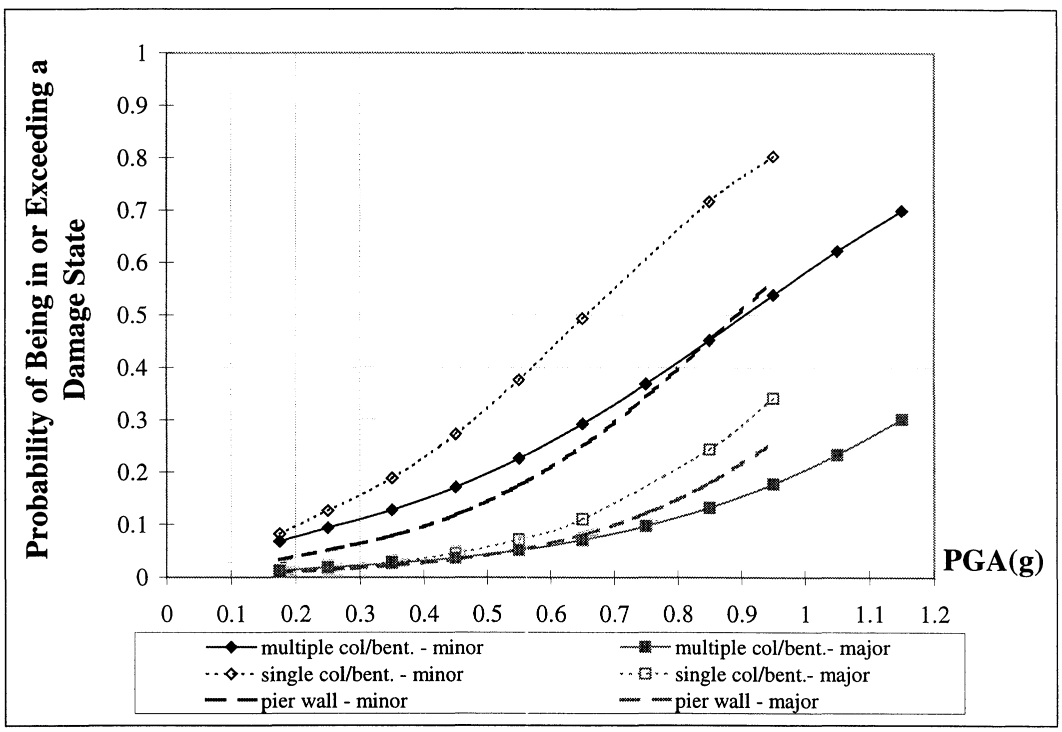


FIGURE 4-56 Performance Comparison of Different Types of Column Bents – Minor Damage and Major Damage

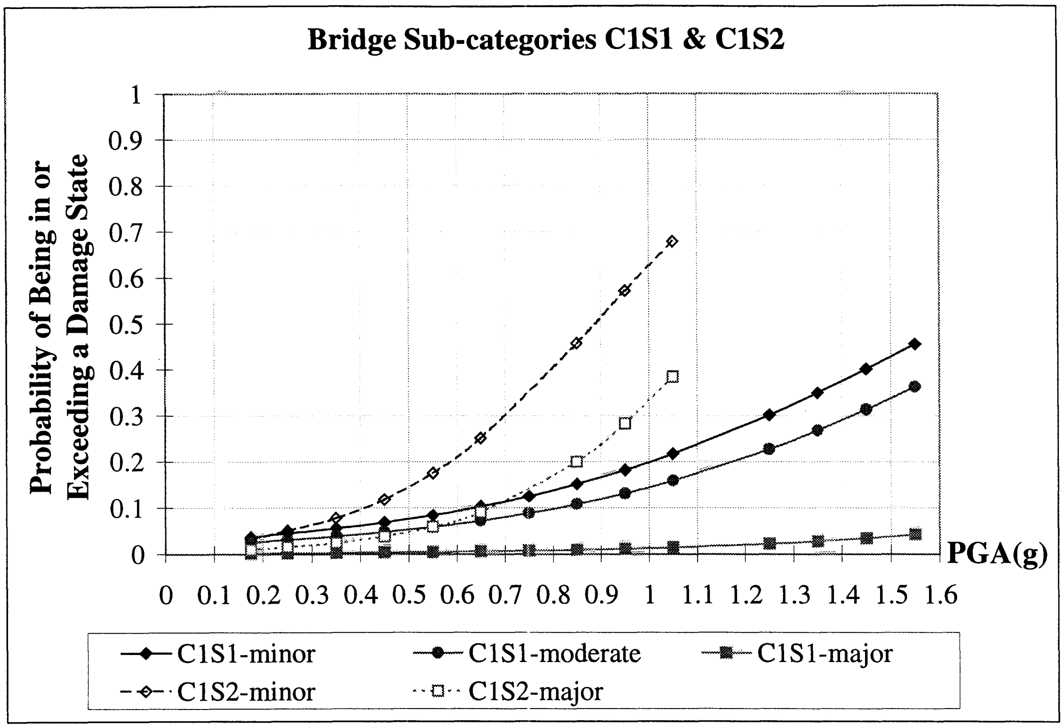


FIGURE 4-57 Empirical Fragility Curves for Single Span Bridges with Monolithic (C1S1) and Non-monolithic (C1S2) Abutments

the least and the most vulnerable single span and multiple span sub-categories. These four sub-categories for concrete bridges were included among the twenty-one sub-categories used in this study. The empirical fragility curves developed in this study were used to validate the bridge class definitions by Basöz and Kiremidjian [1996]. Figure 4-57 shows the empirical fragility curves obtained in this study for the least and the most vulnerable sub-categories for single span concrete bridges. As shown in this figure, the probability of exceeding a given level of damage is higher at all PGA levels for single span bridges with non-monolithic abutments (most vulnerable sub-category) than it is for bridges with monolithic type abutments (least vulnerable sub-catgegory).

Figures 4-58 and 4-59 show the probability of being in a damage state at different PGA levels for the least vulnerable (C1M1), most vulnerable (C1M10) and intermediate level (C1M7) bridges with multiple spans. The probability values represent the ratio of number of damaged bridges to

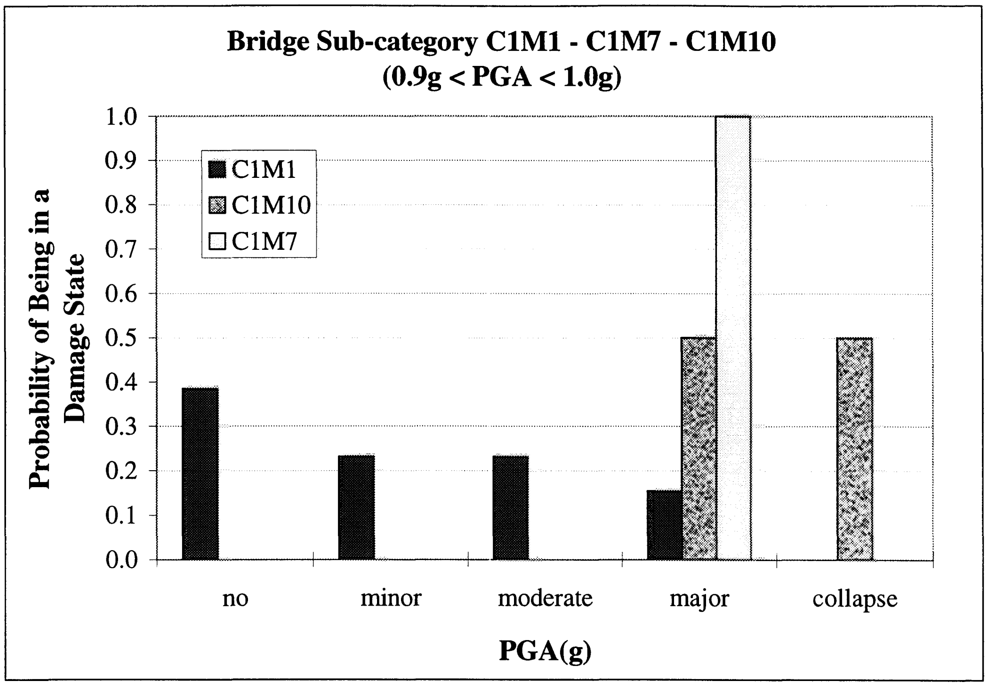


FIGURE 4-58 Damage Probability Matrix for Bridge Sub-categories C1M1, C1M7, and C1M10; 0.6g < PGA < 0.7g

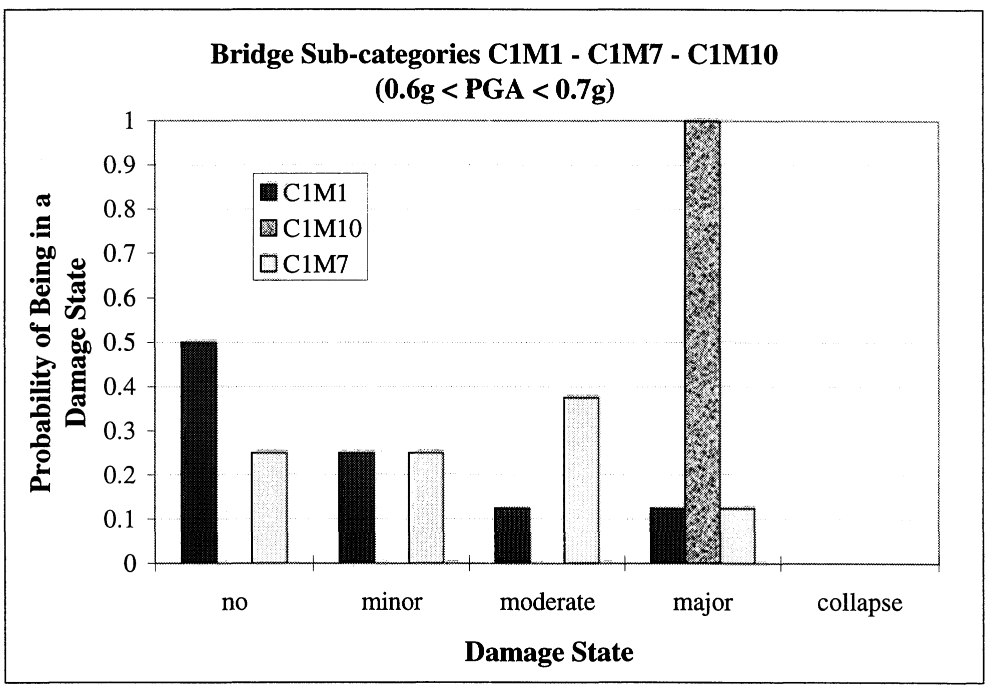


FIGURE 4-59 Damage Probability Matrix for Bridge Sub-categories C1M1, C1M7, and C1M10; 0.9g < PGA < 1.0g

the total number of bridges of a given sub-category at a specified PGA level. For all PGA levels at which damage was observed, higher frequency values were obtained for bridge sub-category C1M10 in comparison to C1M1. The probability values obtained for C1M7 were between the probability values obtained for the least vulnerable sub-category (C1M1) and the most vulnerable sub-category (C1M10) for multiple span concrete bridges. Figure 4-60 shows the empirical fragility curves for the two sub-categories of multiple span bridges: C1M1 (bridges with monolithic abutment type, multiple column bents, continuous span), and C1M7 (bridges with non-monolithic abutment, multiple column bents and continuous spans), respectively. The probability of exceeding a given damage state was higher for bridges in sub-category C1M7 than it was for C1M1 for all damage states. Due to scarcity of data, the fragility curves developed for C1M10 (bridges with non-monolithic abutment, single column bents and discontinuous spans), the most vulnerable bridge sub-category, did not meet the statistical goodness of fit criteria.

In addition to the bridge classes defined in this study, the bridges were grouped according to HAZUS [1997] bridge classes. The observed damage data do not appear to agree well with the available ground motion-damage relationships for most of the bridge classes. Furthermore, the observed relative vulnerabilities between some of the bridge classes do not follow the expected pattern. Figure 4-60 shows the comparison of the empirical and the HAZUS fragility curves for bridge classes HBR3 and HBR9. The "high risk" bridge class HBR9 performed better in the Northridge earthquake than the bridge class HBR3.

<u>Reliability of the inventory data:</u> Another issue investigated in this study was the correctness of the inventory data. The original bridge inventory database contained errors in abutment type and column bent type data. In order to determine the effect of data error on the empirical fragility curves obtained in this study, the ground motion-damage relationships obtained from the corrected data set were compared to those obtained from the uncorrected data. The results of the analysis indicated that the effect of data error on ground motion-damage relationships was more pronounced for smaller data sets.

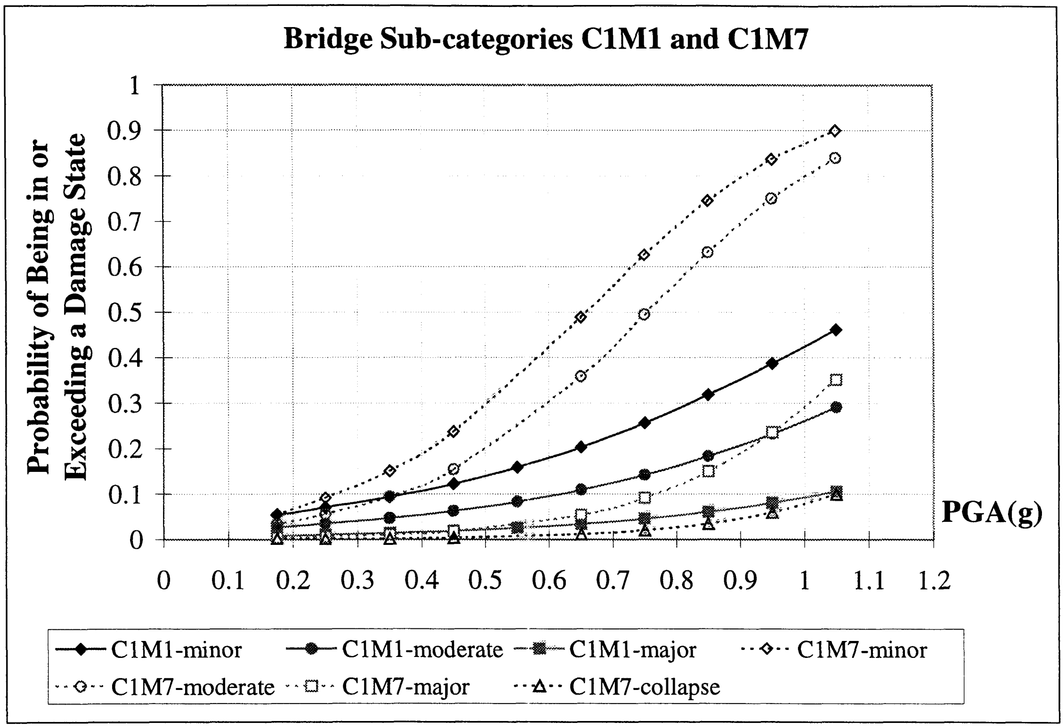


FIGURE 4-60 Empirical Fragility Curves for Sub-categories C1M1 and C1M7

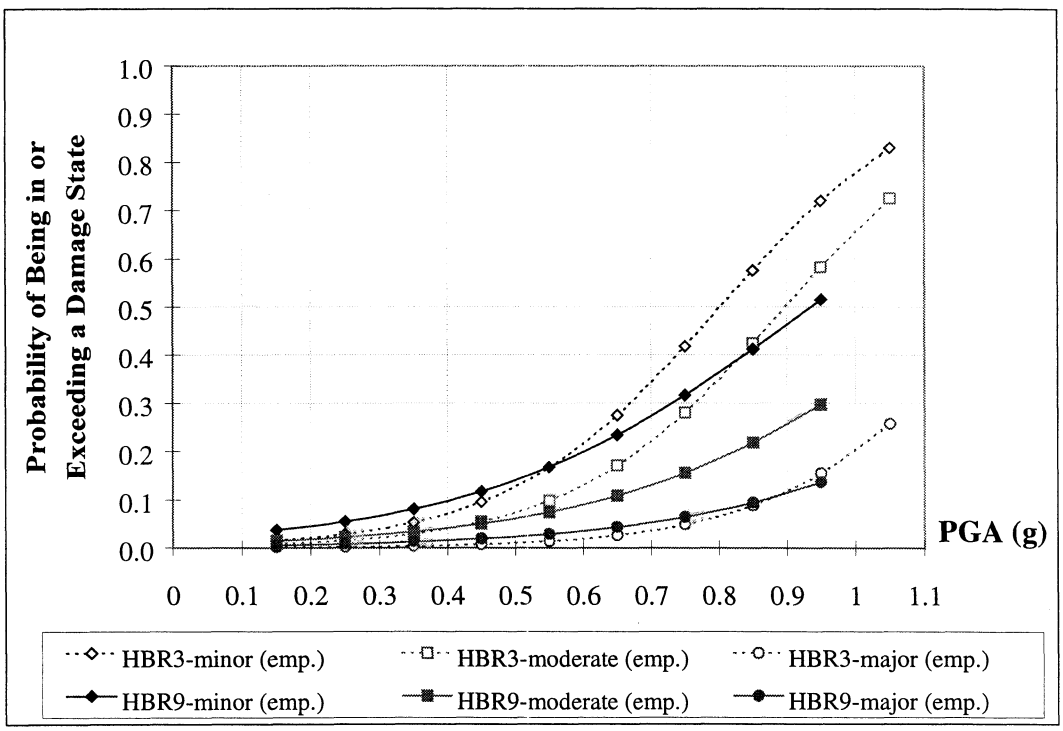


FIGURE 4-61.a Empirical Fragility Curves for HAZUS Bridge Classes HBR3 and HBR9

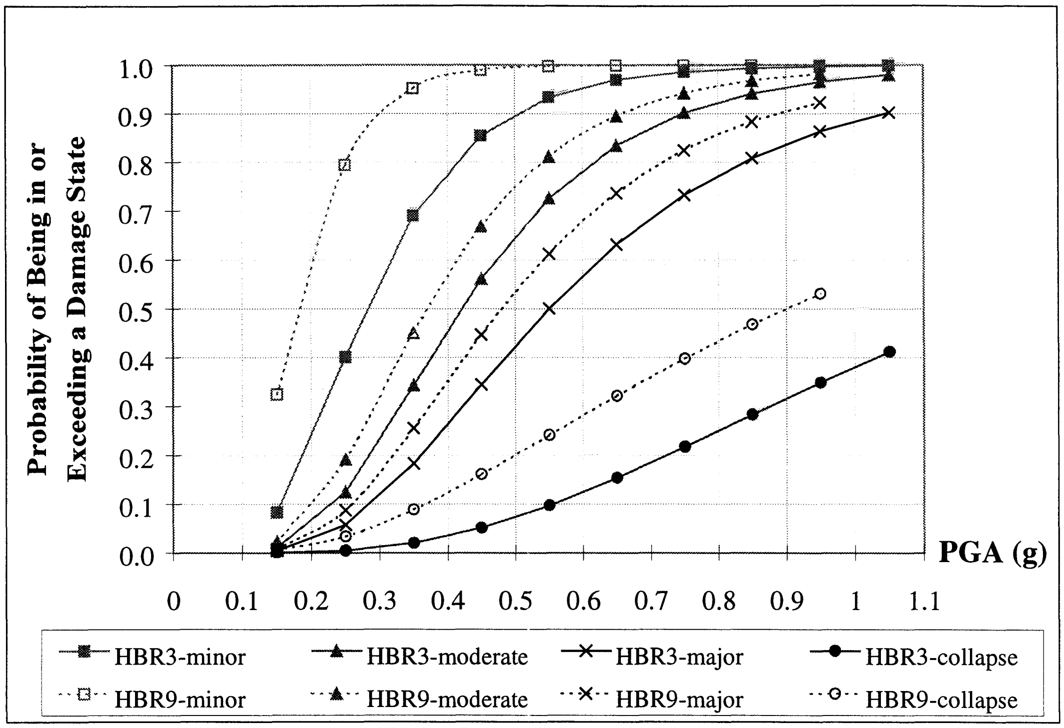


FIGURE 4-61.b HAZUS Fragility Curves for Bridge Classes HBR3 and HBR9

Estimated repair cost: Similar to damage states, several analyses were carried out using the estimated repair cost reported for the damaged bridges. Discrepancies were found between bridge damage data and repair cost data. For some of the bridges, no repair cost was reported. Likewise no damage state was assigned to some bridges for which repair cost estimates were reported. About 75 percent of the repair cost was due to collapsed bridges. A breakdown of component repair cost by damage state showed that column and joint damage accounted for most of the repair cost. Column and joint damage were mostly observed in bridges with *major* damage state. Abutment damage was observed at all damage states, with higher frequency in bridges that suffered *moderate* damage.

The component repair costs were also grouped by bridge sub-categories. Repair cost data existed for only some of the bridge sub-categories; however, the distribution of available component repair cost data by sub-categories reflected the vulnerability levels of the components in these sub-categories. For example, bridge sub-category C1M10 included multiple span bridges with non-monolithic abutment types, single column bents and discontinuous spans.

Similar to damage states, empirical relationships between the repair cost ratio and ground motion levels were developed. These curves facilitate estimation of repair cost of bridges given the ground shaking level and the structural characteristics. The observed repair cost ratio for single span bridges was not more than 10 percent while for multiple span bridges repair cost ratios as high as 50 percent were observed. When the ATC-13 DPMs were compared with the observed repair cost ratios from the Northridge earthquake, it was found that the ATC-13 DPMs agree quite well with the observed damage and the repair cost ratios for low MMI values. In most cases for MMI values VII and higher ATC-13 DPMs overestimated the observed values from the Northridge earthquake.

The results presented in this section provide an overview of the bridge damage from the Northridge earthquake. Several conclusions on ground motion-damage relationships, ground motion-repair cost ratios, and repair cost-damage relationships were drawn in this section. Comparisons of these results with those obtained from the Loma Prieta earthquake bridge damage data are presented in the next section.

SECTION 5

COMPARISON OF RESULTS FROM THE LOMA PRIETA AND THE NORTHRIDGE, CA EARTHQUAKES

The results from the analyses of data on bridge damage in the Loma Prieta and the Northridge earthquakes were discussed in Sections 3 and 4, respectively. These results provide a unique opportunity to understand the seismic performance of bridges. In this section, comparisons of the results from the two earthquakes are presented. Although the two earthquakes had different characteristics, such as magnitude, location and source mechanism, general conclusions on seismic performance of bridges can be drawn based on their structural characteristics and the construction practices.

The total number of state bridges in twelve counties of the San Francisco Bay area is about 2,300. In Los Angeles, Ventura, Orange and Riverside counties there are about 3,500 state bridges. The percentage and number of damaged state bridges in the Loma Prieta and the Northridge earthquakes are shown in figures 5-1 and 5-2, respectively. The total number of damaged bridges during the Northridge earthquake was about three times as large as that from the Loma Prieta earthquake. This difference can be attributed to the fact that damage from the Northridge earthquake was concentrated in the vicinity of the epicenter, an urban area. The majority of the bridge damage in the Loma Prieta earthquake was as far as 60 miles away from the epicenter where soil amplification played a key role in bridge damage.

The economic consequences of bridge failures were not as significant in the Northridge earthquake as they were after the Loma Prieta earthquake. This was mainly due to the different geographic characteristics of the two areas. For example, the highway system that connects the Peninsula to the East Bay in the San Francisco Bay area is not highly redundant while there are several arterial roads in Los Angeles county, the hardest hit urban area in the Northridge earthquake. Furthermore, the restoration time of the bridges with major *damage* or *collapse* was much shorter in the Northridge earthquake than it was in the Loma Prieta earthquake.

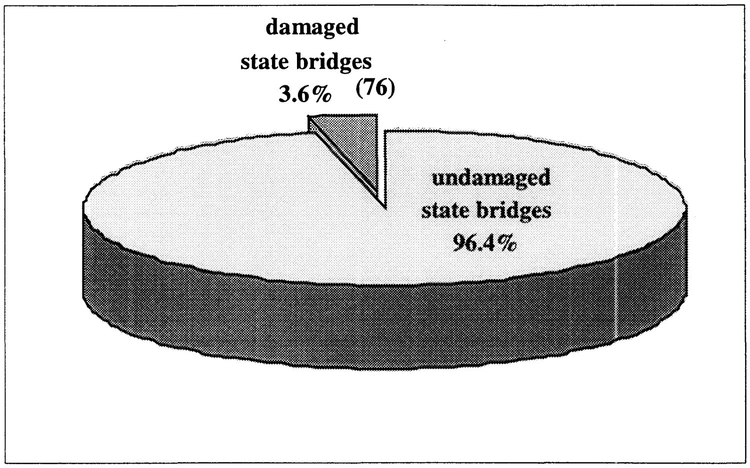


FIGURE 5-1 Percentage and Number of Damaged Bridges in the Twelve Counties of the San Francisco Bay Area

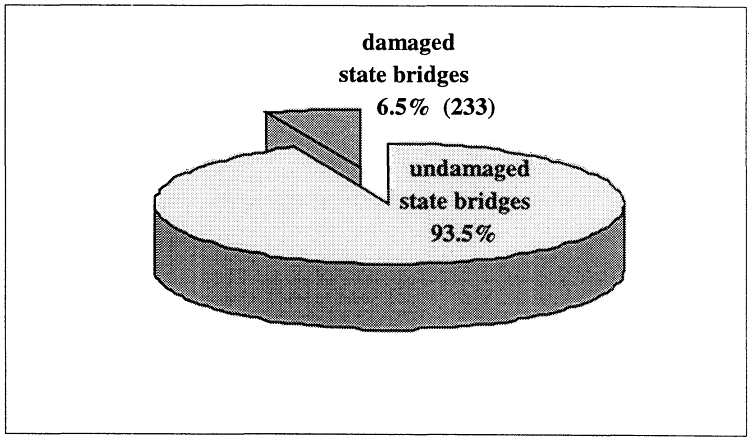


FIGURE 5-2 Percentage and Number of Damaged Bridges in the Four Counties of the Greater Los Angeles Area

The closure of the San Francisco-Oakland Bay Bridge in addition to several viaducts had significant economic consequences following the Loma Prieta earthquake. The number of fatalities due to failure of the Cypress Viaduct was much higher than the total number of casualties due to bridge damage in the Northridge earthquake.

Figures 5-3 and 5-4 show the distribution of bridges in the San Francisco Bay area and the Greater Los Angeles area by design year. In both exposure areas, more than 75 percent of the bridges were built before 1972. In fact, the distribution of bridges by design year interval in the two regions is very similar. It should be noted that, the percentage for "post-1980" design year period in the San Francisco Bay area included bridges designed/built between 1981 and 1989. Distribution of bridges damaged in the Loma Prieta and the Northridge earthquakes are shown in figures 5-5- and 5-6.

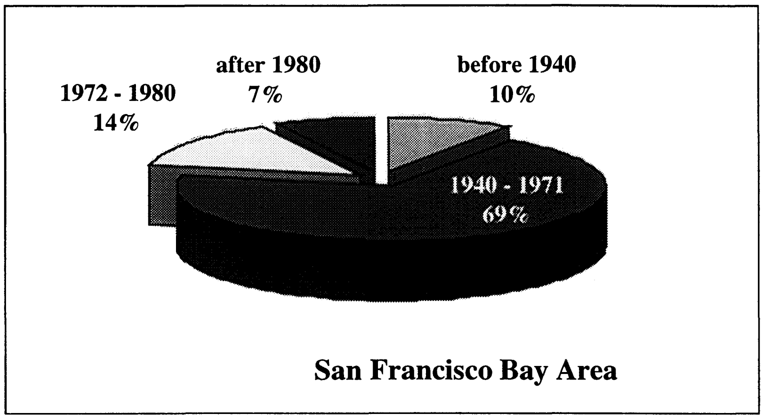


FIGURE 5-3 Distribution of Bridges in the San Francisco Bay Area by Design Year

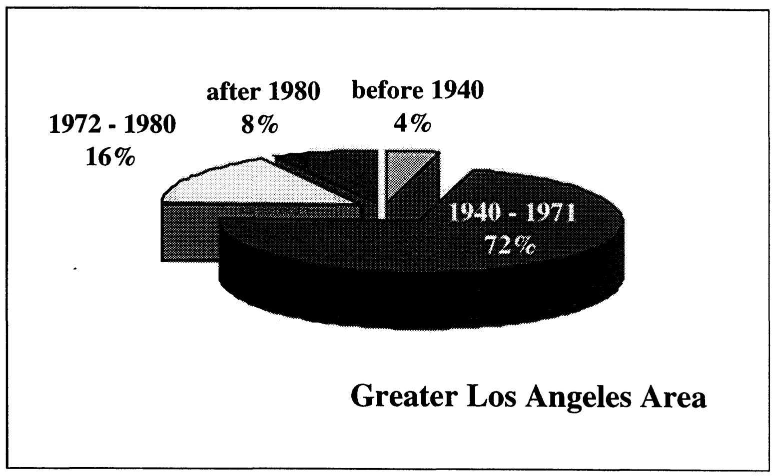


FIGURE 5-4 Distribution of Bridges in the Greater Los Angeles Area by Design Year

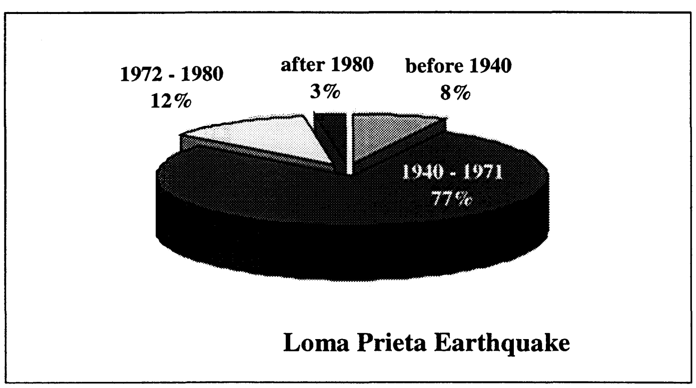


FIGURE 5-5 Distribution of Bridges Damaged in the Loma Prieta Earthquake by Design Year

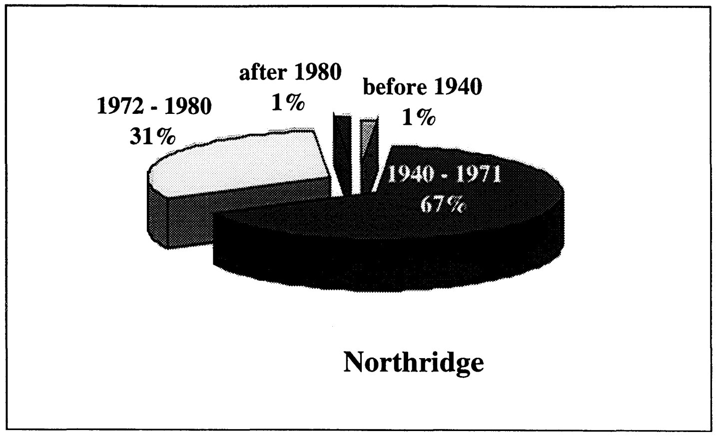


FIGURE 5-6 Distribution of Bridges Damaged in the Northridge Earthquake by Design Year

Figure 5-7 shows the distribution of damaged bridges in the Loma Prieta and Northridge earthquakes by design year interval. The distribution of damaged bridges by design year interval shows similar characteristics to that of the bridge inventories except that a much higher percent of bridges built between 1972 and 1980 sustained damage in the Northridge earthquake.

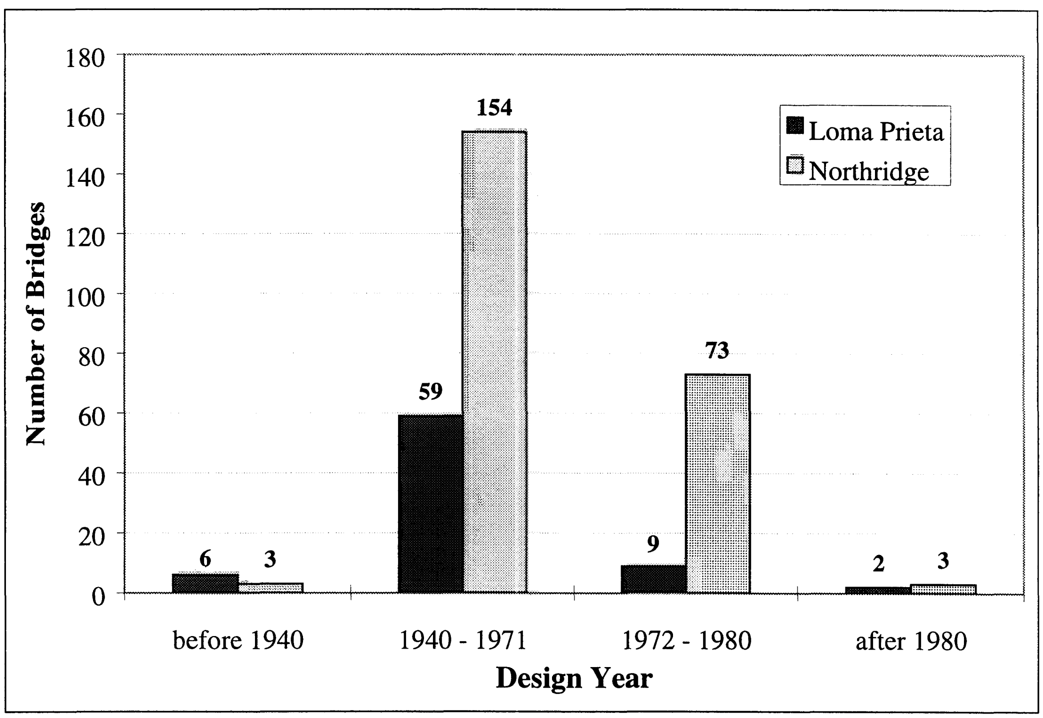


FIGURE 5-7 Number of Bridges Damaged in the Loma Prieta and Northridge Earthquakes by Design Year

Concrete bridges are more common than any other type in the state of California. More than 90 percent of the bridges in the Greater Los Angeles area and about 80 percent of the bridges in the San Francisco Bay area are concrete structures. Similarly, about 82 percent and 91 percent of the damaged bridges in the Loma Prieta and the Northridge earthquakes respectively, were concrete structures.

During the post-earthquake investigations for the Northridge earthquake, bridges were reported to be in one of the four (*minor*, *moderate*, *major* and *collapse*) damage states. In comparison, only two (*minor* and *major*) damage states were used for bridges damaged in the Loma Prieta earthquake. The four level damage assessment provided a better representation of the description for damage details. Furthermore, the empirical ground motion-damage relationships developed based on the four damage states are more informative. A comparison of the damage descriptions for the Loma Prieta and the Northridge earthquakes revealed that *minor* and *moderate damage* in Northridge earthquake reconnaissance reports corresponded roughly to *minor damage* in those of the Loma Prieta earthquake. Similarly, *major damage* and *collapse* from the Northridge earthquake corresponded approximately to *major damage* in the Loma Prieta earthquake. It is important to have a well-defined and consistent set of damage states in order to study and compare the ground motion-damage relationships for bridges in different earthquakes. As described in Singhal and Kiremidjian [1997], Bayesian analysis can be used to update the empirical and/or analytical ground motion-damage relationships, as more data become available.

Multivariate stepwise logistic regression analysis was used to identify the structural characteristics that are most susceptible to damage. Ground motion levels, abutment type and skew were found to have high correlation with the observed damage in both earthquakes. According to the damage data from the Northridge earthquake, column bent type and span continuity were also found to be among the characteristics that are most susceptible to damage. Based on the bridge damage data from the Loma Prieta earthquake data, design year was found to correlate well with the observed damage while column bent type and span continuity were found to be statistically insignificant. Note that the design year used in statistical analyses was the original design year recorded in the database. Some of the bridges, especially at the time of the Northridge earthquake, were seismically upgraded. That is, a bridge built before 1972 might have been retrofitted and hence might have performed better in the Northridge earthquake. The design year used in the analyses was not modified to reflect these upgrades. Therefore, the performance of bridges in the Northridge earthquake did not show much dependence on design year.

Figure 5-8 shows empirical fragility curves developed for bridges built between 1940 and 1972 based on the Loma Prieta and Northridge earthquake data. Higher probabilities of exceeding *minor* and *major* damage states were observed from the Northridge earthquake data. For the Northridge earthquake the PGA values reported by WCFS and for the Loma Prieta earthquake estimated PGA values using Campbell attenuation relationship were used.

In both earthquakes, bridges with monolithic abutments performed better than those with non-monolithic abutments, and bridges with bin or cellular closure type of abutments performed as good as those with monolithic abutments. Similarly, bridges with multiple column bents performed better than single column bent bridges in both earthquakes. Bridges with pier walls performed better than the multiple column bents in most cases.

For the purpose of the correlation studies, concrete bridges with homogeneous structural characteristics which were exposed to peak ground acceleration levels higher than a threshold value were grouped in a data set. This data set was referred to as the *correlation data set*. Bridges in the *correlation data set* were grouped into sub-categories listed in table 2-1, according to their structural characteristics, such as abutment type, column bent type and span continuity. Figure 5-9 shows the distribution of bridges in the San Francisco Bay area and the Greater Los Angeles area by these sub-categories. The distribution in the two areas is very similar. The majority of bridges were multiple span bridges with multiple column bents and continuous span. Likewise, the distribution of damaged bridges by structural characteristics shown in figure 5-10 was very similar in the two areas. The exception was single span bridges with monolithic abutments, none of which were reported as damaged in the Loma Prieta earthquake in contrast to 10 percent of all damaged bridges in the Northridge earthquake. High skew and high ground motion levels contributed to damage at these bridge sites.

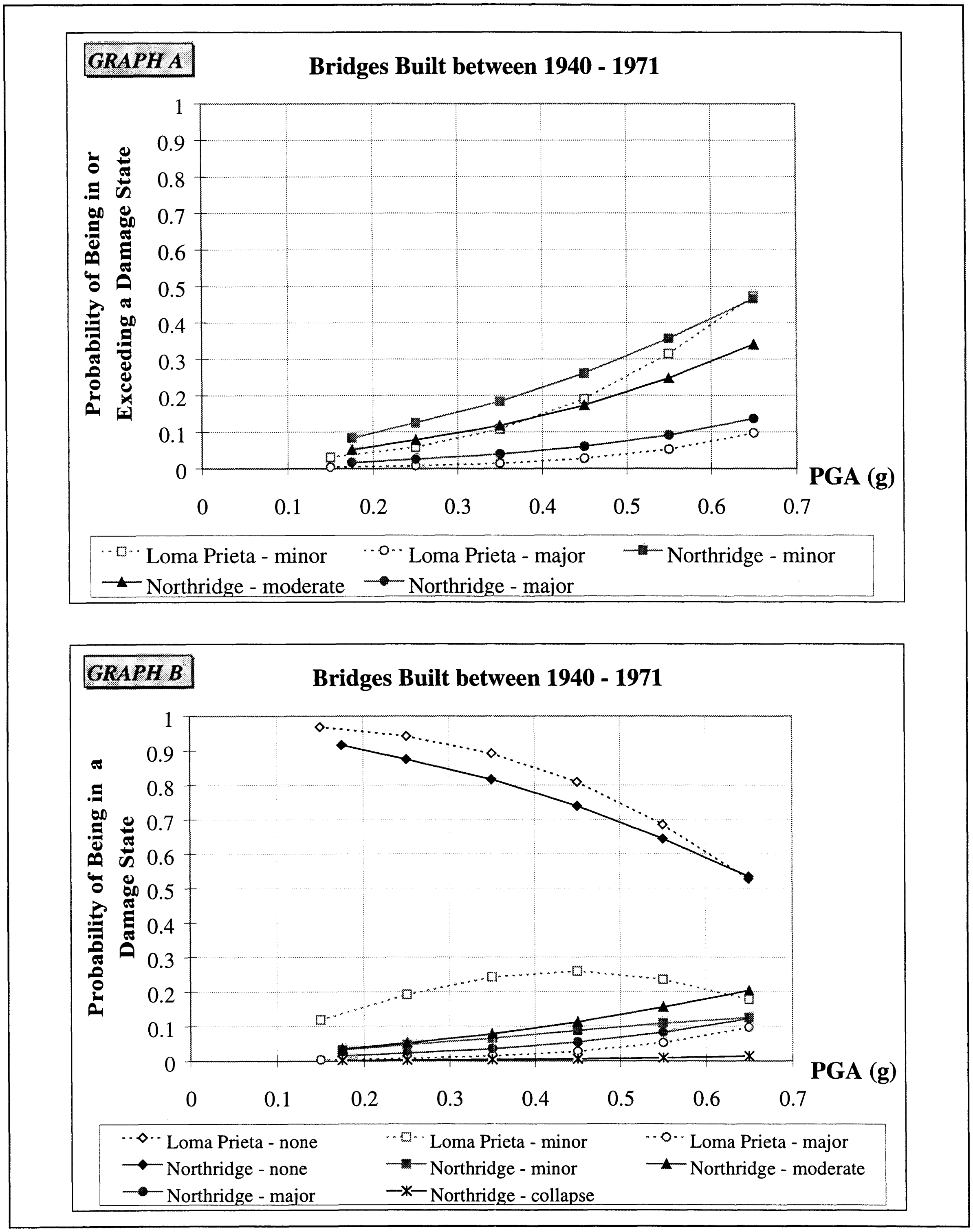


FIGURE 5-8 Empirical Fragility Curves for Bridges Built between 1940 and 1971 in the Loma Prieta and Northridge Earthquakes

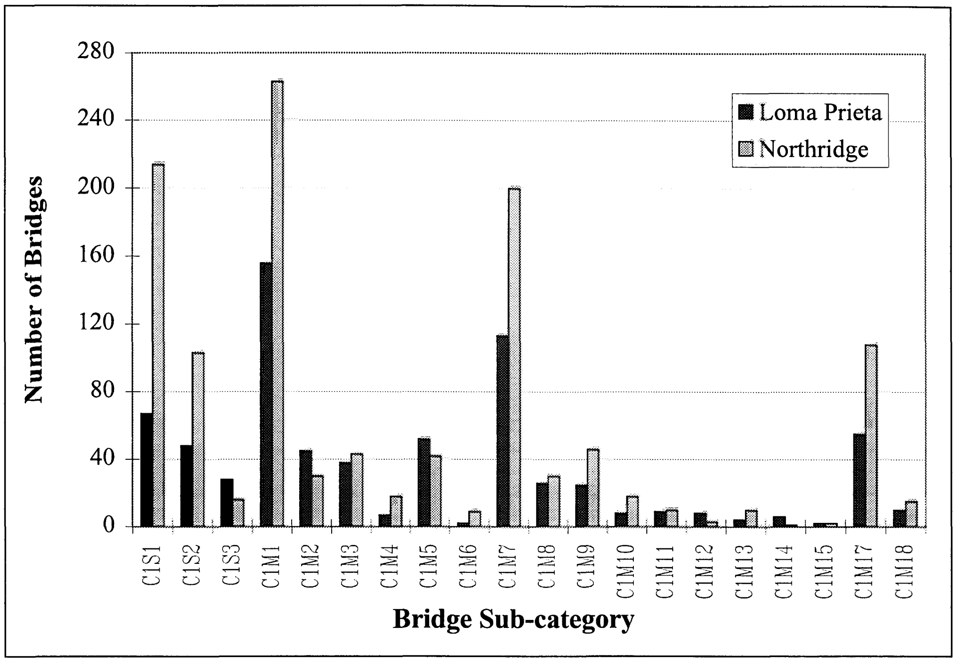


FIGURE 5-9 Number of Bridges in the San Francisco Bay Area and the Greater Los Angeles Area by Bridge Sub-categories (*correlation data set*)

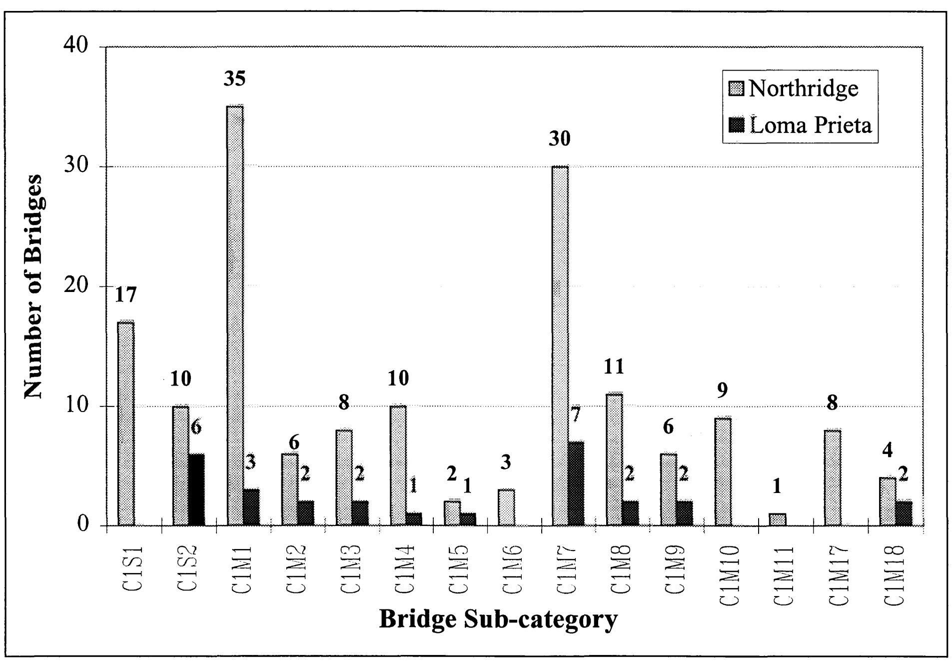


FIGURE 5-10 Number of Damaged Bridges in the Loma Prieta and Northridge Earthquakes by Bridge Sub-categories (*correlation data set*)

For the Northridge earthquake, two sets of PGA levels were available: one reported by USGS [1994] and another by WCFS [1995]. The PGA at a given bridge site was obtained within a GIS by overlaying the ground shaking and the bridge location maps. The highest PGA values obtained at a bridge site were 1.55g and 0.66g for the USGS and WCFS maps, respectively. Since the PGA levels from the two data sets varied considerably the correlation studies were performed for both data sets.

In contrast to the Northridge earthquake, a map of the ground motion intensity levels based on the recordings was not available for the Loma Prieta earthquake. For the Loma Prieta earthquake, a scenario event was generated using GIS. The PGA values obtained from the scenario event were found to be considerably different from the recorded ones at some locations. Nevertheless, due to lack of a better measure of the ground motion at bridge sites, the ground motion map based on the scenario event was used in studying the bridge damage from the Loma Prieta earthquake. Soft soil contributed to the bridge damage significantly in the Loma Prieta earthquake in contrast to the Northridge earthquake.

Ground motion-damage relationships were obtained for various groups of bridges. The ground motion-damage relationships from the two earthquakes are compared in figures 5-11 through 5-13 for multiple span bridges, and bridge sub-categories C1M1 and C1M7[1], respectively. In these figures fragility curves based on different sets of PGA levels for different damage states are shown. In each figure, two graphs are presented: the first graph shows *minor* damage for the Loma Prieta earthquake and *minor* and *moderate* damage for the Nothridge earthquake. The latter one shows *major* damage for the Loma Prieta earthquake and *major* damage and *collapse* for the Nothridge earthquake. For the Northridge earthquake the PGA values reported both by USGS and WCFS were used. For the Loma Prieta earthquake the PGA values estimated using the Campbell attenuation relationship and "observed" PGA levels estimated based on the best line fit in figure 3-3 were used.

Several observations can be made from these figures. The variability of the ground motion estimates was considerable. At a given PGA level, the probability of being in or exceeding a

---

[1] C1M1: multiple span bridges with monolithic type abutments, continuous spans and multiple column bents.
C1M7: multiple span bridges with monolithic type abutments, discontinuous spans and multiple column bents.

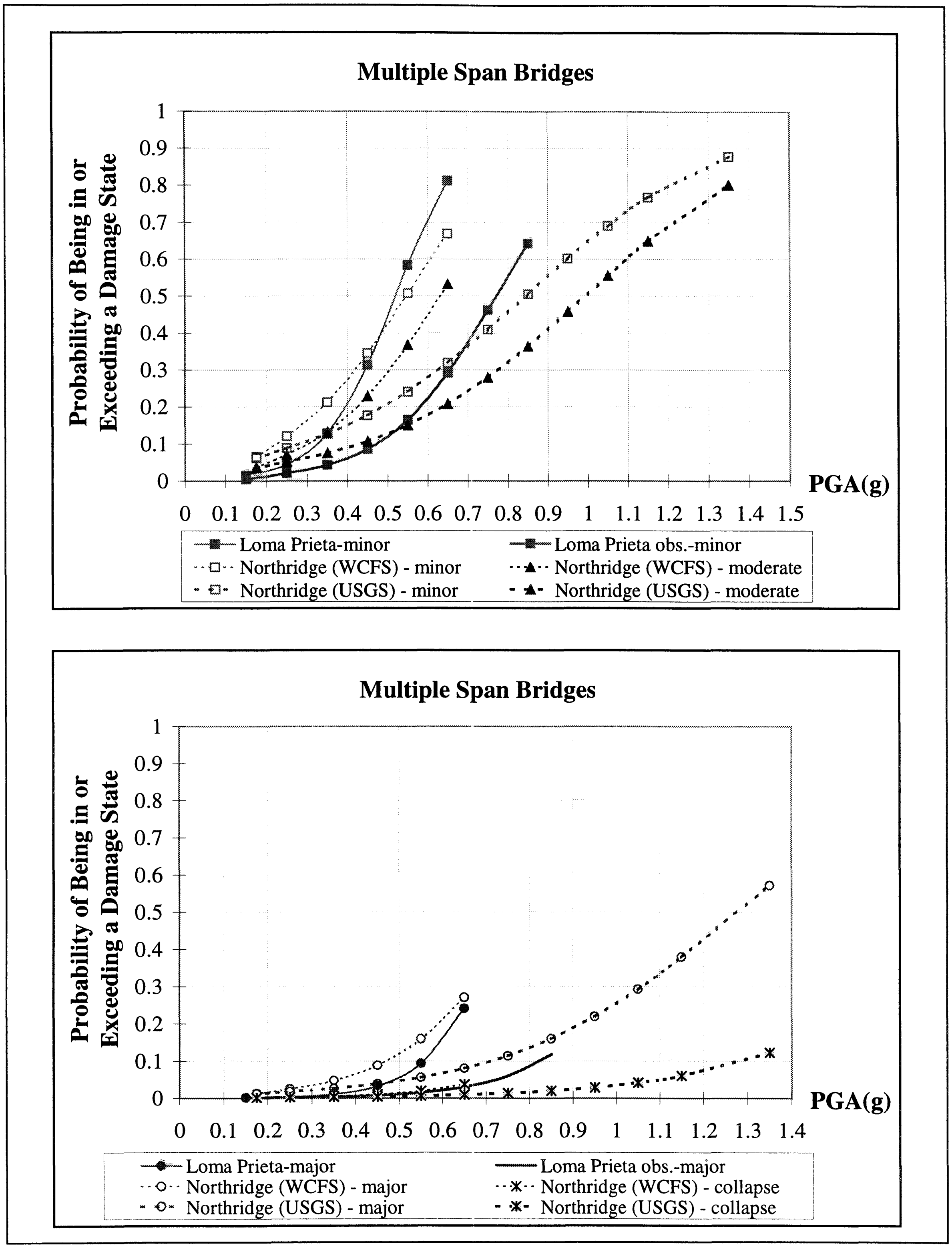


FIGURE 5-11 Empirical Fragility Curves for Multiple Span Bridges obtained from the Loma Prieta and Northridge Earthquakes (*correlation data set*) - Damage States

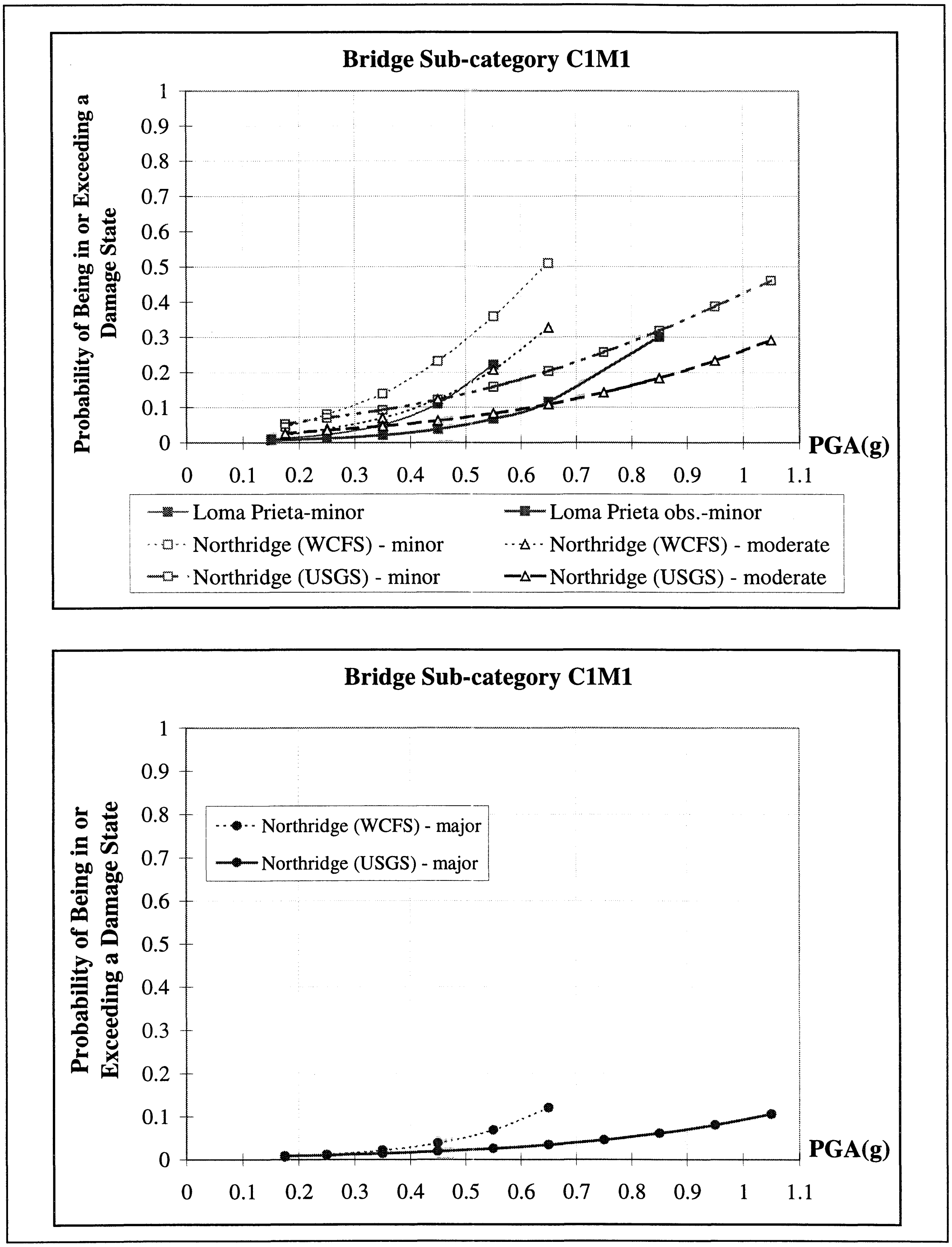
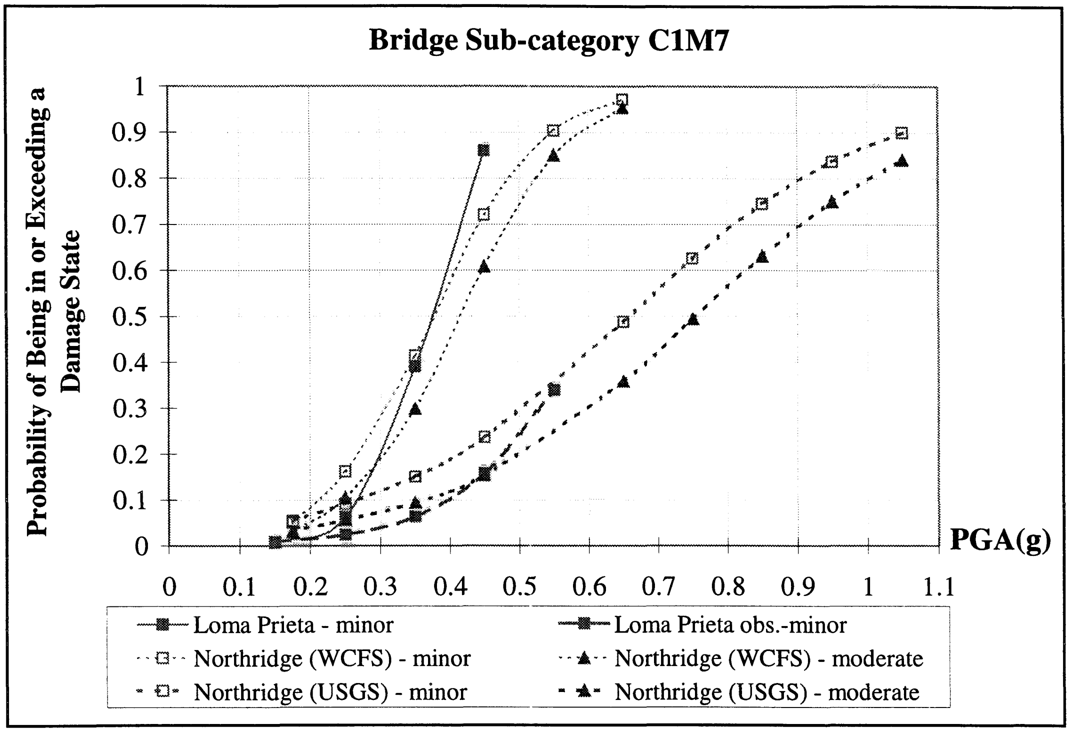


FIGURE 5-12 Empirical Fragility Curves for Bridges in Sub-category C1M1 obtained from the Loma Prieta and Northridge Earthquakes (*correlation data set*) - Damage States



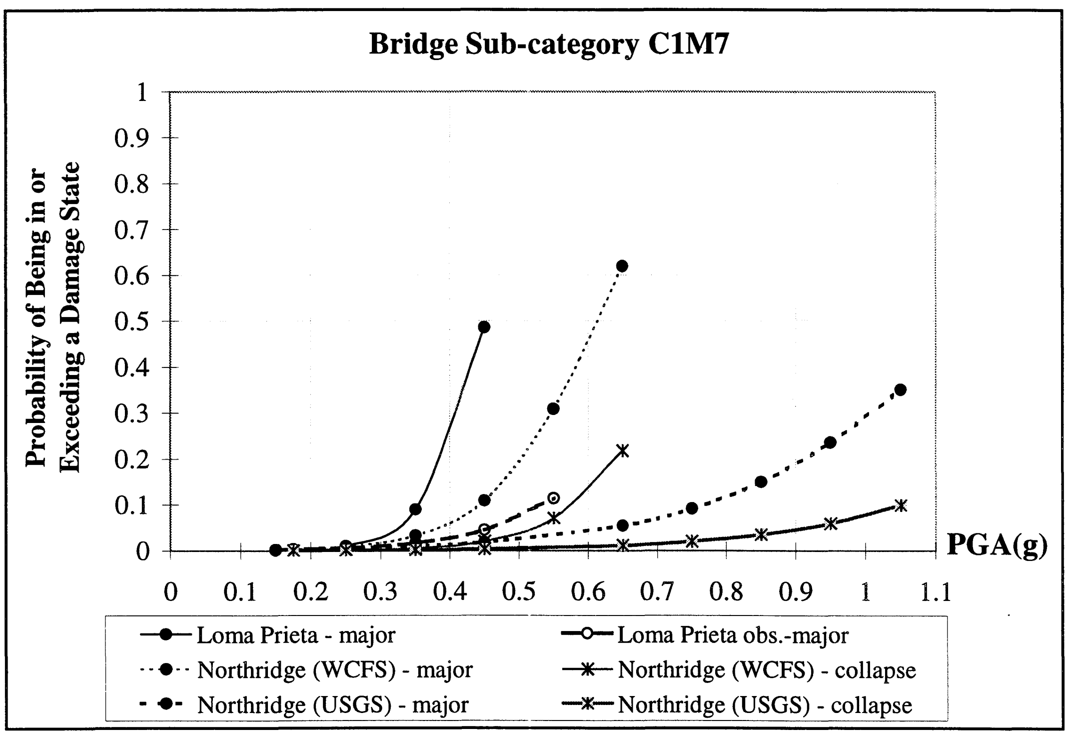


FIGURE 5-13 Empirical Fragility Curves for Bridges in Sub-category C1M7 obtained from the Loma Prieta and Northridge Earthquakes (*correlation data set*) - Damage States

given damage state can vary by as much as a factor of 4. For a given damage state, similar patterns were observed among the fragility curves based on WCFS PGA levels for the Northridge earthquake and those based on USGS PGA levels and "observed" PGA levels. This might be due to the maximum PGA levels observed at a bridge site, e.g., 0.74g for the Loma Prieta earthquake as estimated based on the Campbell attenuation relationship and 0.66g for the Northridge earthquake based on the WCFS data set. Note that among the four PGA levels, the set of "observed" Loma Prieta PGA levels is the least reliable estimate since it was based on a best fit curve of the available data points.

The probability of being in a given damage state showed some variation between the Loma Prieta and the Northridge earthquakes. Among other reasons, this can be due to different definitions of the damage states. For example, for bridge sub-categories C1M1 and C1M7, the probability of being in *minor damage* for the Loma Prieta earthquake was very close to the sum of the probabilities of being in *minor* and *moderate damage* states for the Northridge earthquake. For the multiple span bridges shown in figure 5-11, however, there is not an obvious relationship between the damage probabilities for the two earthquakes.

The damage data from both earthquakes were compared to the fragility curves provided in HAZUS [1997]. In both cases, the HAZUS fragility curves overestimated the exceedance probabilities for all damage states. The poor agreement between the observed and the predicted damage can be partially attributed to the size of the data set. In addition, damage data from a single earthquake are not expected to match exactly with heuristic fragility curves or DPMs, since these functions are intended to represent average values over many earthquakes. A more detailed analysis of the damage data, however, indicated that the structural characteristics considered in the available ground motion-damage relationships do not include super and substructure material and type. Both of these characteristics were found to be highly correlated to damage observed in Loma Prieta and Northridge earthquakes.

The total estimated repair cost for the bridges damaged in the Northridge earthquake was about two thirds of the Loma Prieta earthquake. However, 90 percent of the total estimated repair cost in the Loma Prieta earthquake was due to one structure: the Cypress Viaduct. The bridges that collapsed in the Northridge earthquake constituted 75 percent of all repair costs estimated after the earthquake. Figure 5-14 shows the distribution of repair cost by damage state for the Loma Prieta and the Northridge earthquakes.

Figure 5-15 shows the comparison of estimated repair cost by components for the two earthquakes. Column damage was the costliest in both earthquakes. In contrast to the Northridge earthquake, the repair cost due to abutment damage was negligible in the Loma Prieta earthquake. It should be noted that, the repair costs depicted in figure 5-15 are only a portion of the total estimated repair cost. The figure only includes repair costs for bridges with complete information for individual components, that is, there was no available breakdown by component for collapsed bridges.

Similar to damage states, empirical relationships between ground motion levels and repair cost ratios were developed for different levels of repair cost ratio. Figures 5-16 through 5-18 show the ground motion-repair cost ratio relationships for multiple span bridges and bridge sub-categories C1M1 and C1M7, respectively.

Repair cost ratios greater than 10 percent were not observed in the Loma Prieta earthquake. Therefore, the ground motion-repair cost ratio relationships are presented here only for the 0 percent and the 0 – 10 percent intervals. The probabilities of exceeding or reaching given repair cost ratio intervals were very close for the two earthquakes. Similar to the fragility curves for different damage states, similar patterns were observed among the ground motion-repair cost ratio relationships based on WCFS PGA levels for the Northridge earthquake and those based on USGS PGA levels and "observed" PGA levels. Note that for the sub-category C1M7, the ground motion-repair cost ratio relationship curve based on the set of "observed" Loma Prieta PGA levels did not satisfy the statistical goodness-of-fit criterion.

The estimated repair cost ratios for the bridges damaged in the Northridge earthquake were also compared with the ATC-13 DPMs. The ATC-13 DPMs were found to agree quite well with the observed damage and the repair cost ratios for low MMI values. In most cases for MMI values VII and higher, however, ATC-13 DPMs overestimated the observed values from the Northridge earthquake.

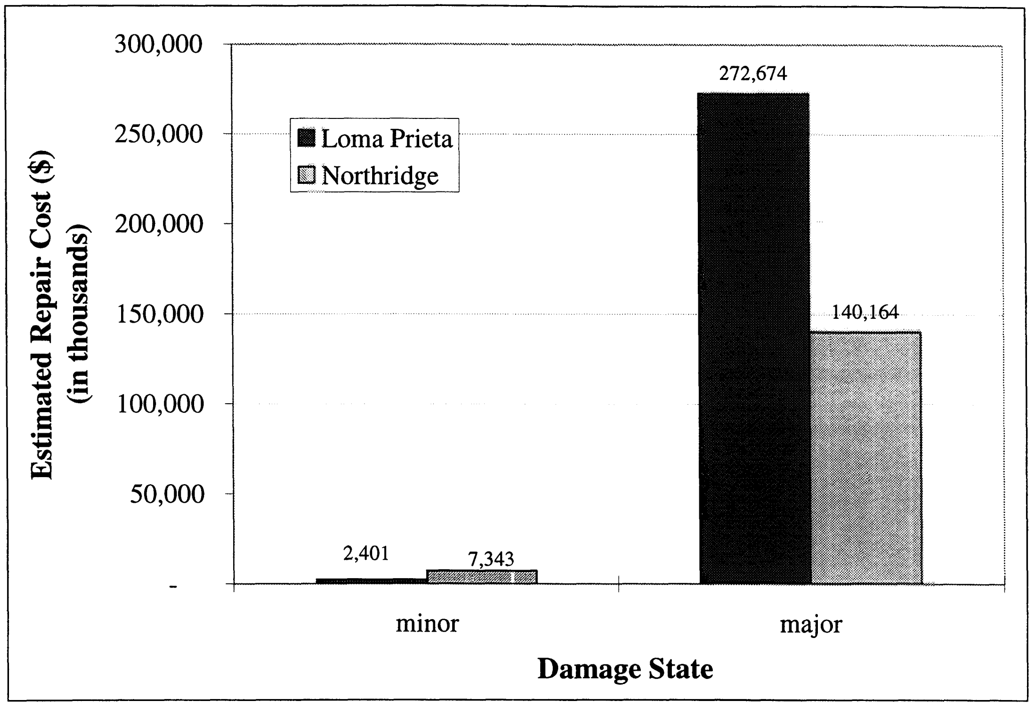


FIGURE 5-14 Estimated Repair Costs in the Loma Prieta and Northridge Earthquakes

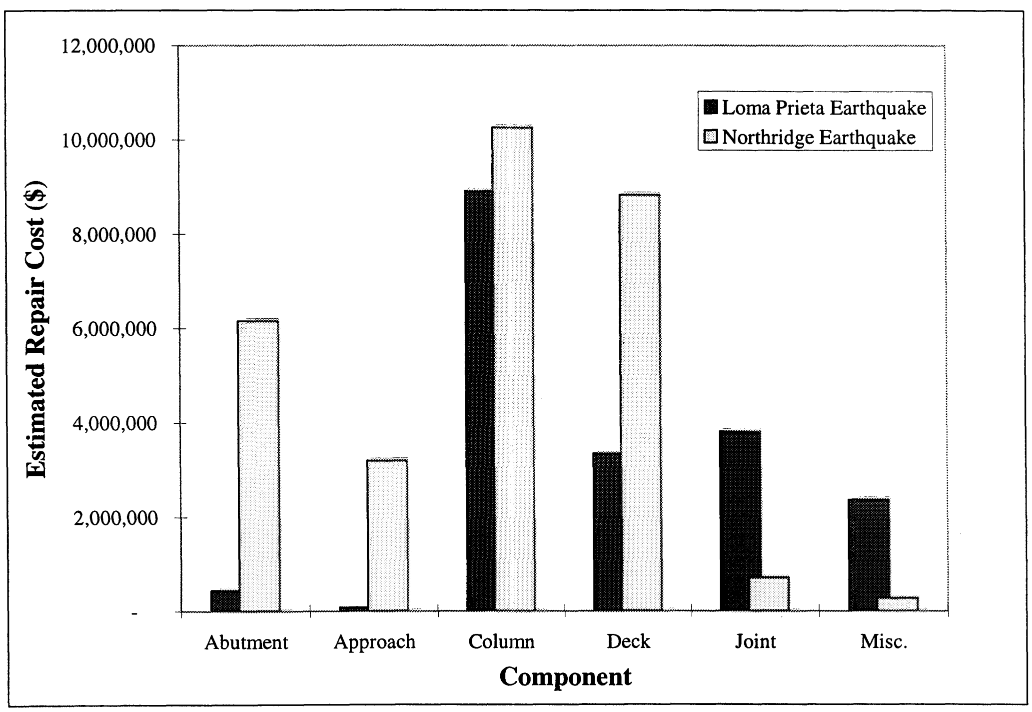


FIGURE 5-15 Estimated Component Repair Costs in the Loma Prieta and Northridge Earthquakes

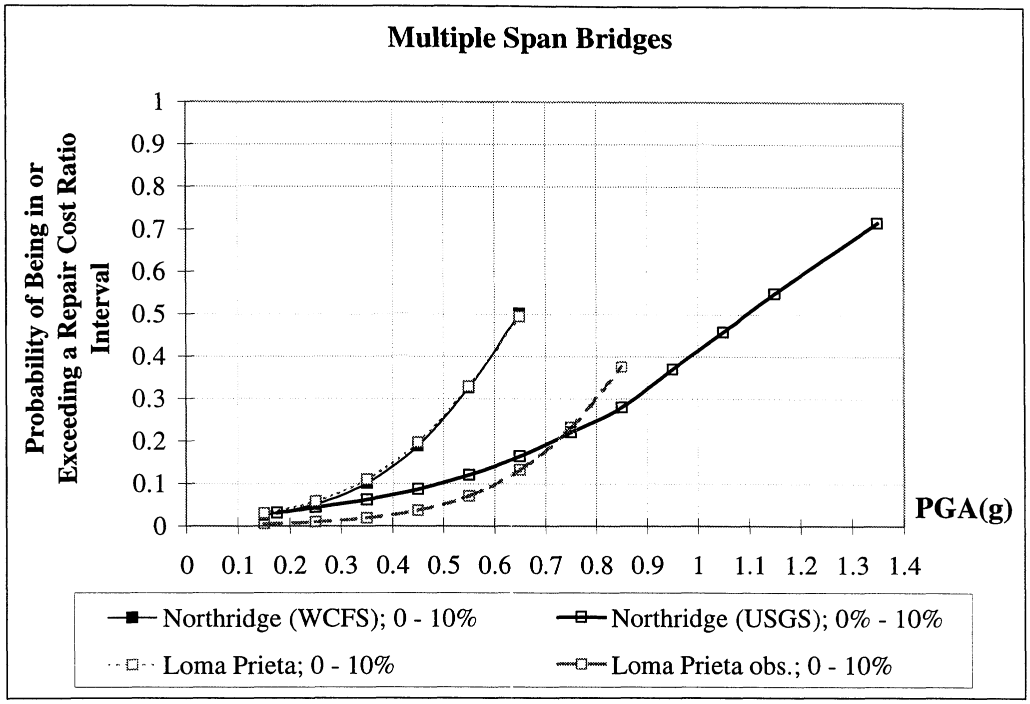


FIGURE 5-16 Empirical Fragility Curves for Multiple Span Bridges obtained from the Loma Prieta and Northridge Earthquakes (*correlation data set*) - Repair Cost Ratio

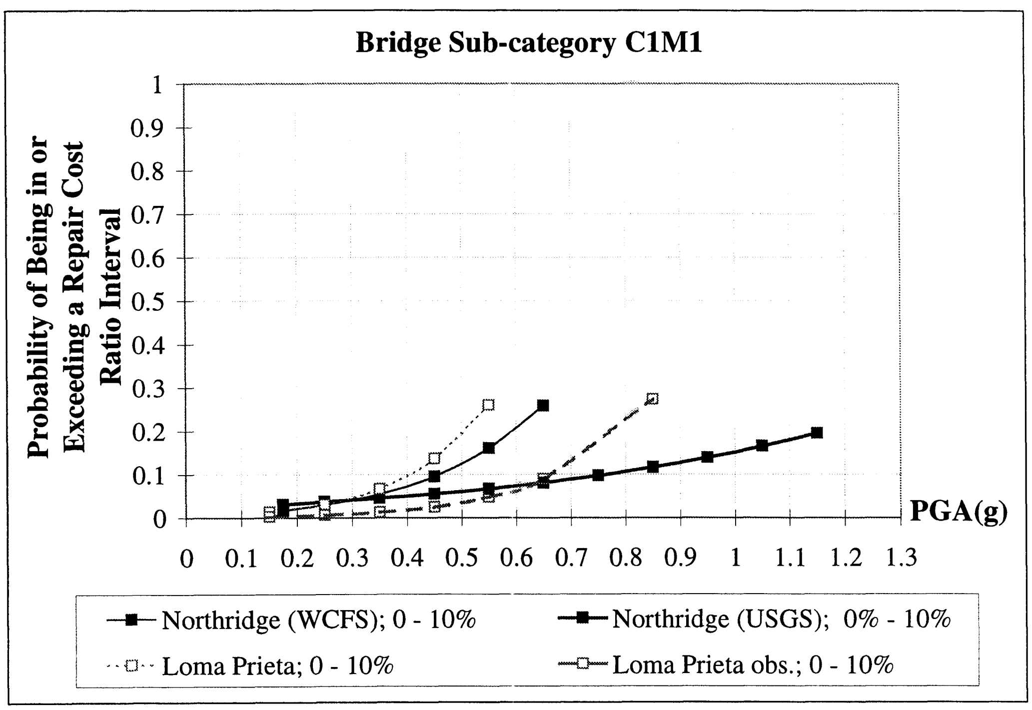


FIGURE 5-17 Empirical Fragility Curves for Bridges in Sub-category C1M1 obtained from the Loma Prieta and Northridge Earthquakes (*correlation data set*) - Repair Cost Ratio

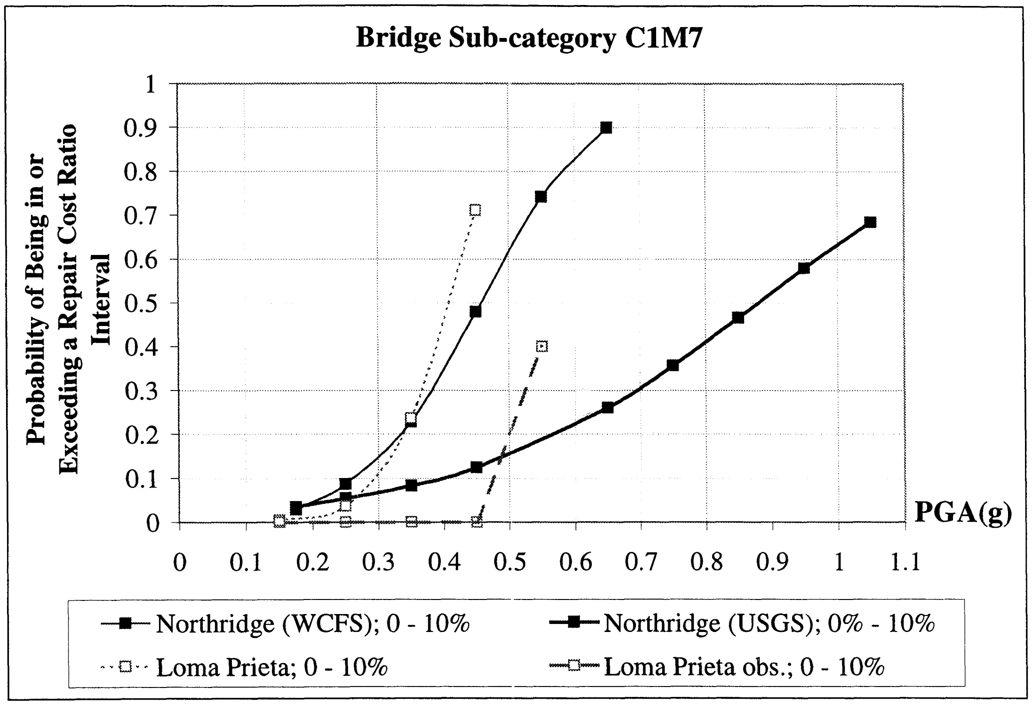


FIGURE 5-18 Empirical Fragility Curves for Bridges in Sub-category C1M7 obtained from the Loma Prieta and Northridge Earthquakes (*correlation data set*) - Repair Cost Ratio

SECTION 6
CONCLUSIONS AND CONTRIBUTIONS

The Loma Prieta and Northridge earthquakes provided valuable data on bridge damage and repair cost. These data were studied in detail with the objective of identifying the structural characteristics that most contribute to bridge damage and repair cost, and correlating ground motion levels with damage states and repair cost ratios. Data on bridge damage and ground motion levels were better documented after the Northridge earthquake in comparison to the Loma Prieta earthquake. Despite the difference in size and quality of data from the Loma Prieta and the Northridge earthquakes several conclusions can be drawn from the analyses:

- In both the Loma Prieta and the Northridge earthquakes, less than five percent of the bridges that were exposed to ground shaking were damaged. As experienced in the past earthquakes, bridges with non-monolithic abutments, discontinuous spans and single column bents performed poorly. High skew contributed to high damage levels. Performance of bridges designed and built before 1971 was poorer than those designed according to more recent standards. In both earthquakes, *collapse* of concrete bridges was observed while steel bridges experienced at most *major damage*[1].

- Two different sets of PGA values were reported for the Northridge earthquake (USGS and WCFS) and three for Loma Prieta (observed, Boore and Joyner, Campbell). At a given PGA level, the probability of being in or exceeding a damage state or repair cost ratio can vary as much as a factor of 4. More attention needs to be paid to the differences in fragility relations due to different assumptions regarding the PGA values.

---

[1] Note that the number of steel bridges exposed to ground shaking was significantly less.

- Different definitions for damage states were used in reporting damage after the Loma Prieta and the Northridge earthquakes. The four level damage state definitions used in the Northridge earthquake were found to be more representative of the observed damage compared to the two level definitions used in the Loma Prieta earthquake. Furthermore, inconsistencies were observed in the definitions of damage states.

- Currently available bridge classes and the corresponding *ground motion-damage relationships* do not properly estimate the observed damage from the Loma Prieta and the Northridge earthquakes. In this study, empirical ground motion-damage relationships were developed for a set of bridge sub-categories. Data, however, were not sufficient to develop empirical fragility curves for all of the sub-categories. Logistic regression analysis was found to be very effective in developing the empirical fragility curves. Although the fragility curves developed for the Loma Prieta and Northridge earthquakes showed similar trends, it should be recognized that they are based on data from only two earthquakes and they should be updated as more data become available. In order to update the empirical fragility curves using future damage data, consistent definitions for damage states are needed.

- Several structural characteristics such as, abutment type, column bent type and span continuity were used to group bridges. Bridge sub-categories based on the classification proposed by Basöz and Kiremidjian [1996] were adopted in this study. Ground motion-damage relationships and ground motion-repair cost ratio relationships were obtained for these sub-categories. In that study, the authors defined the least and the most vulnerable sub-categories for single and multiple span bridges. The observed damage data from the Loma Prieta and the Northridge earthquakes were used to validate the assumptions used in defining bridge classes. The data showed that bridges grouped under the proposed "least vulnerable sub-category" for single or multiple span bridges did in fact perform better than bridges in the proposed “most vulnerable sub-category".

  A more detailed analysis of the damage data indicated that the structural characteristics considered in the available ground motion-damage relationships do not adequately reflect the seismic vulnerability of bridges. The currently available bridge classes do not consider the effect of the structural material and type, substructure type, and design details, such as column reinforcement and/or seat width. The structural characteristics selected to define the bridge sub-categories were found to be highly correlated with the observed damage.

- The reliability of the bridge inventory data (Caltrans SMS database, 1993) used in this study was investigated, revealing incorrect attribute values for several characteristics. These attributes include abutment type, column bent type and design year. Errors in the abutment type data for bridges damaged in the Northridge earthquake were corrected based on bridge plans. The effect of data error on the probability of exceeding or reaching any given damage state was observed to be larger for smaller data sets as expected. It was beyond the scope and resources available for this project to correct all the erroneous attribute values in the Caltrans SMS database. The effect of data error might be considered to be negligible for a broad group of bridges, such as all multiple span bridges or all concrete bridges. However, the correction of the available bridge inventories is crucial for the vulnerability assessment of individual bridges or small groups of bridges and should be pursued further.

- The total estimated repair cost of $190 million for the bridges damaged in the Northridge earthquake was about two thirds of the repair cost estimated from the Loma Prieta earthquake. However, 90 percent of the total cost in the Loma Prieta earthquake was due to one structure: the Cypress Viaduct. The bridges that collapsed in the Northridge earthquake constituted 75 percent of all repair cost estimated after the earthquake. Columns were the most damaged component in both earthquakes. In contrast to the Northridge earthquake, the repair cost due to abutment damage was negligible in the Loma Prieta earthquake. In the Northridge earthquake, column and joint damage were mostly observed in bridges in *major* damage state. Although abutment damage was observed at all damage states, it was a major part for bridges that suffered *moderate* damage.

The results from this study can be used to assist in decisions for mitigation, such as prioritization of bridges for seismic retrofitting, and during post-earthquake response and recovery activities. More specifically the areas that can benefit from the results of this project include the following:

- *Database of bridge damage and repair cost*: A comprehensive database on bridge damage and repair cost was compiled for the two most recent major earthquakes with significant consequences in the United States. This database includes several structural characteristics that were used in the vulnerability assessment of bridges in this study. Many structural characteristics that are necessary to better estimate bridge vulnerability and to perform bridge specific vulnerability assessment are not included in any of the currently available bridge inventories. The database compiled in this study can be used in identifying the deficiencies of those inventories and setting priorities to improve them.

- *Classification of bridges and ground motion-damage relationships*: Analytical methods used to assess individual bridge vulnerabilities and to determine the damage state of a bridge under a particular ground shaking level require detailed information. Most of the time, however, this detailed information is not available and the data gathering is very time consuming and tedious. Besides, with the large number of bridges, it is rather difficult, if not impossible, to evaluate the seismic response of each individual bridge in detail. The classification used in this study, which is based on structural characteristics, can be used to assess the seismic vulnerability of a large bridge inventory when detailed data are not available.

Empirical fragility curves were developed based on the damage data from the Loma Prieta and the Northridge earthquakes. These fragility curves can be used to improve currently available ground motion damage relationships.

- *Damage state definitions:* The damage state definitions proposed in this study are based on observed damage descriptions and can be used to: (*i*) develop analytical fragility curves to assess the vulnerability of bridges, and (*ii*) to establish a post-earthquake investigation form that will assist in compiling comprehensive bridge damage data in a consistent manner.

- *Repair cost estimates*: The empirical repair cost-damage relationships developed in this study can be used to estimate the direct loss from damage to bridges in earthquakes. The repair cost data from the Northridge earthquake can be used to update the DPMs provided in ATC-13. Due to lack of data the estimated repair costs used in this study did not include costs from traffic closures. In order to evaluate the functionality of a highway system, restoration functions, i.e., relationships between damage state and repair time, are essential. Further research is necessary to develop these relationships based on the limited available data on repair time, and expert opinion.

The results obtained in this study provide a validation tool for analytical studies that are necessary to evaluate the vulnerability of individual bridges. These results are specific to California earthquakes and to California bridge design and construction practices. However, similar results can be obtained for other states by using subsets of bridge data compiled in this study which are representative of bridge design and construction practices outside of California.

SECTION 7
REFERENCES

1. Agresti, A. [1996]. *An Introduction to Categorical Data Analysis.* John Wiley and Sons, New York.

2. Astaneh-Asl, A., B. Bolt, K. M. McMullin, R. R. Donikian, D. Modjtahedi, S-W. Cho. [1994]. "Seismic Performance of Steel Bridges During the 1994 Northridge Earthquake." *Report to California Department of Transportation*, UCB/CE-Steel-94/01, (April).

3. ATC-13, [1985]. *Earthquake Damage Evaluation Data for California*, Report ATC-13, Applied Technology Council, Redwood City, California.

4. Basöz, N., and A. S. Kiremidjian. [1996]. *Risk Assessment for Highway Transportation Systems. Technical Report No. 1184.* John A. Blume Earthquake Engineering Center, Civil Engineering Department, Stanford University, Stanford, California.

5. Boore, D. M., W. B. Joyner, and T. E. Fumal. [1993]. "Estimation of Response Spectra of Peak Accelerations from Western North American Earthquakes: Interim Report." *USGS Open-File Report 93-509*, Menlo Park, California.

6. Boore, D. M, W. B. Joyner, and T. E. Fumal. [1997]. "Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes: A Summary of Recent Work." *Seismological Research Letters*, Vol. 68, No. 1, 128-153.

7. Buckle, I. G., and I. M. Friedland. [1996]. "Seismic Retrofitting Manuals for Highway Systems: System Assessment, Screening, Evaluation, and Retrofitting." *Third US-Japan Workshop on Seismic Retrofit of Bridges,* Osaka, Japan, (December).

8. Buckle, I., G. [1994]. *The Northridge, California Earthquake of January 17, 1994: Performance of Highway Bridges.* Technical Report NCEER-94-0008.

9. California Department of Transportation, (Caltrans). [1989]. *The Loma Prieta Earthquake.* Caltrans PEQIT Report, Division of Structures, Sacramento, CA.

10. California Department of Transportation, (Caltrans). [1993]. *OSM&I Coding Guide for SMS*. Division of Structures, Office of Structures Maintenance and Investigations, Sacramento, California.

11. California Department of Transportation, (Caltrans) [1994a], *The Northridge Earthquake*, Caltrans PEQIT Report, Division of Structures, Sacramento, CA.

12. California Department of Transportation, (Caltrans) [1994b]. *Supplementary Bridge Damage Reports*. Division of Structures, Sacramento, CA.

13. Campbell, K. W. (1991). "An Empirical Analysis of Peak Horizontal Acceleration for the Loma Prieta, California, Earthquake of 18 October 1989." Bulletin of Seismological Society of America, Vol. 81, No. 5, 1838-1858.

14. Dewey, J. W., B. G. Reagor, and L. Dengler. [1994]. "Isoseismal Map of the Northridge, California, Earthquake of January 17, 1994." *Poster talk presented at the 89th Annual Meeting of the Seismological Society of America*. Pasadena, California, (April).

15. Earthquake Engineering Research Institute, [EERI]. [1995]. *Northridge Earthquake of January 17, 1994 Reconnaissance Report*. Earthquake Spectra: Vol. 11, Supplement C, (April), 287-372.

16. HAZUS. [1997]. *Standardized Earthquake Loss Estimation Methodology*. National Institute of Building Sciences, Prepared by Risk Management Solutions, Inc., Menlo Park, California.

17. Hosmer, D. W., and S. Lemeshow. [1989]. *Applied Logistic Regression*. John Wiley and Sons, New York.

18. SAS/STAT User's Guide [1995]: Basics, Release 6.07 Edition. Gary, NC: SAS Institute.

19. Singhal, A. and A. S. Kiremidjian. [1997]. "Bayesian Updating of Fragilities with Application to RC Frames." *Accepted for publication in Journal of Structural Engineering, ASCE.*

20. Somerville, P., C. Saikia, D. Wald, and R. Graves. [1996]. "Implications of the Northridge Earthquake for Strong Ground Motions from Thrust Faults." *Bulletin of Seismological Society of America*. Vol. 86, No. 1B, (February). S115-S125.

21. USGS. [1994]. US Geological Survey, *Open-File Report 94-197.* Menlo Park, California.

22. Thiel, C. (editor). [1990]. "Competing Against Time." Report to Governor George Deukmajian from the Governor's Board of Inquiry of the 1989 Loma Prieta Earthquake.

23. Werner, S. D., C. E. Taylor and J. E. Moore II. [1996]. "Seismic Risk Analysis of Highway /Roadway Systems." Vol. I Strawman Report for Research Project Seismic Vulnerability of Existing Highway Construction, prepared for NCEER (October).

24. Woodward-Clyde Federal Services (WCFS). [1995]. *Contoured Ground Motion Parameters for the 1994 Northridge Event.* ASCII files.

25. Yashinsky, M. [1995]. "Northridge Earthquake, Lifeline Performance and Post-Earthquake Response: Bridges and Roadways." *TCLEE Monograph No. 8*, ed. A. J. Schiff, (August).

26. Yashinsky, M., P. Hipley, and Q. Nguyen. [1995]. *The Performance of Bridge Seismic Retrofits During the Northridge Earthquake.* Caltrans Office of Earthquake Engineering, Sacramento, California.

APPENDIX A

QUESTIONNAIRE ON DAMAGE STATES FOR CONCRETE BRIDGES

The objective of this questionnaire is to capture expert opinion on damage definitions for components of concrete bridges. The results can be used in developing analytical fragility curves to assess the vulnerability of bridges, and/or developing a post-earthquake investigation form that will assist to compile bridge damage more consistently than the current practice.

In general, different structures experience different damage types for a given level of ground shaking. For vulnerability assessment of bridges, the damage state of a bridge can be expressed by one of the five qualitative assignments: *no damage*, *minor damage*, *moderate damage*, *severe damage* and *collapse*. The damage state of a bridge is a function of the damage state of its components, such as abutments, superstructure and substructure. In this questionnaire, a set of preliminary damage states for components of concrete bridges are defined based on observed bridge damage in the past earthquakes, mainly the Northridge earthquake. The damage states for: (*i*) abutments, (*ii*) substructures (column bents and/or pier walls), and (*iii*) connections and bearings of concrete bridges are listed respectively in Tables 1 through 3.

In Tables 1 through 3, the first column shows the damage state index, *I*, which is an indication of the severity of the damage. The index *I* increases with increasing severity of the damage. The descriptions for possible damage to the component, i.e., the column, connections and bearings, or the abutments, are listed in column 2. In order to fill in columns 3, 4 and 5, please answer the following questions:

Column 3 -**Agree? (yes/no)**- : Would you agree with the damage state index assigned to each damage description,

Column 4 -**New Index**- : If your answer in column 3 is *no*, what damage state index would you assign to the given damage description?

Column 5 -**Remarks**- : List any other damage descriptions that should be included for the given damage severity index of the respective component.

Please, provide any other comments and/or suggestions that you might have.

Thank you for your time and interest.

July 1996, Nesrin Basöz, Ph.D.

Post-doctoral Researcher
The John A. Blume Eqk. Eng. Ctr
Department of Civil Engineering
Stanford University
Stanford, CA 94305-4020

Phone: (415) 725-3168
Fax: (415) 725-9755
E-mail: nesrin@leland.stanford.edu

Your name: ____________________________

List of the earthquakes that you have been involved in post-earthquake bridge damage inspection:

____

Damage State Index	Description		Agree?	New Index	Remarks
1	a) hairline cracks in columns	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		at column faces.
1	b) cracks at column top and bottom (1/16" to 1/8") and minor spalling	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	N	3	
		Expert 5	Y		
		Expert 6	N	1	
		Expert 7	Y		
		Expert 8	Y		should be < 1/8"
1	c) spalls at column faces (2' 2')	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	N	3	
		Expert 5	Y		
		Expert 6	N	3	
		Expert 7			Specify depth; 1" or less in depth :1, >1" in depth : 2
		Expert 8	Y		spalls in column face(s) (2' 2')
1	d) spalled column flares ({flare height/column height} ratio < 1/3 or	Expert 1	Y		
	adequate reinforcement exists)	Expert 2	Y		As log as spiral protects concrete - flare is sacrificed
		Expert 3	Y		
		Expert 4	N	3	
		Expert 5	Y		
		Expert 6	N	3	
		Expert 7	N	2	
		Expert 8	Y		

Damage State Index	Description		Agree?	New Index	Remarks
1	e) cracks at bent cap connection	Expert 1	Y		
		Expert 2	N		Depends on size - can be serious
		Expert 3	Y		
		Expert 4	N		Depending on crack size
		Expert 5	N	2 or 3	"cracks" very general; depending on connection detail
					this could be very serious
		Expert 6	Y	2	
		Expert 7			Specify size of crack and crack density. For hairline
					cracks: 1, for larger cracks: 2
		Expert 8	Y		In columns or in caps? Isn't this table limited to columns?
2	a) shear cracks	Expert 1	Y		
		Expert 2	Y		
		Expert 3			
		Expert 4	N	4 or 5	
		Expert 5	Y		
		Expert 6	Y	2	
		Expert 7	Y		Crack depth and density must be specified. 1/8" cracks or
					wider, <1' apart
		Expert 8	Y		< 1/8"
2	b) spalled column flares ({flare height/column height} ratio > 1/3 or	Expert 1	Y		
	inadequate reinforcement exists)	Expert 2	Y		see comment for 1d
		Expert 3	N	1	We design column flares to be damaged
		Expert 4	N	3	
		Expert 5	~Y		
		Expert 6	N	3	
		Expert 7	Y		
		Expert 8	Y		"severely spalled column flares"
2	c) flexural failure (formation of plastic hinges, buckling of longitudinal	Expert 1	Y		
	reinforcement over a length of one column diameter)	Expert 2	Y		Beginning of flexural damage, bird caging of concrete steel
		Expert 3	Y		
		Expert 4	N	4 or 5	Severe damage

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 5	~N	~3	In single column bents: very serious. Also aftershock
					earthquake resistance is diminished for all configurations.
					Rectangular single column bents or multiple hinges on
					multi-column bents: index = 3
		Expert 6	N	5	
		Expert 7	N	3	
		Expert 8	Y		"slight buckling of longitudinal reinforcement..."
2	d) cracks exposing core	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	N	3	
		Expert 5	Y		
		Expert 6	N	4	
		Expert 7	Y		Assuming no broken rebar
		Expert 8	Y		
3	a) shear failure	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	N	4 or 5	Severe damage
		Expert 5	Y		
		Expert 6	N	4	Needs to be explained
		Expert 7	Y		
		Expert 8	Y		upper portion offset from lower portion > 1/2"
3	b) flexural failure (without formation of a plastic hinge due to inadequate	Expert 1	Y		
	confinement steel -due to steel rupture or broken welds-, inadequate	Expert 2	Y		
	anchorage of the steel, inadequate lap splices)	Expert 3	Y		
		Expert 4	N	4 or 5	Severe damage
		Expert 5	Y		
		Expert 6	N	5	
		Expert 7	Y		Any flexural failure would be I = 3.
		Expert 8	Y		"severel buckled bars"

Damage State Index	Description		Agree?	New Index	Remarks
3	c) vertical pull of the longitudinal column reinforcement	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		But usually occurs only at footing
		Expert 4	N		unclear
		Expert 5	?	?	Very general term; pullout (debound) - serious;
					elastic/plastic strain not necessarily so.
		Expert 6	N	5	
		Expert 7	Y		Assume that "pull" means elongation of rebars. For this
					to occur many other characteristics would have to exist
		Expert 8	?		
3	d) ground displacement at column base	Expert 1	Y		
		Expert 2	N		This is seen for very minor damage
		Expert 3	N	1	Depends on degree or magnitude of displacement.
					On 5/14 we had one 1 foot of ground displacement
					and no damage at that bent.
		Expert 4	N		Depends on amount of movement
		Expert 5	~N	2 or 3	Could mean nothing or indicate elastic rocking of
					foundation (no big deal) - or could mean hinge below
					ground level (again, very general)
		Expert 6	Y	3	
		Expert 7	N	1	Ground displacement was seen around almost every column
		Expert 8	?		
3	e) tilting of substructure due to foundation failure	Expert 1	Y		
		Expert 2	Maybe		How much beyond capacity?
		Expert 3	Y		
		Expert 4	N	4 or 5	
		Expert 5	Y		
		Expert 6	N	4	
		Expert 7	Y		
		Expert 8	Y		"out of plumb column failure (pulverized section of column
					with little horizontal offset) due to foundation failure

Damage State Index	Description		Agree?	New Index	Remarks
1	a) cracks in shear keys	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	2	Expand on type of key, some are more important than
					others
		Expert 7	Y		
		Expert 8	Y		
1	b) cracks in barrier rail	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
1	c) minor wing wall (WW) cracking	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
1	d) cracks in closure wall	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
1	e) minor curb spalling	Expert 1	Y		
		Expert 2	Y		

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
1	f) minor crack and spalling at the abutment	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	2	The spalling forces it into next index
		Expert 7	Y		Assuming no bearing loss
		Expert 8	Y		
1	g) cracked slope paving	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
2	a) spalling of soffit (20"x8"x1/2")	Expert 1	Y		
		Expert 2	Y		
		Expert 3	N	1	Not serious
		Expert 4	N	3 or 4	Depending on cause of spall
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		OK but we never observed this
2	b) pile cap damage	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		But it depends on size of crack
		Expert 4	N	3	Depends on damage. Pile cap cannot be
					easily inspected.

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 5	?	?	cannot tell what degree of damage this, nor can one
					see the connection
		Expert 6			More detail (could be 3)
		Expert 7	Y		Need more detail
		Expert 8	Y		
2	c) diaphragm crack	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		But it depends on size of crack
		Expert 4	Y		
		Expert 5	~Y	1 or 2	could be nothing; intermediate diaphragms can
					tolerate damage without affecting integrity
		Expert 6	Y		
		Expert 7	N	1	Size, depth and density
		Expert 8	Y		
2	d) curtain wall cracking	Expert 1	Y		
		Expert 2	Y		
		Expert 3	N	1	
		Expert 4	Y		
		Expert 5	N	1	Non-structural element; ensure ....
		Expert 6	N	1	
		Expert 7	N	1	
		Expert 8	N	1	
2	e) end diaphragm damage	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6			Needs to be expleianed; what kind of damage?
		Expert 7	N	1	Need more detail
		Expert 8	Y		
2	f) moderate WW cracking and spalling	Expert 1	Y		
		Expert 2	Y		
		Expert 3	N	1	
		Expert 4	Y		
		Expert 5	Y		

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
2	g) minor settlement of the approach slab (~ £ 6")	Expert 1	Y		
		Expert 2	Y		
		Expert 3	N	1	
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	1	
		Expert 7	Y		
		Expert 8	Y		
2	h) small movement of the abutment (~1"-2")	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	1	
		Expert 7	Y		
		Expert 8	Y		"moderate"
2	i) extensive cracking and spalling of shear keys	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	N	3	
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
2	j) cracks and spalls in the abutment seat	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	N	2 to 4	
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		

Damage State Index	Description		Agree?	New Index	Remarks
2	k) backwall cracking	Expert 1	Y		
		Expert 2	Y		
		Expert 3	N	1	Depends on crack size
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	1	
		Expert 7	N	1	
		Expert 8	Y		
2	l) anchor bolt damage (no breaks)	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	1	
		Expert 7	N	1	
		Expert 8	Y		
2	m) shear key damage (no failures)	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4			Define damage, crack size
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	N	1	
		Expert 8	Y		What is the difference between this and 2i?
3	a) shear key failure	Expert 1	Y		
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	N	4	Assuming an offset is present
		Expert 8	Y		3"+ offset
3	b) WW backwall separation > 0.2 ft.	Expert 1	Y		
		Expert 2	OK		
		Expert 3	N	2	

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 4	N	4	would probably result from severe damage elsewhere
		Expert 5	~N	2	again, no real effect to structural integrity
		Expert 6	Y		
		Expert 7	N	2	
		Expert 8	Y		
3	c) pull out of restrainers from the backwall	Expert 1	Y		could combine with 3d and 3e
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	4	Depends on bridge geometry
		Expert 7	Y		
		Expert 8	Y		
3	d) damage to restrainer hardware	Expert 1	Y		could combine with 3c and 3e
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	~Y/N	~2?	Depends on degree of damage; quantify
		Expert 6			What kind of damage? Total failure?
		Expert 7	N	2	
		Expert 8	Y		
3	e) longitudinal restrainer failure	Expert 1	Y		could combine with 3c and 3d
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	4	Depends on bridge geometry
		Expert 7	N	2	
		Expert 8			
		Expert 9			
3	f) rocker bearing failure due to breaking of keeper plates	Expert 1	Y		
		Expert 2	OK		
		Expert 3	N	2	
		Expert 4	Y		
		Expert 5	~N	~2	They all break, with little effect to structural load
					capacity

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 6	N	2	
		Expert 7	N	2	
		Expert 8	Y		
3	g) bearing pedestal and anchorage failure	Expert 1	Y		
		Expert 2	OK		
		Expert 3	Y		
		Expert 4			
		Expert 5	~N	~2	see above; not considered serious
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		What does "anchorage" refer to?
3	h) rotation and displacement of the rocker bearings (~2")	Expert 1	Y		
		Expert 2	OK		
		Expert 3	N	2	
		Expert 4	N	2	
		Expert 5	~N	~2	likewise above
		Expert 6	N	2	
		Expert 7	N	2	
		Expert 8	Y		
3	i) moderate movement of the abutment (~2"-10")	Expert 1	Y		
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	N	3 or 4	10" would probably indicate moderate pile damage
		Expert 5	Y		
		Expert 6			Which direction, longitudinal/lateral? up/down?
		Expert 7	Y		
		Expert 8	Y		
3	j) approach slab rotation	Expert 1	Y		in hinge plane?
		Expert 2	OK		
		Expert 3	N	2	
		Expert 4	N	2	
		Expert 5	N	2	Bridge usually is OK. This only effects approached and
					is easily repaired with ....
		Expert 6	N	2	
		Expert 7	N	2	
		Expert 8	Y		

Damage State Index	Description		Agree?	New Index	Remarks
3	k) WW breakage	Expert 1	Y		
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	N	2	
		Expert 5	~N	~2	Does not usually affect bridge capacity
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
3	l) large approach settlement (~ 1 ft)	Expert 1	Y		is this same as 4d?
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	N	2	
		Expert 5	-		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
3	m) large spalls under girders	Expert 1	Y		
		Expert 2	OK		
		Expert 3	N	2	
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		"large spalls on girder soffits"
3	n) joint seal failure	Expert 1	Y		
		Expert 2	OK		
		Expert 3	N	1	
		Expert 4	N	2	
		Expert 5	~N	1, 2 or 3	Really depends on many factors; compression seals
					can be destroyed by normal earthquake movements;
					same with assemblies
		Expert 6	N	1 or 2	
		Expert 7	N	2	
		Expert 8	N	2	
4	a) vertical and/or lateral offsets	Expert 1	Y		I believe 4 should identify damage that would

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 2	OK		close the bridge and that 5 indicates
					damage that would kill travelers.
					Add a damage state 5. The worst damage is
					if the structure comes off the abutment seat or
					settlement that could kill motorists hitting the
					abutment.
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	~N	2	Many vertical/lateral offsets at abutments with no
					indications of damage or reduction in capacity
		Expert 6	N		Depends on how much? Is there bearing failure?
		Expert 7	Y		For >2"-4"; less than 2" I = 3
		Expert 8	Y		
4	b) tilting and movement (>10") of abutments implying	Expert 1	Y		
	foundation problems	Expert 2	OK		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	3	
		Expert 7	Y		
		Expert 8	Y		
4	c) foundation failure (e.g. tilting, severe pile damage)	Expert 1	Y		
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
4	d) settlement of backfills (> 1')	Expert 1	Y		
		Expert 2	OK		
		Expert 3	N	3	
		Expert 4	Y		
		Expert 5	N	2	See (I) above
		Expert 6	N	3	
		Expert 7	N	3	
		Expert 8	Y		

Damage State Index	Description		Agree?	New Index	Remarks
4	e) large crack or broken concrete at the abutments	Expert 1	Y		
		Expert 2	OK		
		Expert 3	N	3	
		Expert 4	Y		
		Expert 5	~N	3 or 4	Abutments can sustain damage of this nature and
					remain serviceable; quantify
		Expert 6	Y		
		Expert 7	N	3	
		Expert 8	Y		"large crack (> 1/2") or severely fractures concrete at
					the abutments

Damage State Index	Description		Agree?	New Index	Remarks
1	a) railing cracks	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		should be in miscellaneous category
		Expert 7	Y		
		Expert 8	Y		
1	b) slight movement	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6			needs definition
		Expert 7	Y		
		Expert 8	Y		
1	c) settlement at hinges (~1/2")	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	N	2 or 3	
		Expert 5	Y		
		Expert 6	Y		concrete <=1/2"
		Expert 7	Y		
		Expert 8	N		any settlement is bad - higher category
1	d) minor spalling	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
1	e) cracks in hinges	Expert 1	Y		
		Expert 2	Y		
		Expert 3	Y		

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 4	N	2 or 3	
		Expert 5	Y		
		Expert 6	N		need size of cracks to determine index
		Expert 7			Depends on size. <1/8" I = 1; >1/8" I = 2 or 3
		Expert 8	Y		
2	a) shear key failure	Expert 1	N		index level 2 is not consistent with abutments
					(which has keys and bearings). Siggest merging
					the two.
		Expert 2	?		What does failure mean? There is always going
					to be steel sticking up.
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	3	interior or exterior?
		Expert 7	N	3	
		Expert 8	Y		displacement >= 2"
2	b) keeper bar failure without unseating	Expert 1	N		
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	~N	1 or 2	They all fail with little effect to capacity
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
2	c) damage to restrainer hardware	Expert 1	N		
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	~Y		
		Expert 6	N		what kind of damage?
		Expert 7	Y		
		Expert 8	Y		
2	d) longitudinal restrainer failure	Expert 1	N		
		Expert 2	OK		
		Expert 3	Y		

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 4	Y		
		Expert 5	N	3 or 4	No earthquake resistance left and can imply
					unseating
		Expert 6	N	3	Depends on bridge geometry
		Expert 7	N	3	
		Expert 8	Y		
2	e) rocker bearing failure due to breaking of keeper plates	Expert 1	N		
		Expert 2	Y		
		Expert 3	Y		
		Expert 4			Define failure
		Expert 5	~N	1 or 2	See 2b above
		Expert 6	N	3	Did the bearings fall? what kind of failure?
		Expert 7	Y		
		Expert 8	Y		
2	f) bearing pedestal and anchorage failure	Expert 1	N		without too much movement. You have to be
		Expert 2	Y		concerned about the aftershocks! Should you
					close the bridge? put in a falsework?
		Expert 3	N	3	
		Expert 4	N	3	
		Expert 5	~Y		
		Expert 6	N	3	
		Expert 7	Y		
		Expert 8	Y		
2	g) rotation and displacement of the rocker bearings (~2")	Expert 1	N		
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	N	3	
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		Difference between this and 2e?
2	h) misalignment of finger joints	Expert 1	N		
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	Y		

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
2	i) tearing of anchorage and fastings out of the deck	Expert 1	N		
		Expert 2	?		When does this become a safety concern?
		Expert 3	Y		
		Expert 4	?		
		Expert 5	Y		
		Expert 6	N	3	Depends on bridge geometry
		Expert 7	Y		
		Expert 8	N	3	
2	j) shattering of rail and overhang spalling at hinges	Expert 1	N		
		Expert 2	?		When does this become a safety concern?
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	Y		
		Expert 7	Y		
		Expert 8	Y		
2	k) residual movements at the expansion or movement joints	Expert 1	N		
	(less than half of available seat width)	Expert 2			You should be concerned about the aftershocks!
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	~N	2 or 3	Depends on location and redundancy
		Expert 6	N		Some seats are 24" to 36", define seat size
					and/or length of movement
		Expert 7	Y		
		Expert 8	Y		
2	l) cracking of girder seats	Expert 1	N		
		Expert 2			What is that?
		Expert 3	Y		
		Expert 4	N	3	
		Expert 5	Y		

Damage State Index	Description		Agree?	New Index	Remarks
		Expert 6	N		What size of cracks? light to medium: 1,
					heavy: 2
		Expert 7			Depends on size. 1/8" or less in width I = 2
					1/8" or more in width I = 3
		Expert 8	Y		
2	m) soffit spall	Expert 1	N		
		Expert 2	OK		
		Expert 3	Y		
		Expert 4	Y		Depending on cause
		Expert 5	Y		
		Expert 6	N		What is the cause of the spall?
		Expert 7	Y		
		Expert 8	Y		
2	n) joint seal damage (no breaks)	Expert 1	N		
		Expert 2	OK		
		Expert 3	N	1	
		Expert 4	Y		
		Expert 5	~N	1 or 2	Depends on degree of damage - almost all
					get some damage
		Expert 6	N	1	could be old condition
		Expert 7	N	1	
		Expert 8	Y		"joint seal damage (type A split; type B loses
					compression"
3	a) residual movements at the expansion or movement joints	Expert 1	-		See note for ondex level 2
	(more than half of available seat width)	Expert 2	-		
					Again, 4 should indicate closure of bridge and
					5 should mean death to passengers.
		Expert 3	Y		4: cannot drive on bridge due to bearing failure,
					extreme hinge opening, etc.
		Expert 4	Y		
		Expert 5	~Y		
		Expert 6	N	4	Bridge should be closed
		Expert 7	Y		
		Expert 8	Y		

Damage State Index	Description		Agree?	New Index	Remarks
3	b) tearing of modular joints	Expert 1	-		
		Expert 2	-		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	~Y		
		Expert 6	Y		
		Expert 7	N	2	
		Expert 8	Y		
3	c) large spalls (concrete broken due to pounding)	Expert 1	-		
		Expert 2	-		
		Expert 3	N	2	
		Expert 4	Y		
		Expert 5	~N	2 or 3	Depends on size and location
		Expert 6	Y		But where the spalls occur matter
		Expert 7	N	4	More detail, bearing capacity is the key
		Expert 8	Y		
3	d) joint seal failure	Expert 1	-		
		Expert 2	-		
		Expert 3	N	2	
		Expert 4	N	2	
		Expert 5	~N	2 or 3	Depends on joint
		Expert 6	N	2	Seals are cheap and don't affect the integrity
					of the bridge
		Expert 7	N	2	
		Expert 8	Y		
3	e) differential settlement (~2")	Expert 1	-		
		Expert 2	-		
		Expert 3	Y		
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N		Need to know location and direction. >2": 2
					>4": 3 (vertical? horizontal? longitudinal?)
		Expert 7	Y		
		Expert 8	Y		

Damage State Index	Description		Agree?	New Index	Remarks
4	a) span drop due to insufficient seat width	Expert 1	-		
		Expert 2	N		should be 5. I saw expansion joint devices
					(metal fingers) sticking out a foot above the
					roadway (in Kobe). Automobiles had crashed
					into them ejecting the driver through the windshied.
		Expert 3	Y		Broken large bearings.
		Expert 4	Y		
		Expert 5	Y		
		Expert 6	N	5	State 5 should be bridge down state, state 4
					should be bridge closed
		Expert 7	Y		
		Expert 8	Y		
4	b) hinge restrainer or equalizing bolt failure	Expert 1	-		
		Expert 2	N		
		Expert 3	N	3	Who cares?
		Expert 4	N	1	Fail routinely due to thermal movements
		Expert 5	Y		Assuming unseated
		Expert 6	N	2	The equilizer bolts are for thermal equalizing
					only and will fail in the smallest earthquake
		Expert 7	N	3	
		Expert 8	Y		"hinge restrainer and equalizing bolt failure"

APPENDIX B

BRIDGE INVENTORY CODES FOR STRUCTURES OTHER THAN HIGHWAY BRIDGES

Attribute	Inventory Code	Description
County suffix	M	Maintenance station or other Caltrans Facility
	R	Safety roadside rest
	S	Sign
	T	Toll plaza
	W	Scale or inspection facility
	X	Transit facility or park-and-ride lot
Bridge number suffix	M	Buried hazard or miscellaneous structures
	Y	Structure does not serve state highway traffic
	W	Drainage pumping plants
Type of service (on the structure)	3	Railroad
	4	Pedestrians only
	0	Building or plaza
	B	Miscellaneous structure
	C	Retaining walls
	D	Pedestrians over an obstacle other than a road
	J	Other
	K	Pipelines, runways, etc.
	X	Grade crossing
	W	Drainage pumps
Structure closed or open	G	New structure not yet open to traffic
	K	Bridge closed to all traffic
	M	Closed or partially closed

APPENDIX C

EMPIRICAL FRAGILITY CURVES

The statistical software package SAS was used to perform the logistic regression analyses. For each of the analyses performed in this study, the input data set, the *p*-value associated with χ^2 test, parameter estimate, standard error, Wald statistics and odds ratio, are provided. The curves fitted to the data set using logistic regression analysis are presented both in terms of probabilities of exceeding a given level of the dependent variable (***GRAPH A***) and in terms of the probabilities of reaching a given level of the dependent variable (***GRAPH B***).

The **number of bridges** row shows the number of bridges in different damage states or repair cost ratio intervals. When the dependent variable used in the analysis is the *damage state*, then the numbers in this row corresponds to number of bridges with *no damage*, *minor damage*, *moderate damage*, *major damage* and *collapse* in this order. For the repair cost ratio; the numbers in the **number of bridges** row correspond to repair cost ratios 0; 10% - 20%; 20% - 30%; 30% - 40%; 40% - 50% and larger than 50%.

In order to determine whether the independent variables in the model are "significantly" related to the outcome variable, observed values of the response variable to predicted values obtained from models with and without the variable in question are compared. In general, a model is accepted with a p-value of the Wald chi-square statistic at a significance level of less than or equal to 0.05. A chi-square test with (t - 2) degrees of freedom is used to test whether the parameter estimate on the model is zero. The variable t represents the number of response variable levels. When data exist only for two damage states, the chi-square statistics is not applicable and is denoted by N/A. The odds ratio reported in these figures is the exponentiated value of the corresponding parameter estimate. Note that for parameter estimates larger than about 6.5, the odds ratio becomes larger than 1,000 and is reported as 999 indicating sparse data. The bridge class sub-categories used to group bridges are listed in table 2-1 and are given in table C-1 for convenience.

Definitions of different data sets used throughout the report are also presented here. The empirical fragility curves are obtained from the *correlation data set*.

TABLE C-1 Description of Bridge Sub-categories (based on Basöz and Kiremidjian, [1996])

Bridge Sub-category	Abutment Type	Column Bent Type	Continuity
Single Span Bridges			
C1S1	monolithic	N/A	N/A
C1S2	non-monolithic	N/A	N/A
C1S3	partial integrity	N/A	N/A
Multiple Span Bridges			
C1M1	monolithic	multiple	continuous
C1M2	monolithic	multiple	discontinuous
C1M3	monolithic	single	continuous
C1M4	monolithic	single	discontinuous
C1M5	monolithic	pier wall	continuous
C1M6	monolithic	pier wall	discontinuous
C1M7	non-monolithic	multiple	continuous
C1M8	non-monolithic	multiple	discontinuous
C1M9	non-monolithic	single	continuous
C1M10	non-monolithic	single	discontinuous
C1M11	non-monolithic	pier wall	continuous
C1M12	non-monolithic	pier wall	discontinuous
C1M13	partial integrity	multiple	continuous
C1M14	partial integrity	multiple	discontinuous
C1M15	partial integrity	single	continuous
C1M16	partial integrity	single	discontinuous
C1M17	partial integrity	pier wall	continuous
C1M18	partial integrity	pier wall	discontinuous

TABLE C-2 Description of Abutment Types

Inventory Code	Description	Abutment Type
A	Diaphragm	*monolithic*
E	Rigid Frame	*monolithic*
B	Seat	*non-monolithic*
C	Cantilever	*non-monolithic*
D	Strutted	*non-monolithic*
F	Bin	*partial*
G	Cellular Closure	*partial*

Event: *Loma Prieta earthquake.*

Dependent variable: *Damage state.*

Independent variable: *Peak ground acceleration based on the scenario event.*

Grouped by	Data set
Bridge sub-categories	*correlation data set*
Number of spans	*correlation data set*
NIBS bridge classes	*highway bridge data set*
Abutment type	*correlation data set*
Column bent type	*correlation data set*
Design year	*highway bridge data set*

DATA SET DEFINITIONS

Highway bridge data set: State highway bridges.

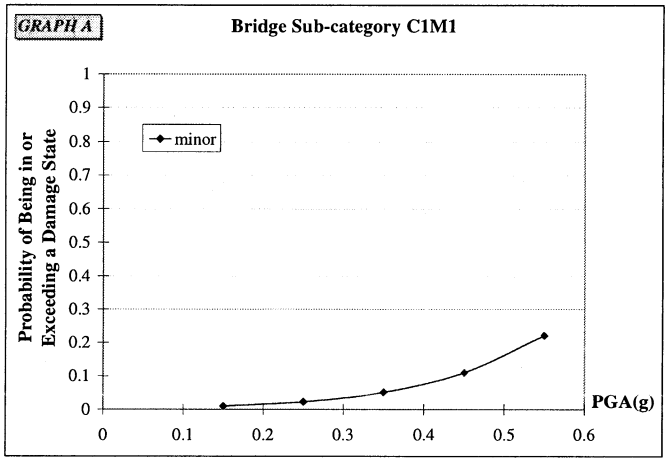
Concrete highway bridge data set: State highway bridges with concrete superstructure type and substructure material (when applicable).

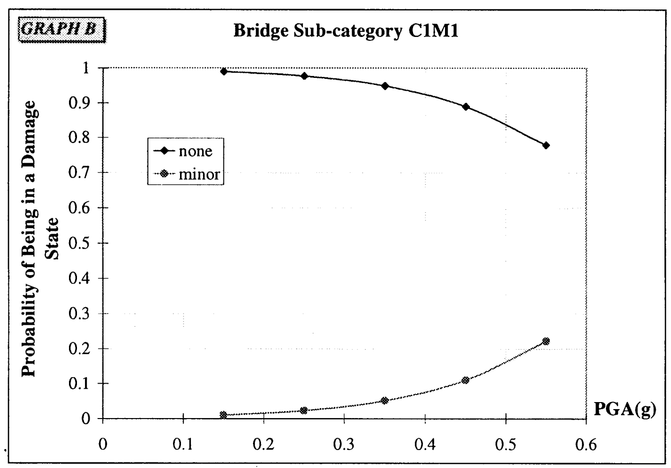
Homogeneous data set: State highway bridges with concrete superstructure type and substructure material (when applicable) with unique abutment and column bent type (excludes bridges with incomplete information).

Correlation data set: State highway bridges with concrete superstructure type and substructure material (when applicable) with unique abutment and column bent type exposed to a PGA level larger than or equal to a threshold value (excludes bridges with incomplete information).

Number of bridges 153, 3
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
8.3065	4.4761	0.0635	999

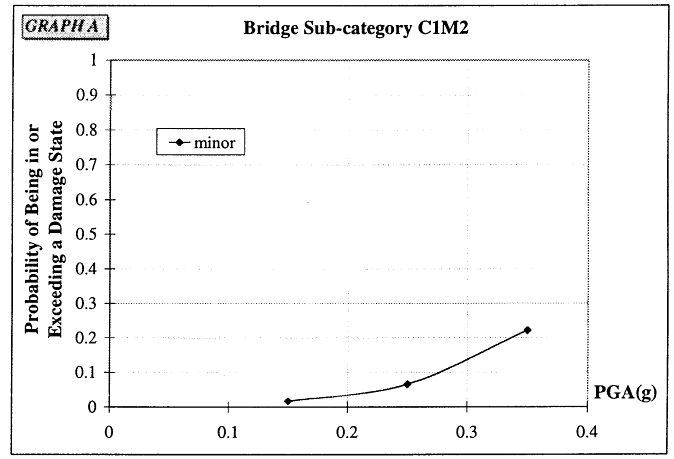


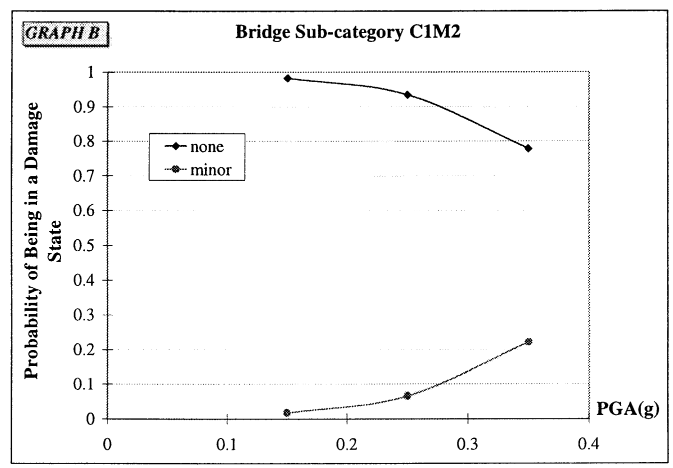


Loma Prieta Earthquake *Bridge sub-categories*

Number of bridges 43, 2
Chi-square N/A

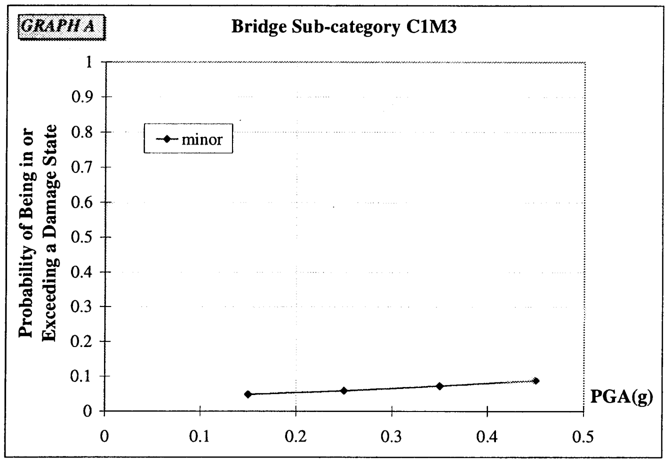
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
13.9898	11.3567	0.218	999

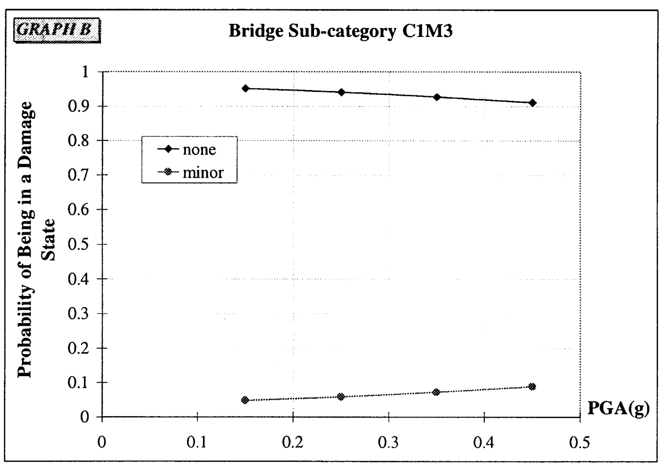




Number of bridges 36, 2
Chi-square N/A

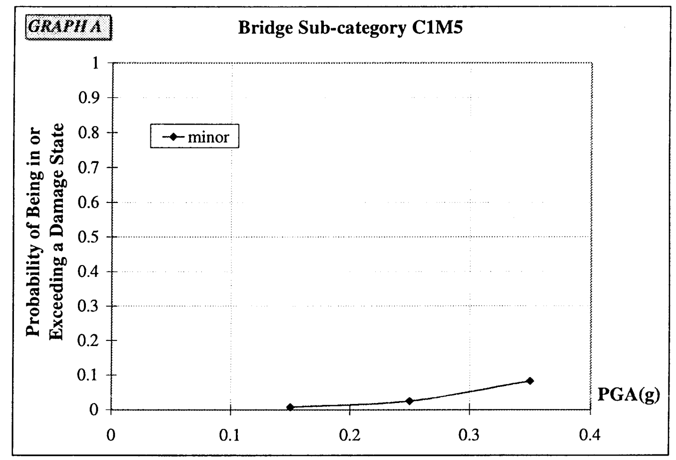
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.1754	9.5764	0.8203	8.806

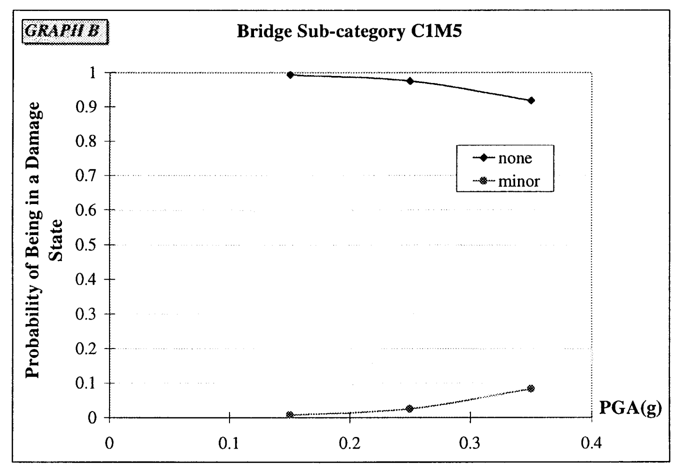




Number of bridges 51, 1
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
12.6058	17.8223	0.4794	999

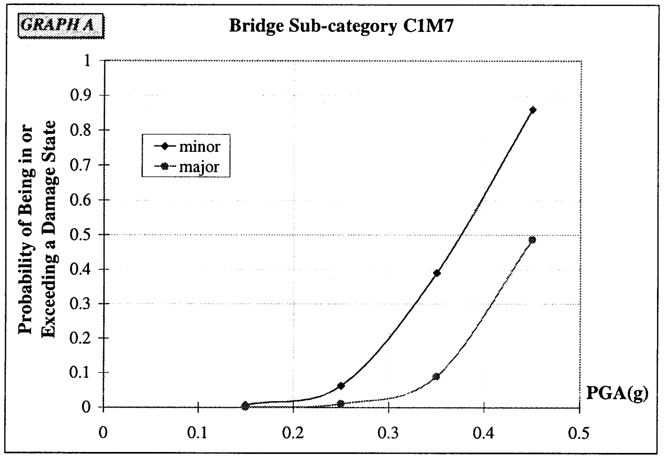


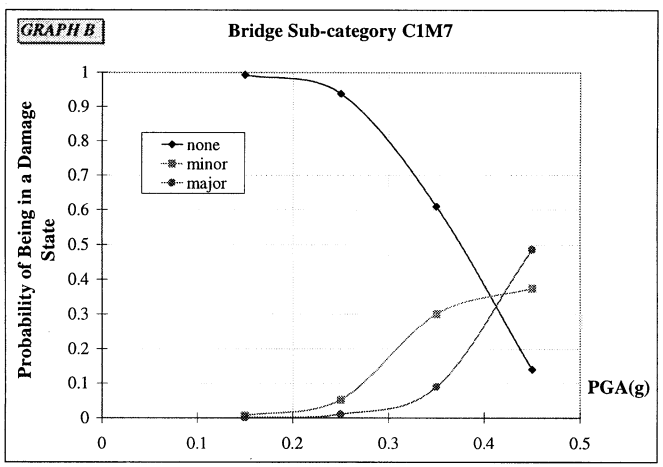


Loma Prieta Earthquake ***Bridge sub-categories***

Number of bridges 106, 5, 2
Chi-square with 1 DF 3.2006; p = 0.0736

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
22.6094	5.9625	0.0001	999

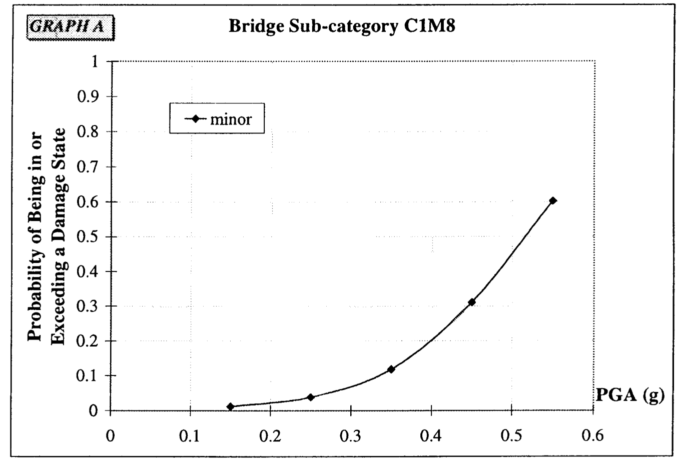


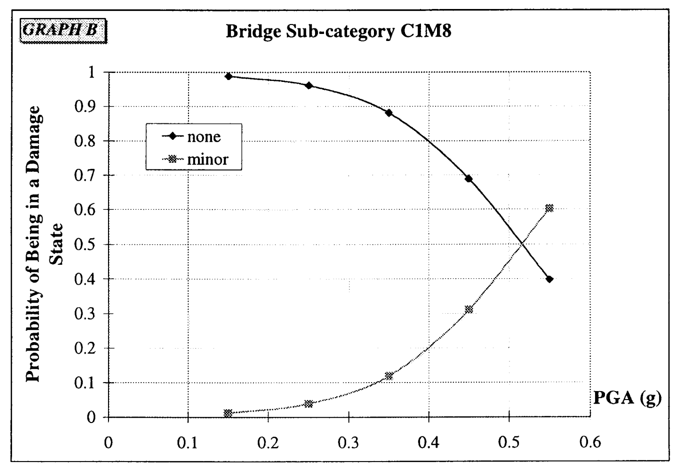


Loma Prieta Earthquake ***Bridge sub-categories***

Number of bridges 24, 2
Chi-square N/A

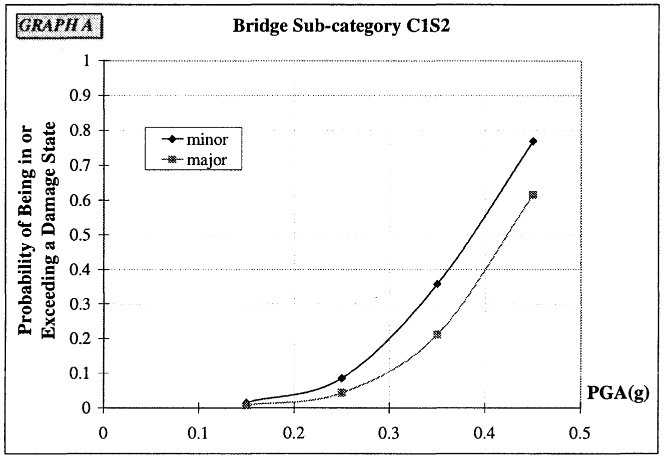
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
12.0991	7.1943	0.0926	999

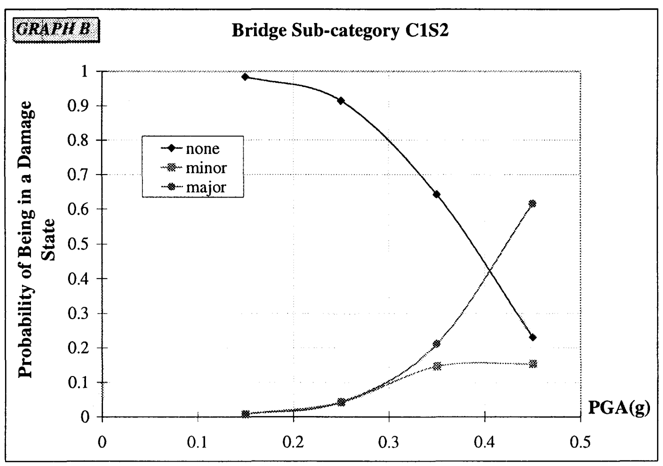




Number of bridges 42, 2, 4
Chi-square with 1DF 8.9052; p = 0.0028

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
17.8695	5.3005	0.0007	999





Loma Prieta Earthquake *Bridge sub-categories*

Validity of the model fit is questionable for C1M4.

Number of bridges 6, 1
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
56.5599	374	0.8798	999

Validity of the model fit is questionable for C1M9.

Number of bridges 23, 2
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
153.2	366.2	0.6757	999

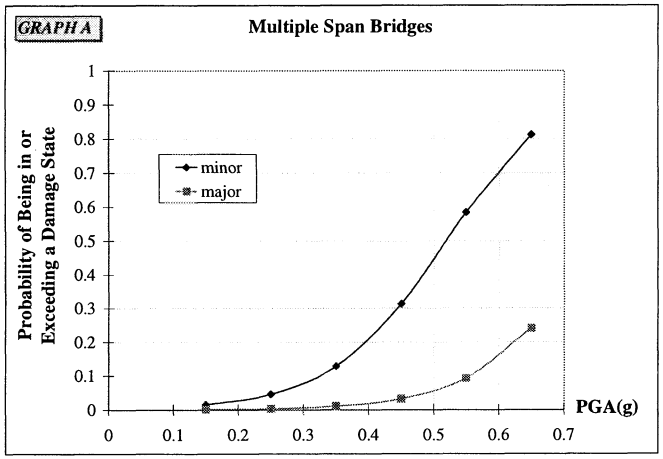
Validity of the model fit is questionable for C1M18.

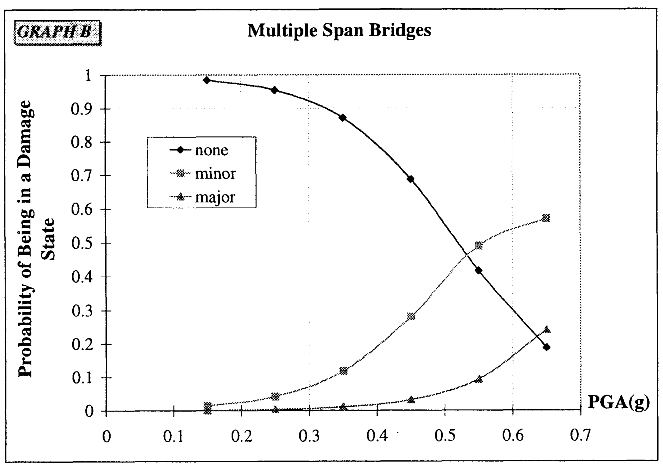
Number of bridges 8, 2
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
117.5	1450.8	0.9355	999

Number of bridges 544, 20, 4
Chi-square with 1 DF 1.3654; p = 0.2426

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
11.2618	1.9823	0.0001	999

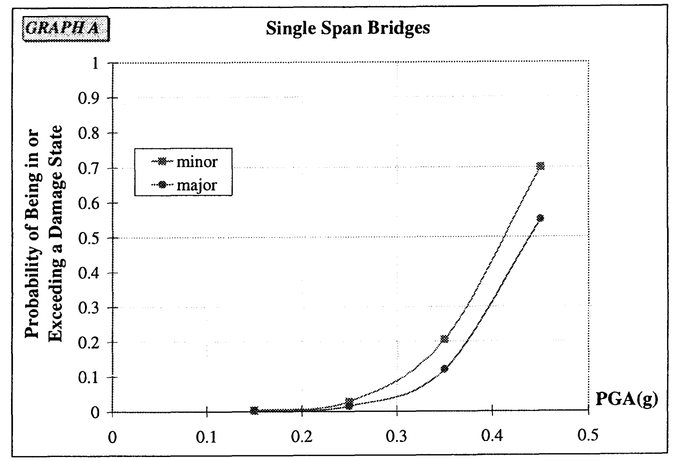


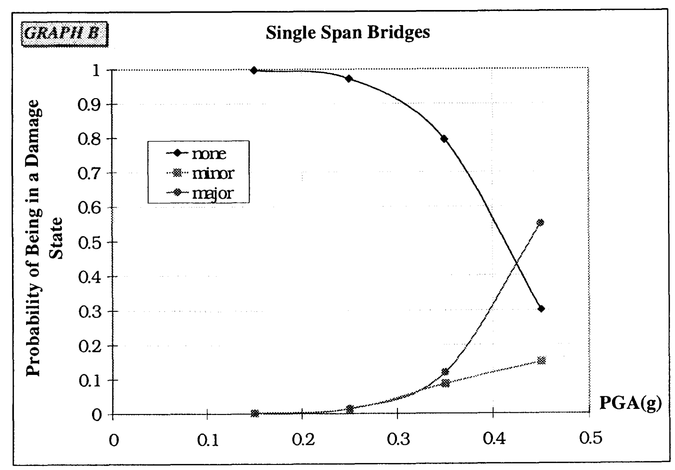


Loma Prieta Earthquake Number of Spans

Number of bridges 137. 2, 4
Chi-square with 1 DF 9.5881; p = 0.002

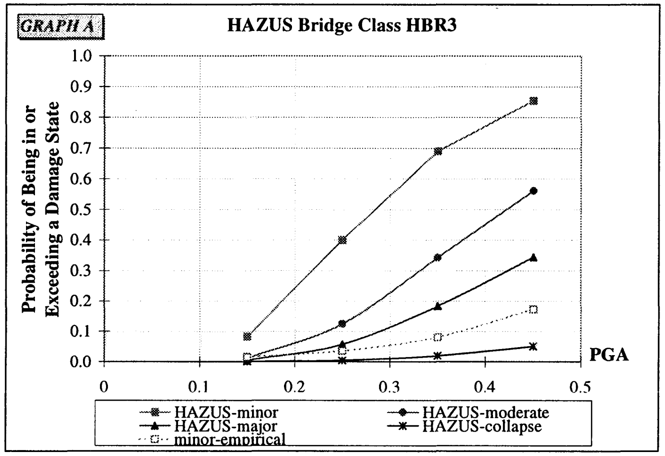
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
21.9938	5.3284	0.0001	999

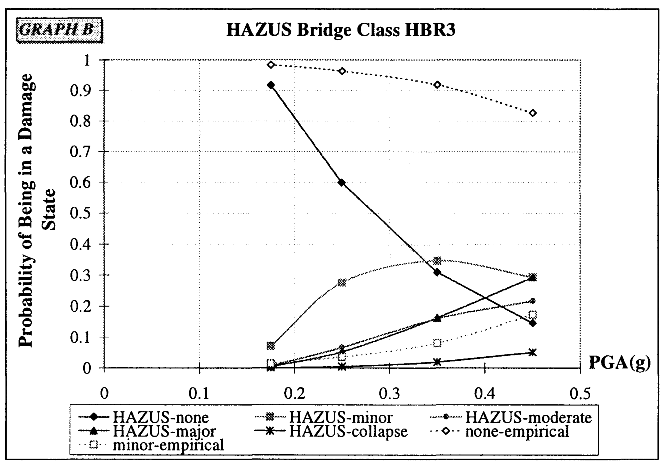




Number of bridges 297, 8
Chi-square N/A

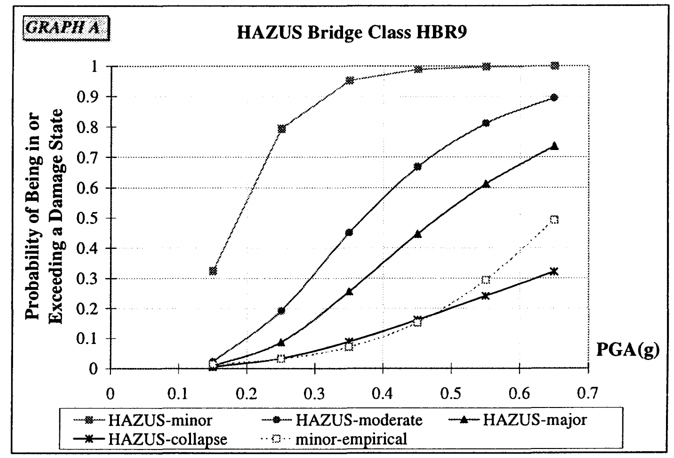
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
8.592	4.2591	0.0437	999

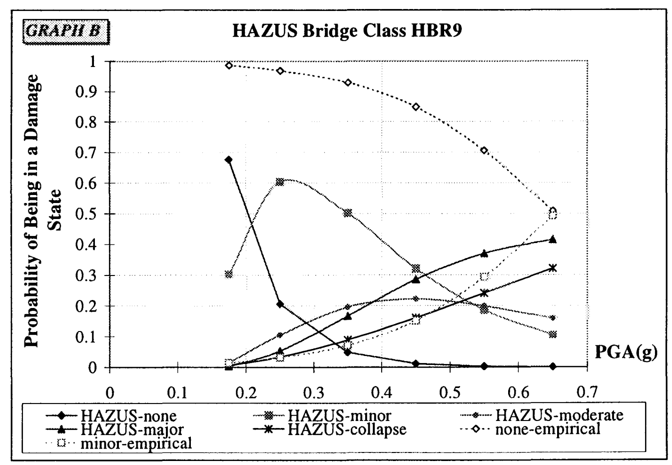




Number of bridges 161, 5
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
8.4552	8.0862	0.3932	999

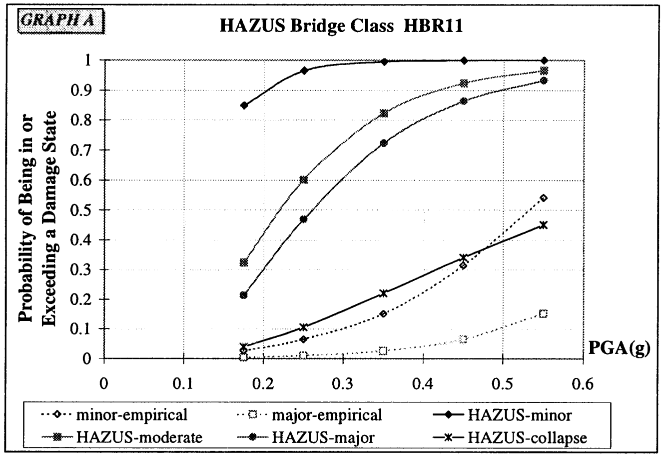


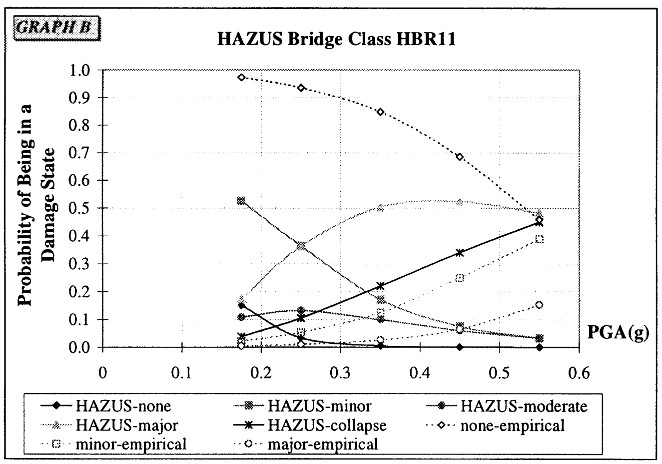


Loma Prieta Earthquake *HAZUS Bridge Classes*

Number of bridges 520, 24, 5
Chi-square with 1DF 0.25, p = 0.6171

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
9.4706	1.884	0.0001	999

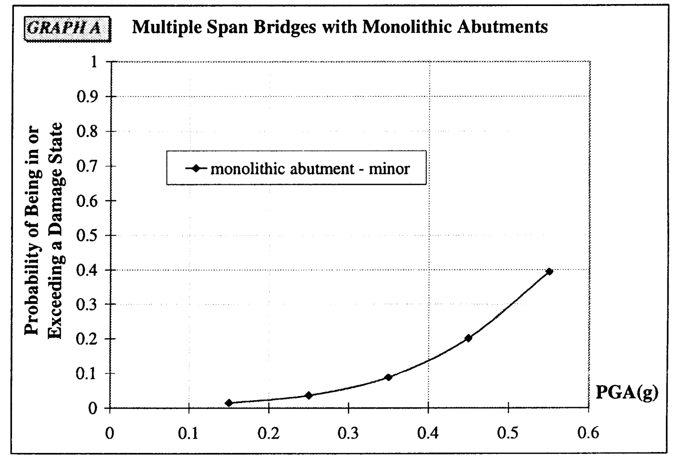


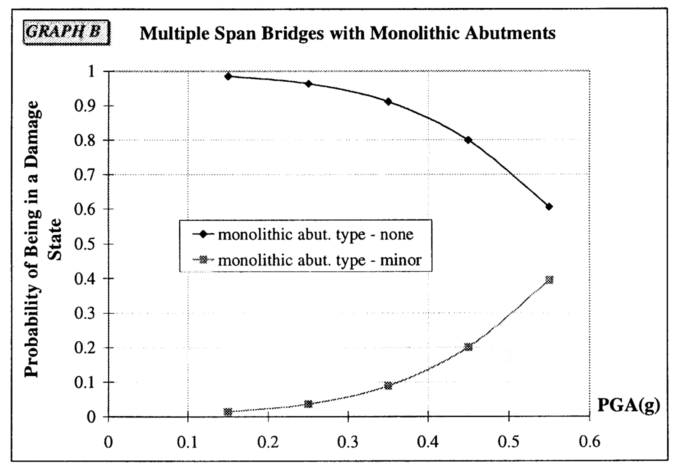


Loma Prieta Earthquake *Abutment type*

Number of bridges 291, 9
Chi-square N/A

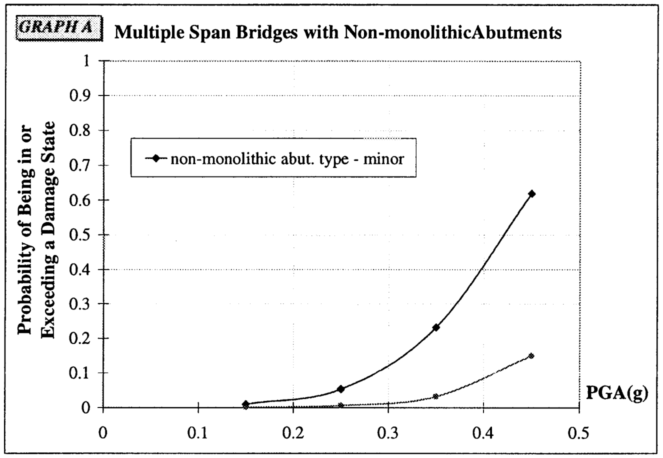
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
9.4814	2.9388	0.0013	999

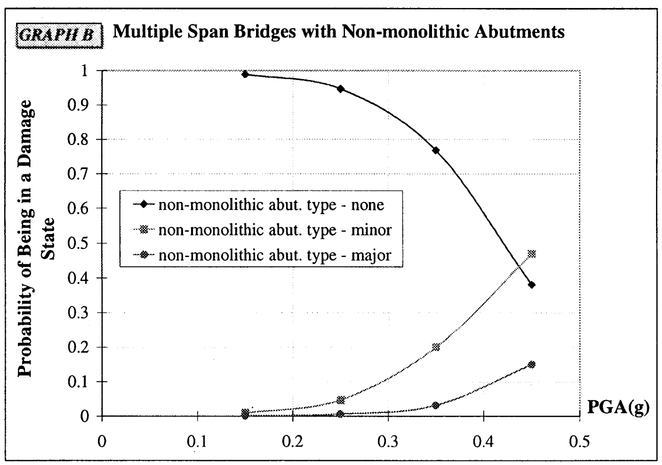




Number of bridges 178, 9, 2
Chi-square with 1 DF 0.6222; p = 0.4302

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
16.8464	3.7816	0.0001	999

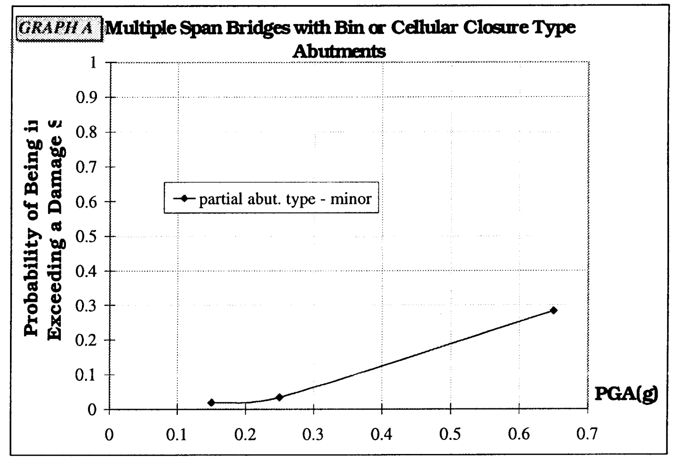


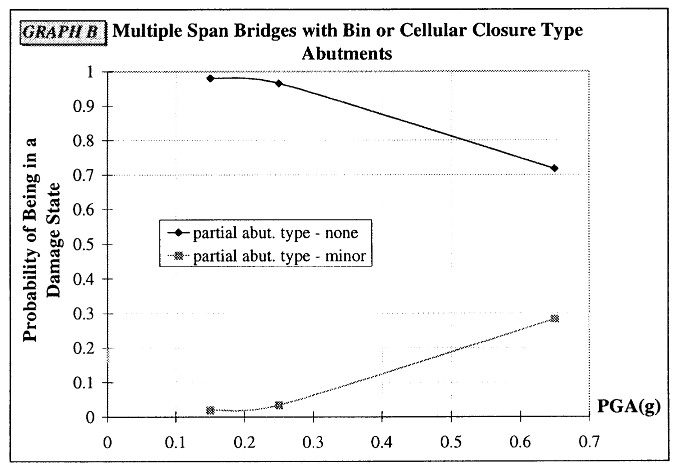


Loma Prieta Earthquake *Abutment type*

Number of bridges 75, 2
Chi-square N/A

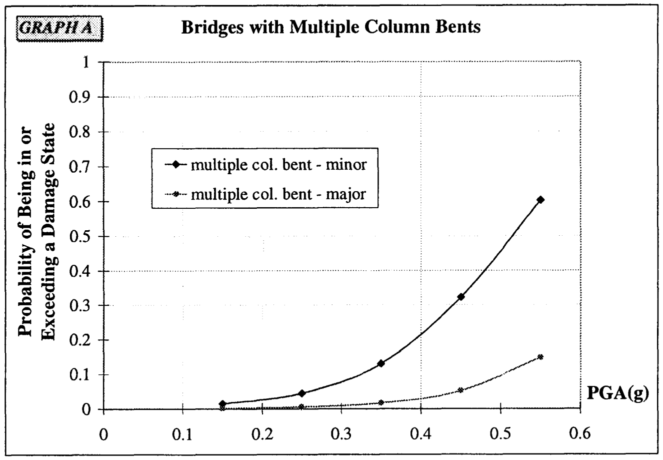
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.0089	4.8177	0.2123	407.025

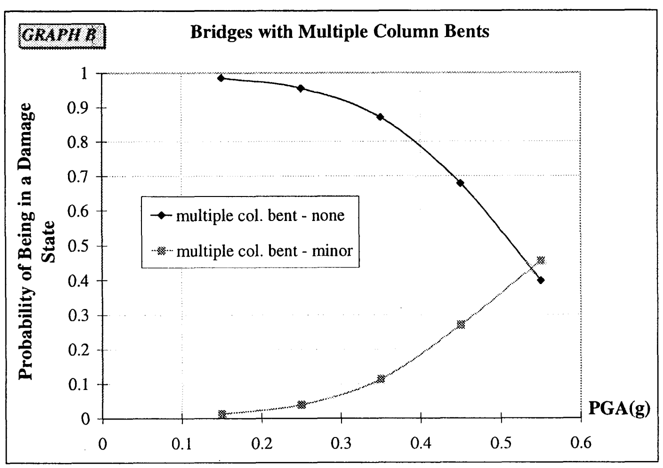




Number of bridges 336, 12, 2
Chi-square with 1 DF 1.2530; p = 0.2630

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
11.5953	2.5448	0.0001	999

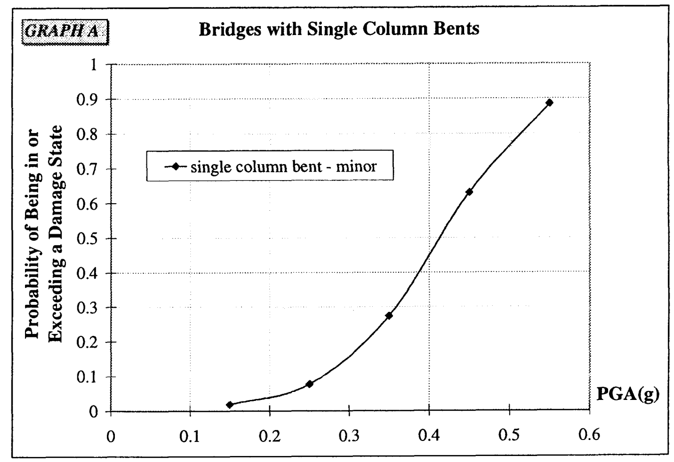


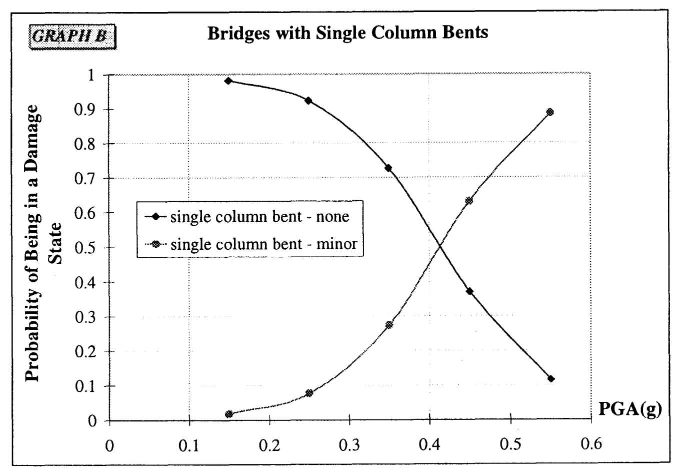


Loma Prieta Earthquake — *Column bent type*

Number of bridges 75, 5
Chi-square N/A

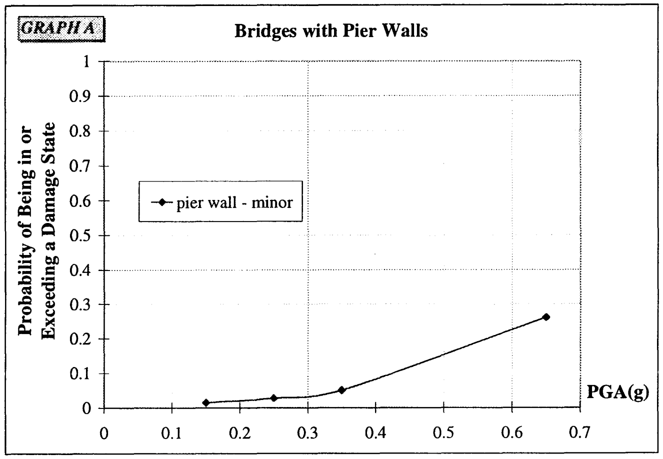
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
15.1012	5.3181	0.0045	999

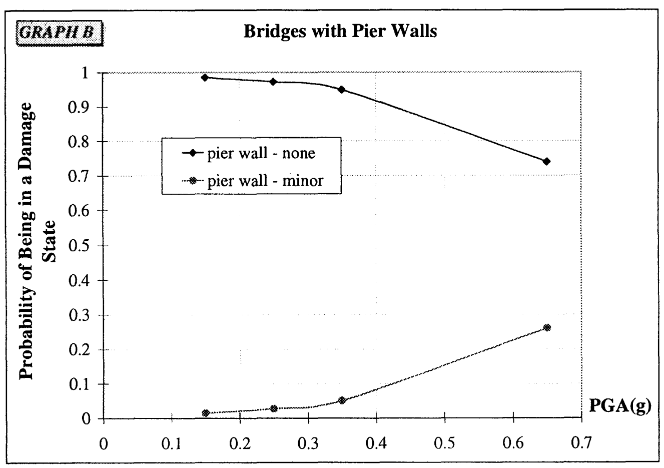




Number of bridges 133, 3
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.3098	4.7254	0.1818	549.913





Bridges built before 1940.

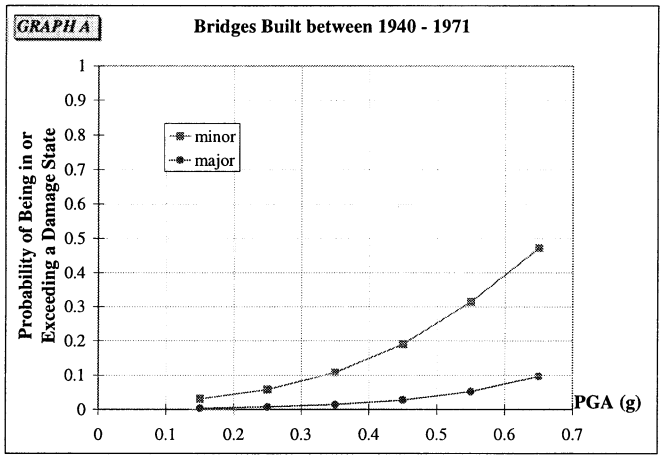
Validity of the model fit is questionable.

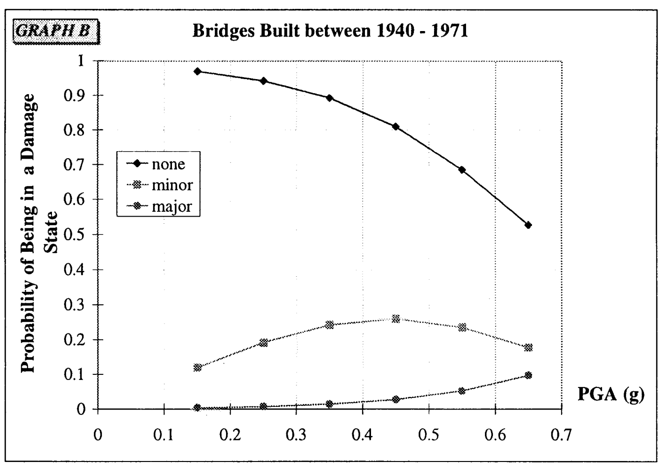
Number of bridges 87, 2, 2
Chi-square with 1 DF 0.0; p = 0.9954

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
-110.2	1745.2	0.9497	0.00025

Number of bridges 639, 27, 4
Chi-square with 1 DF 0.4658; p = 0.4949

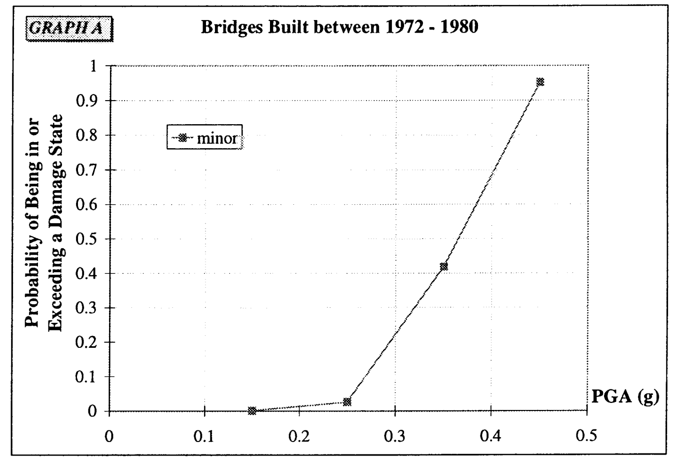
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.6853	1.4996	0.0001	800.535

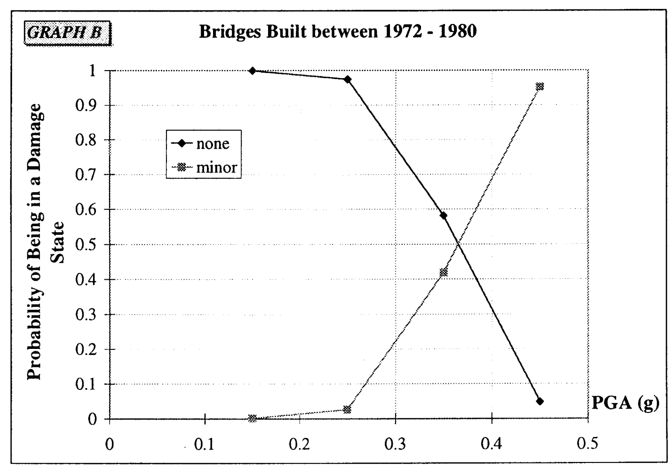




Number of bridges 136, 5
Chi-square N/A

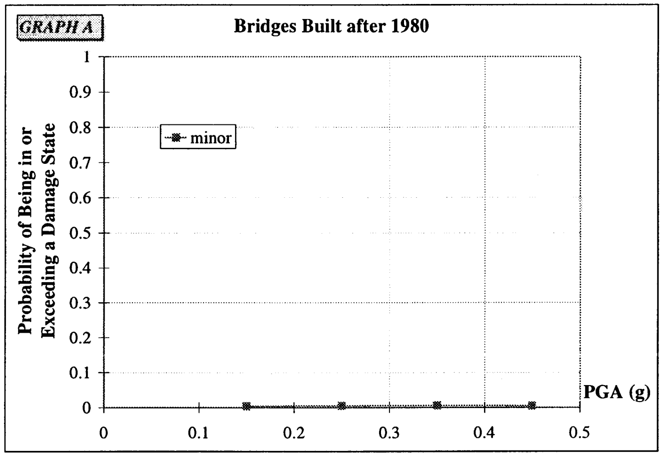
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
33.085	10.622	0.0018	999

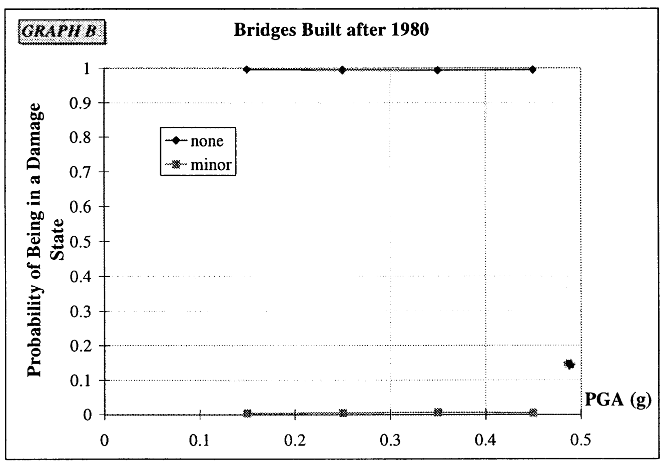




Number of bridges 207, 1
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.5681	11.6295	0.8252	13.042





APPENDIX D

EMPIRICAL FRAGILITY CURVES

Event: *Loma Prieta earthquake.*

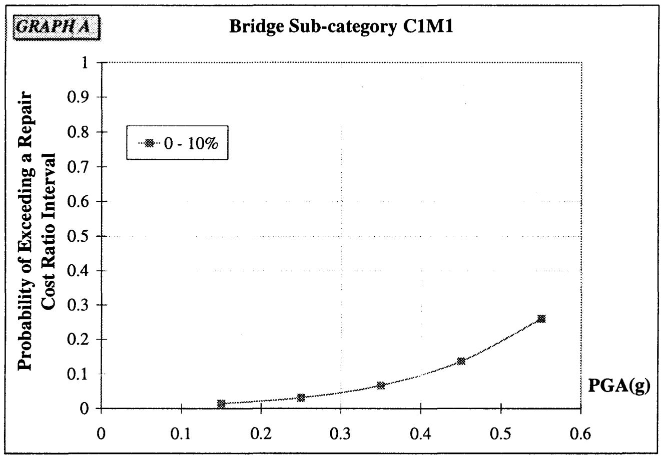
Dependent variable: *Repair cost ratio.*

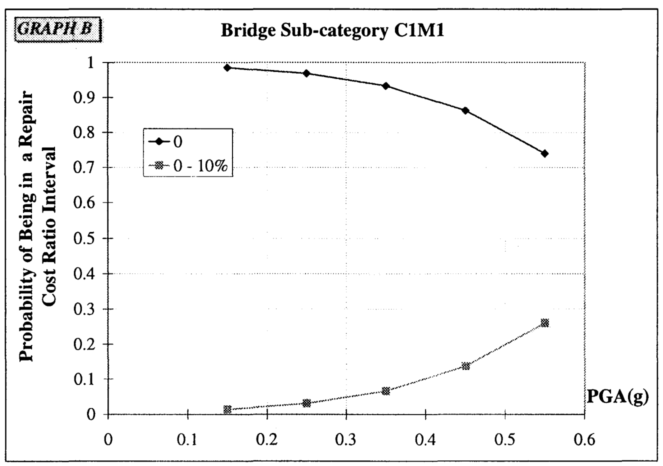
Independent variable: *Peak ground acceleration based on the scenario event and Campbell attenuation relationship.*

Grouped by	Data set (conditional on damage)
Bridge sub-categories	*correlation data set*
Number of spans	*correlation data set*
Design year	*highway bridge data set*

Number of bridges 152, 4
Chi-square N/A

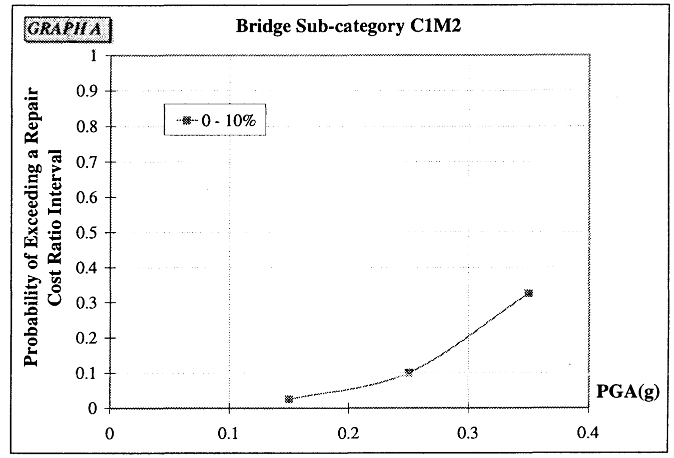
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
7.9759	4.059	0.0494	999

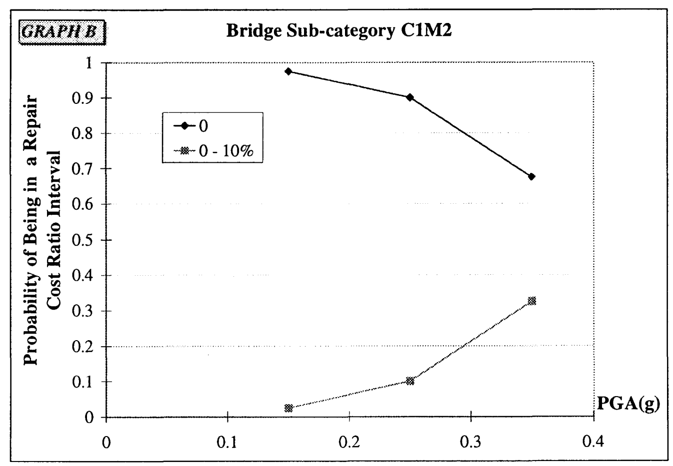




Number of bridges 42, 3
Chi-square N/A

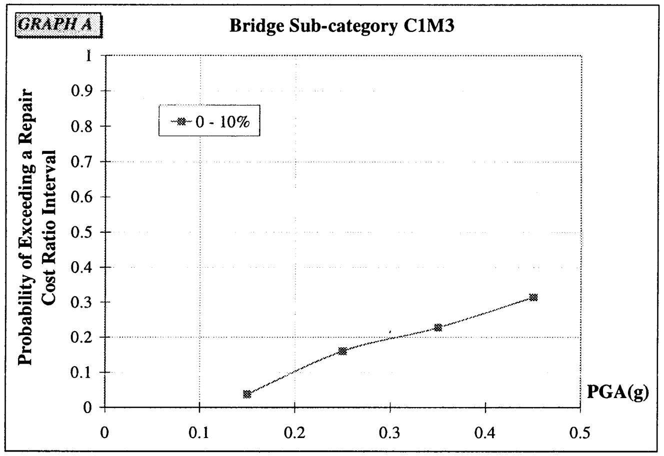
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
14.663	9.723	0.1315	999

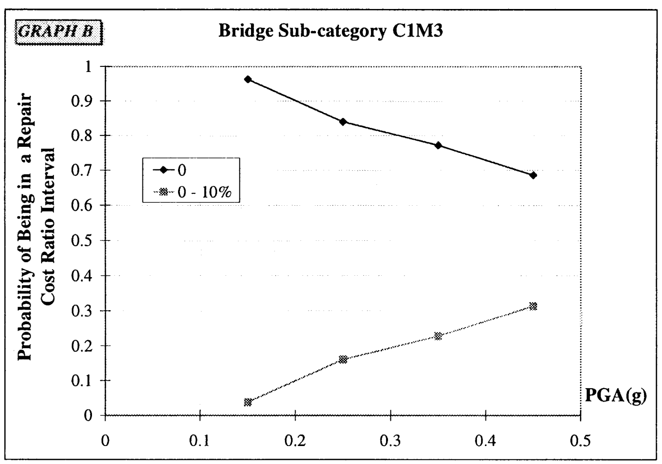




Number of bridges 33, 5
Chi-square N/A

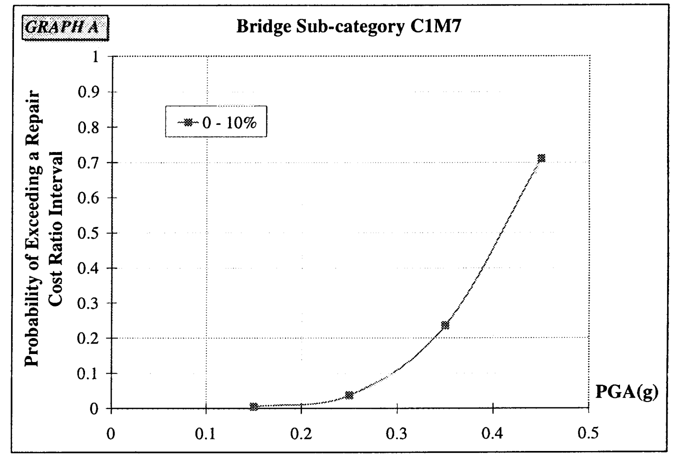
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.365	6.082	0.473	78.665

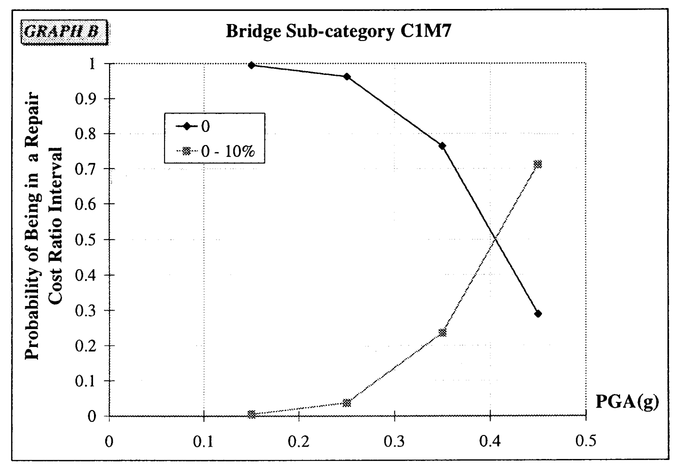




Number of bridges 108, 7
Chi-square N/A

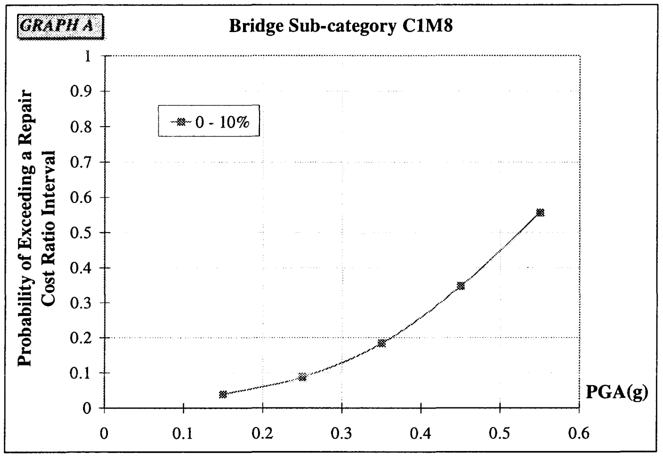
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
20.765	6.696	0.0019	999

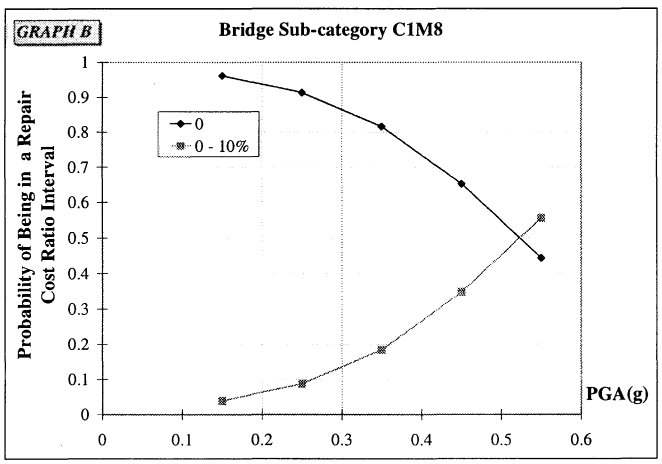




Number of bridges 23, 3
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
8.577	5.502	0.119	999





Validity of the model fit is questionable for C1M4.

Number of bridges 5, 2
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
-78.1626	610.2	0.8981	0.0003

Validity of the model fit is questionable for C1M9.

Number of bridges 24, 1
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
113.3	1176.6	0.9233	999

Validity of the model fit is questionable for C1M10.

Number of bridges 7, 1
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
-81.763	1332.7	0.9511	0.0006

Validity of the model fit is questionable for C1M12.

Number of bridges 7, 1
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
-95.5	1873.9	0.9594	0.0007

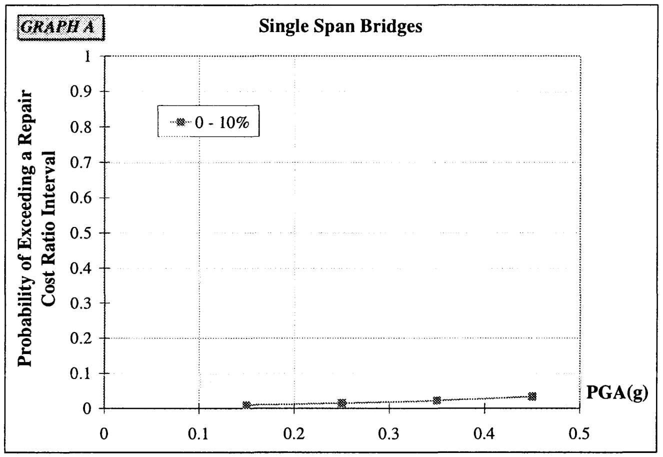
Validity of the model fit is questionable for C1M14.

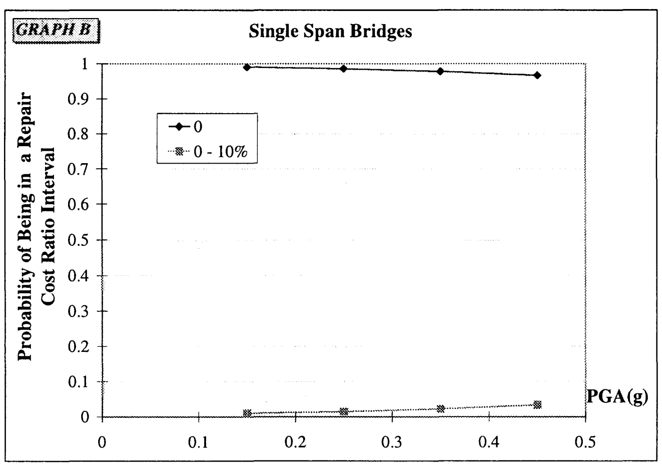
Number of bridges 4, 2
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
-99.5639	1778.6	0.9554	0.0007

Number of bridges 202, 4
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.198	4.073	0.3027	66.568

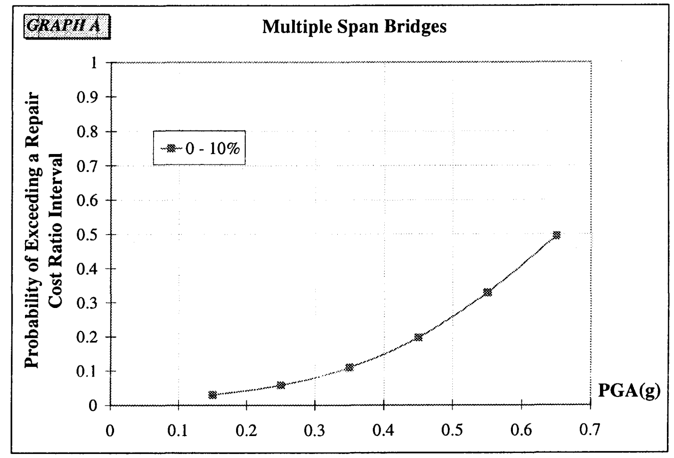


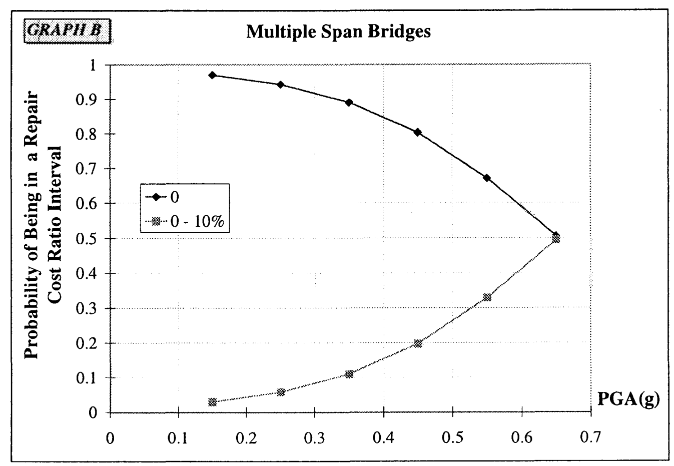


Loma Prieta Earthquake — *Number of Spans*

Number of bridges 539, 27
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.9101	1.8833	0.0002	999

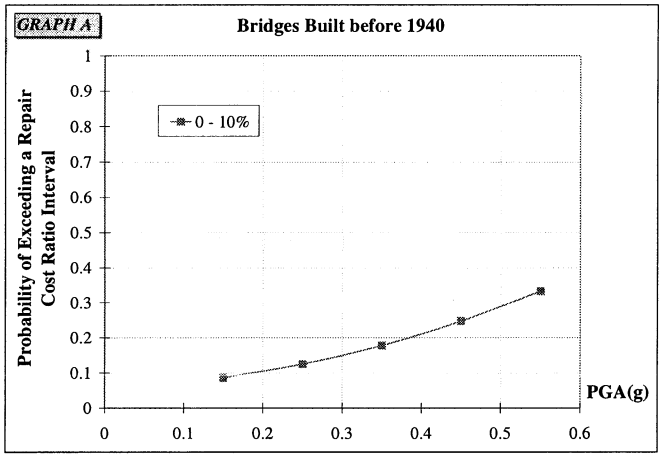


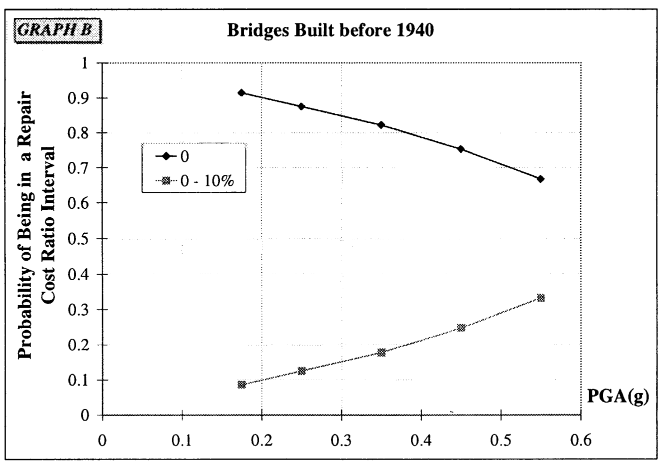


Loma Prieta Earthquake ***Design Year***

Number of bridges 83, 10
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.1662	3.2211	0.1959	64.47

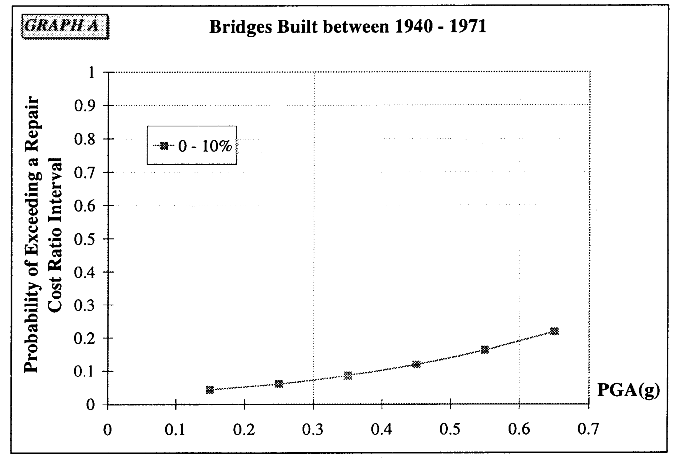


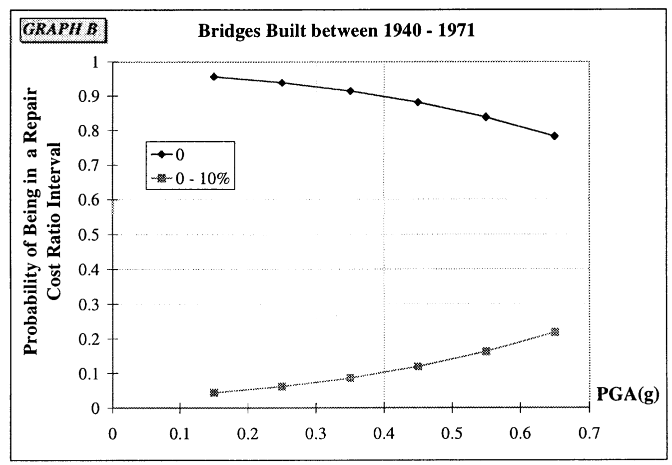


Loma Prieta Earthquake ***Design Year***

Number of bridges 635, 35
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.6167	1.6738	0.0307	37.216

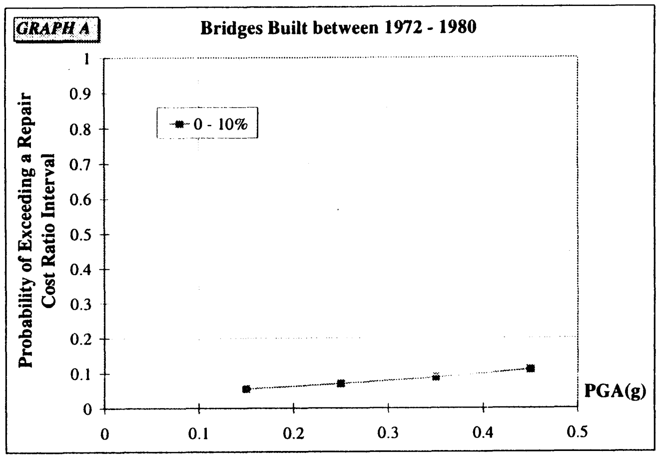


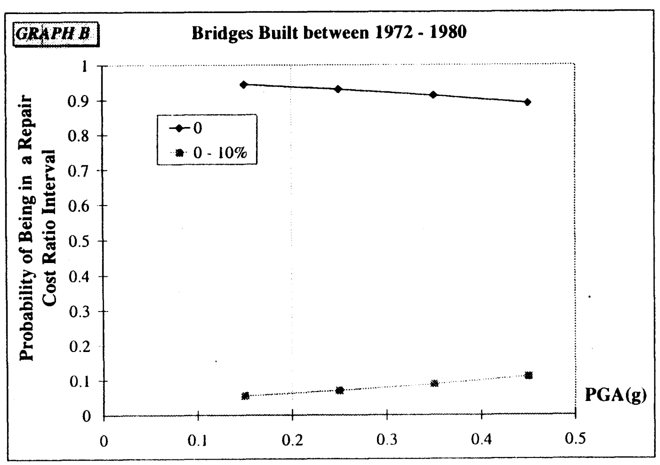


Loma Prieta Earthquake — *Design Year*

Number of bridges 132, 9
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.4872	5.3126	0.6397	12.028

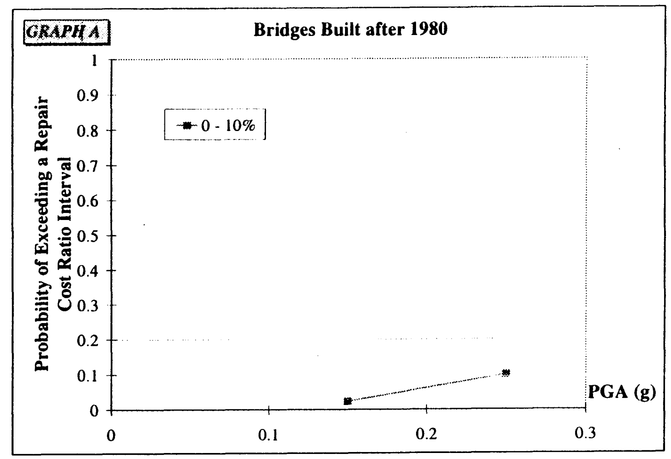


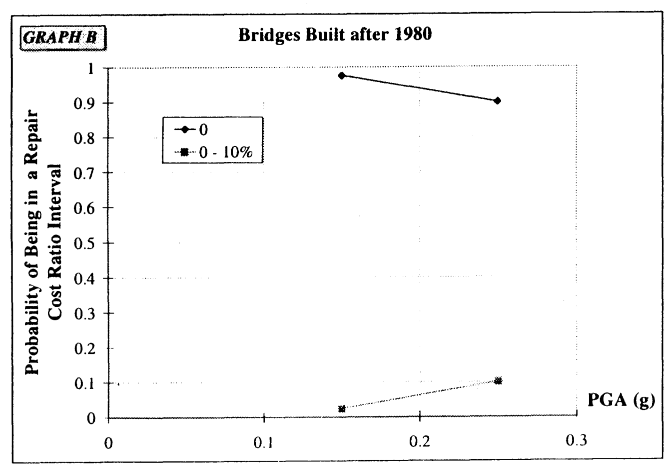


Loma Prieta Earthquake — *Design Year*

Number of bridges 68, 4
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
15.1635	11.81	0.1992	999





APPENDIX E

EMPIRICAL FRAGILITY CURVES

Event: *Northridge earthquake.*

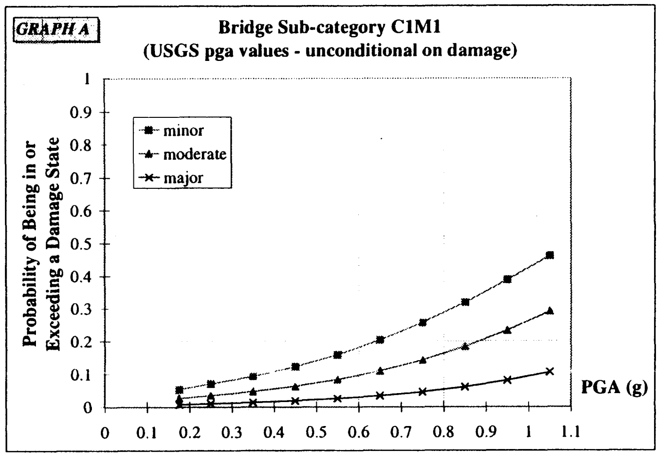
Dependent variable: *Damage state.*

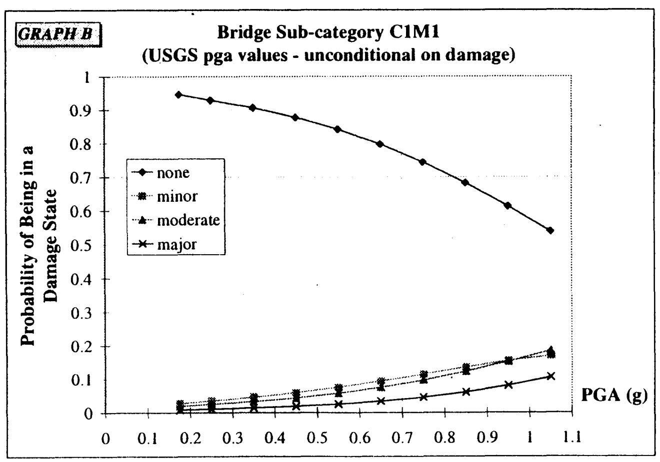
Independent variable: *Peak ground acceleration based on USGS [1994].*

Grouped by	Data set
Bridge sub-categories	*correlation data set*
Number of spans	*correlation data set*
NIBS bridge classes	*highway bridge data set*
Column bent type	*correlation data set*
Abutment type	*correlation data set*
Retrofit history	*correlation data set*
Design year	*highway bridge data set*

Number of bridges 228, 16, 13, 6
Chi-square with 2DF 0.2596; p = 0.8783

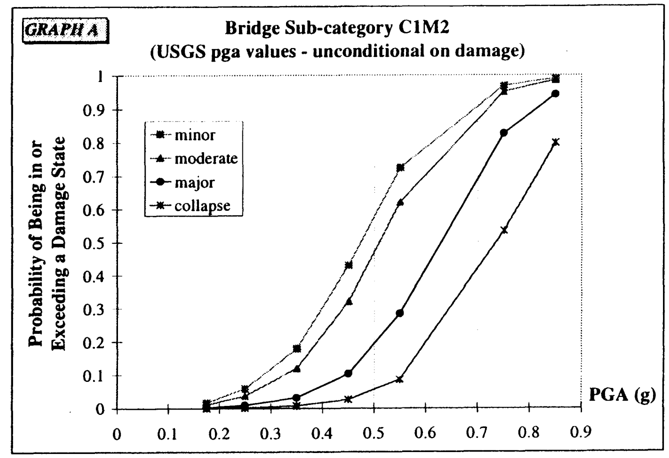
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.0251	0.6207	0.0001	20.597

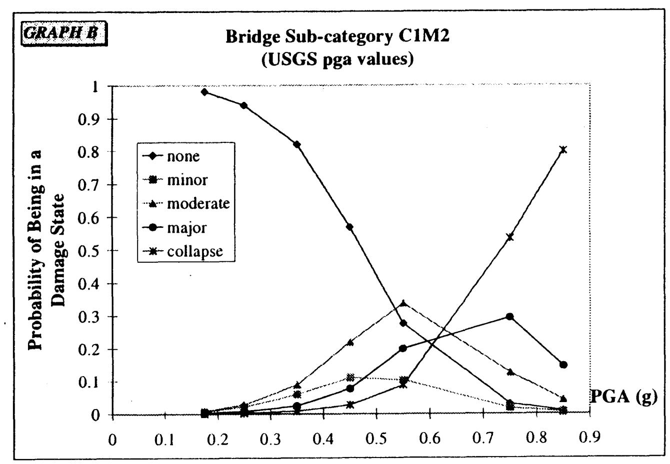




Number of bridges 24, 1, 2, 1, 2
Chi-square with 3DF 23.0508; p = 0.0001

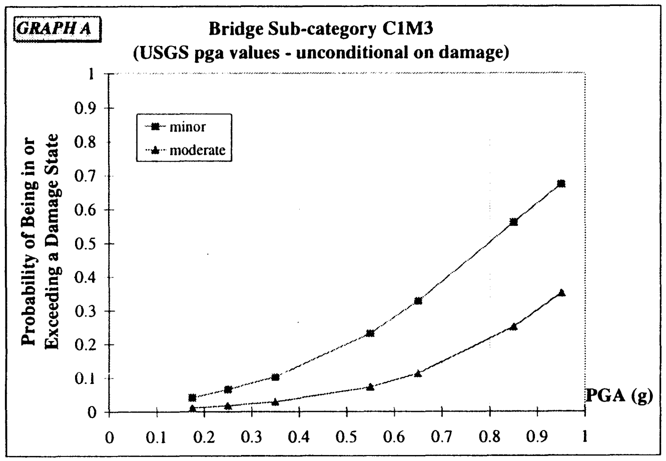
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
12.4267	3.7351	0.0009	999

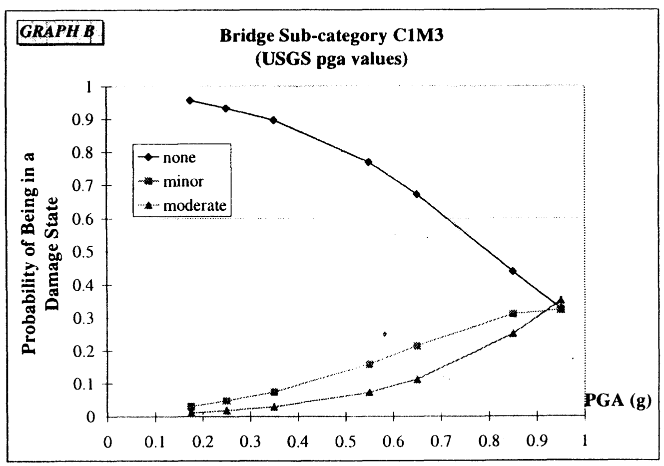




Number of bridges 35, 5, 3
Chi-square with 1DF 0.0521; p = 0.8194

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.8180	1.7399	0.0056	123.717

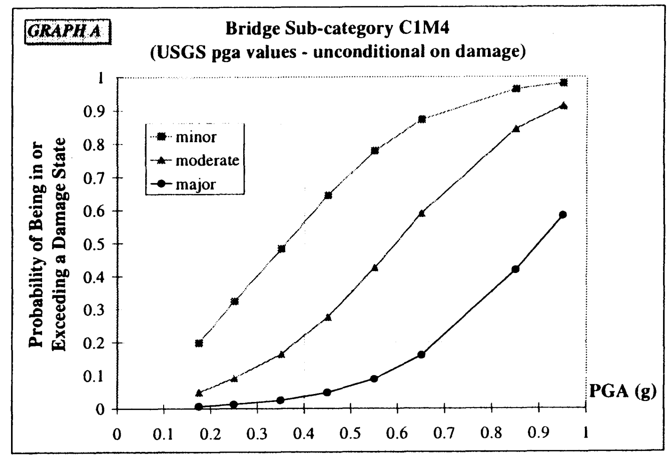


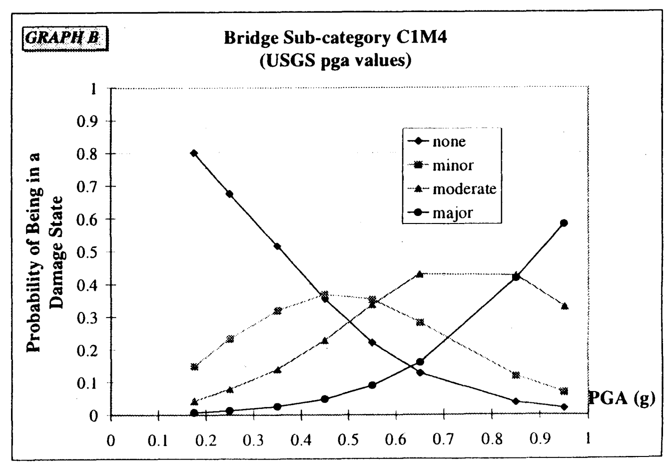


Northridge Earthquake | Bridge Sub-categories

Number of bridges 8, 4, 4, 2
Chi-square with 2DF 0.6649; p = 0.7172

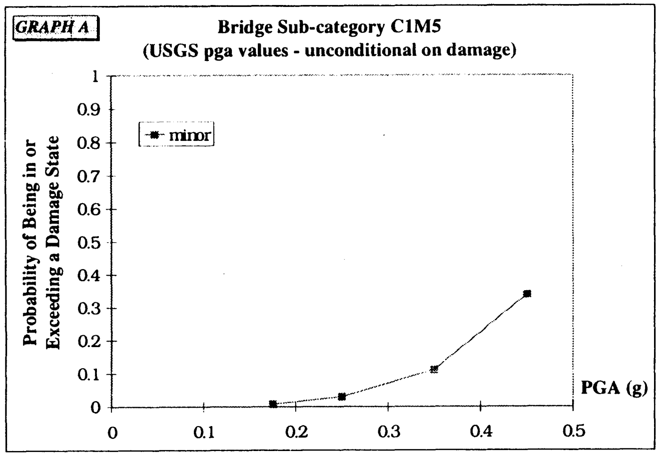
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.6356	2.4402	0.0065	761.475

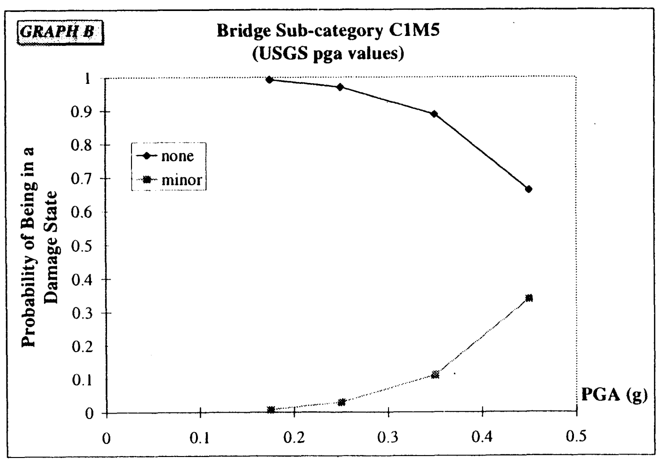




Number of bridges 39, 2
Chi-square N/A

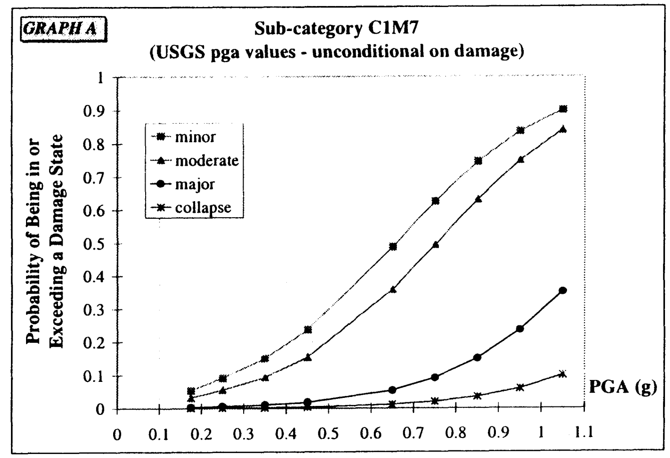
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
14.124	8.6177	0.1012	999

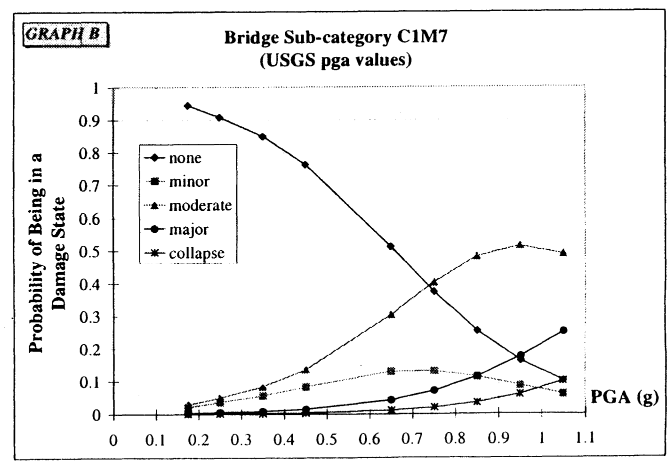




Number of bridges 170, 9, 17, 3, 1
Chi-square with 3DF 2.9939; p = 0.3926

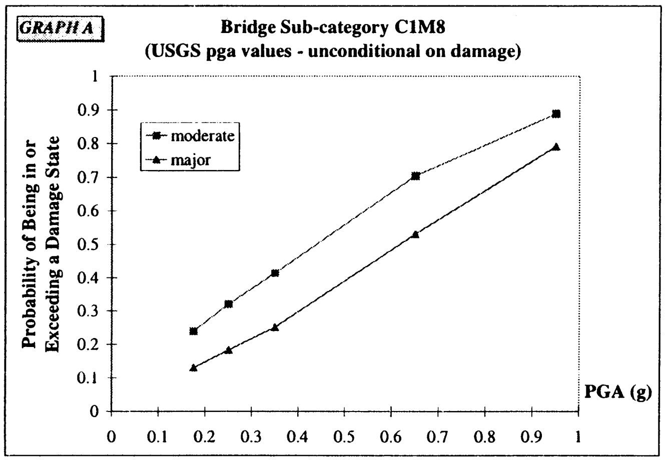
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
5.6053	0.9328	0.0001	271.863

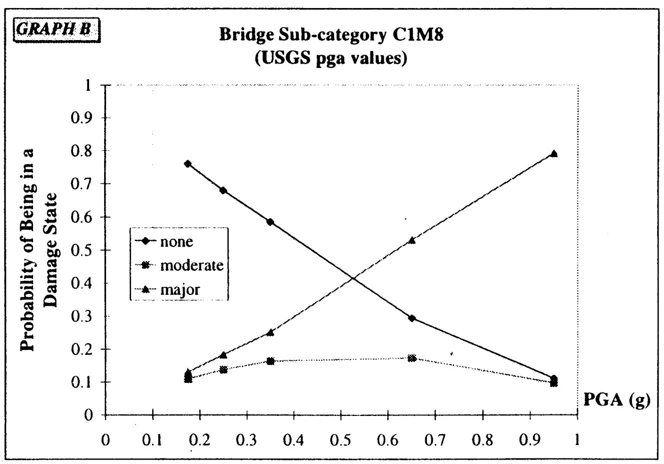




Number of bridges 18, 4, 7
Chi-square with 1DF 0.0249; p = 0.8746

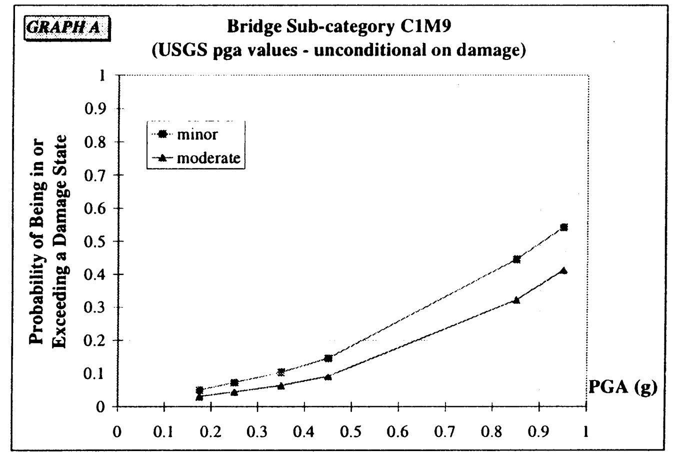
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.0475	2.2072	0.0667	57.253

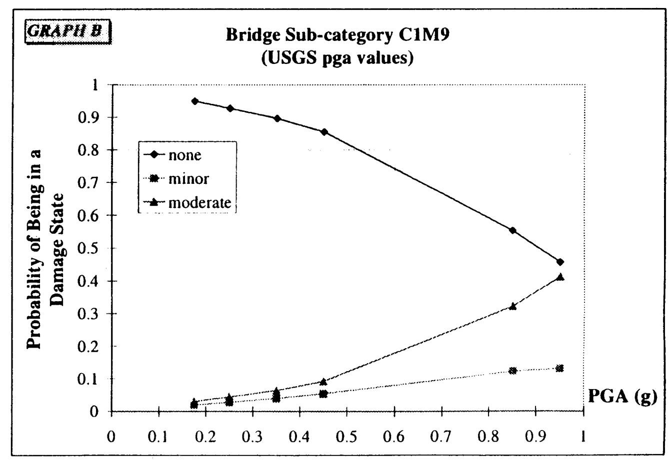




Number of bridges 40, 2, 4
Chi-square with 1DF 2.8379; p = 0.0921

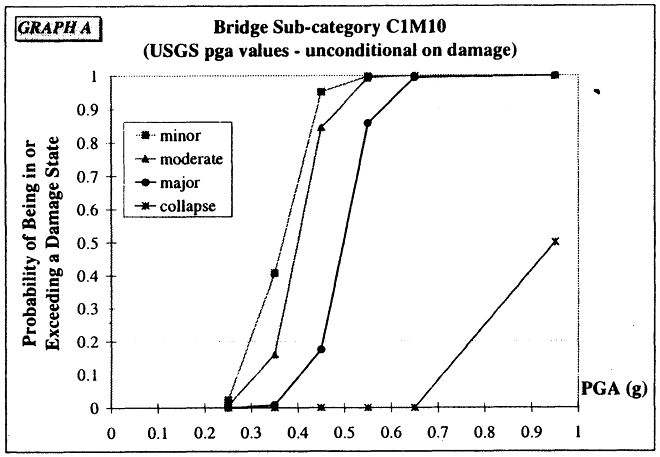
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.8822	1.5668	0.0132	48.529

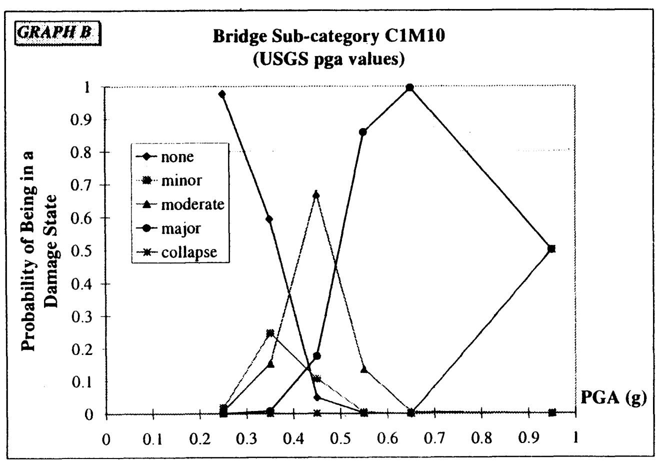




Number of bridges 9, 1, 2, 5, 1
Chi-square with 3DF 7.7383; p = 0.0517

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
33.5052	12.3706	0.0068	999



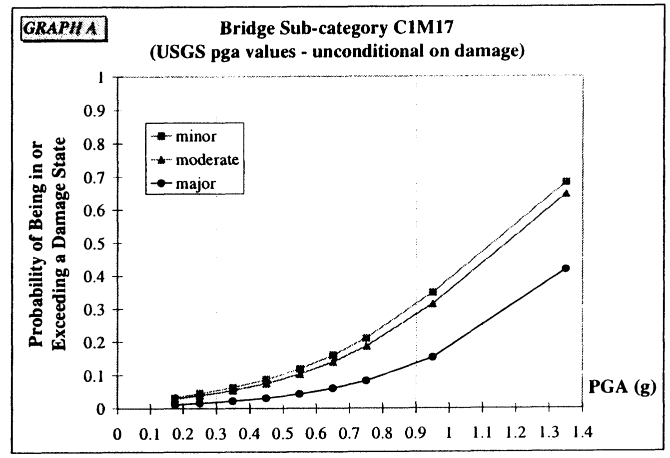


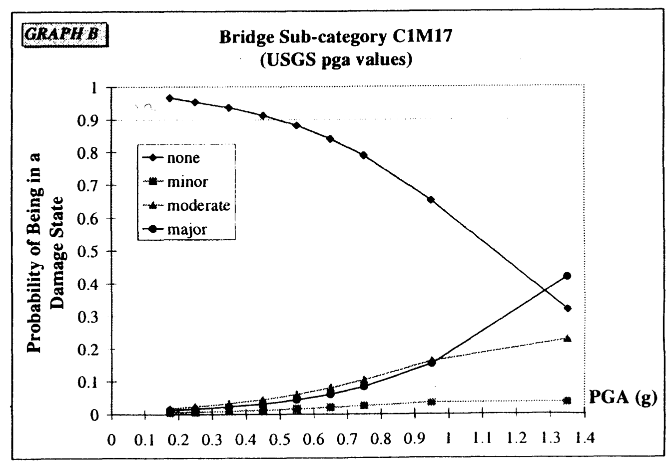
Northridge Earthquake

Bridge Sub-categories

Number of bridges 100, 1, 4, 3
Chi-square with 2DF 13.877; p = 0.001

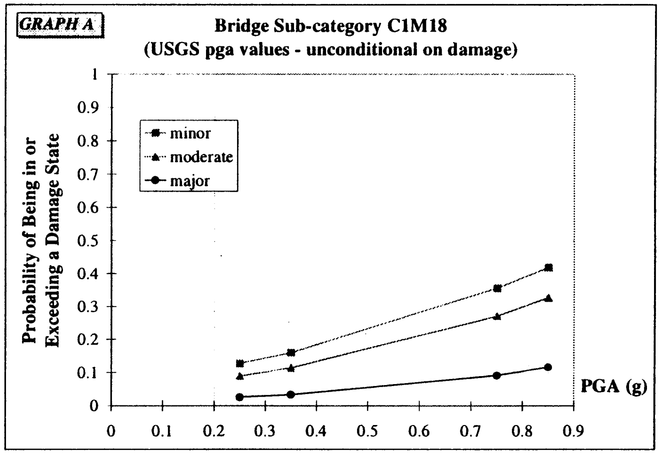
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.4587	1.2769	0.0068	31.777

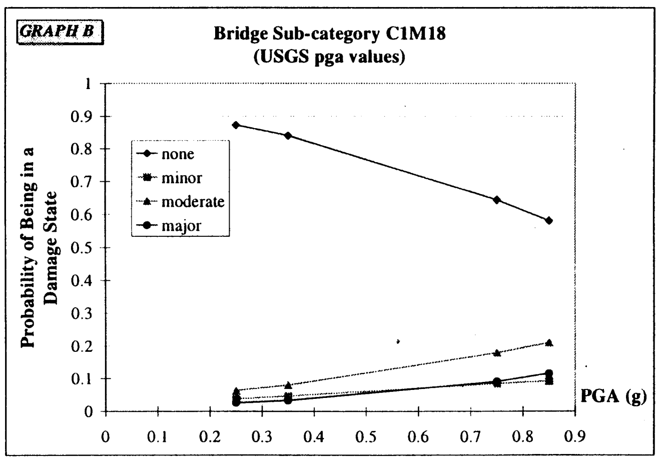




Number of bridges 11, 1, 2, 1
Chi-square with 2DF 4.4907; p = 0.1059

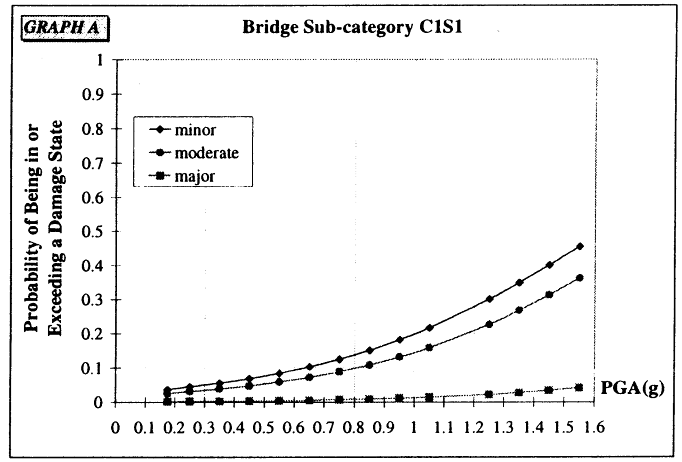
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.6627	2.4275	0.2727	14.334

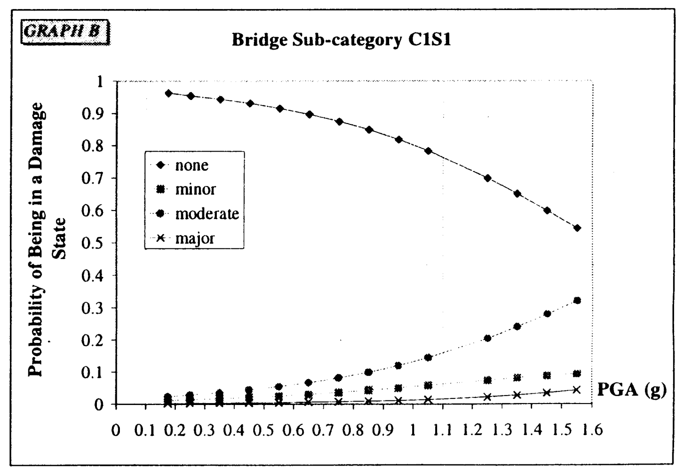




Number of bridges 197, 5, 11, 1
Chi-square with 1 DF 5.9637; p = 0.0507

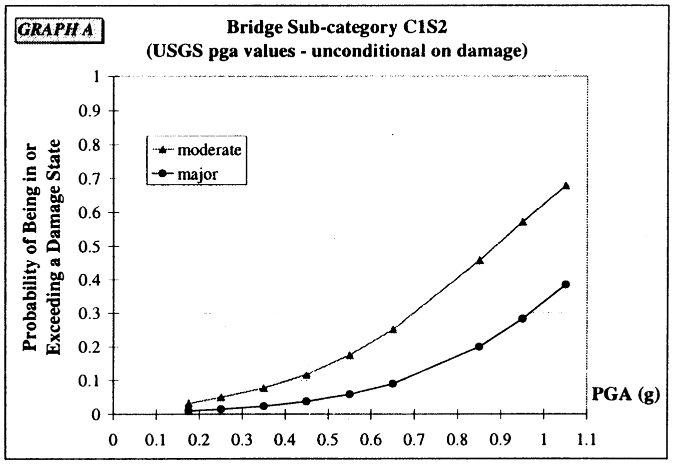
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.2107	0.6464	0.0006	9.1225

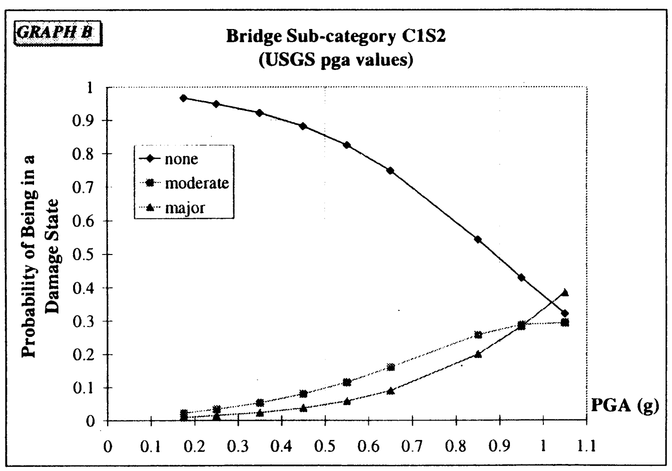




Number of bridges 83, 6, 4
Chi-square with 1DF 1.1852; p = 0.2763

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.6	1.1566	0.0001	99.483





Validity of the model fit is questionable for C1M6.

Number of bridges 6, 0, 1, 2
Chi-square with 1DF 0.0044; p = 0.9469

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
66.159	161.6	0.6823	999

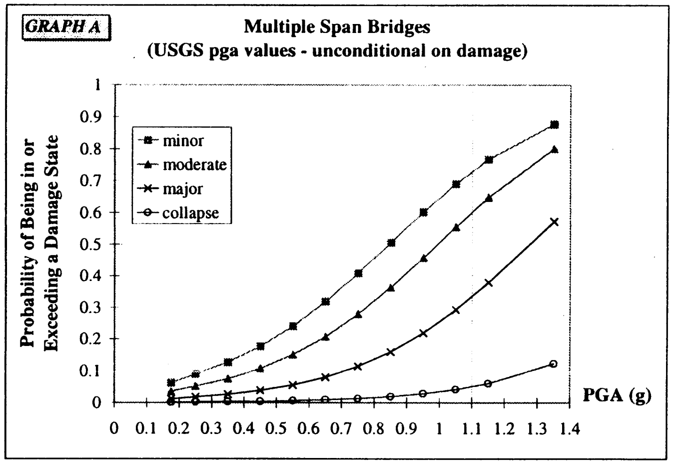
Validity of the model fit is questionable for C1M11.

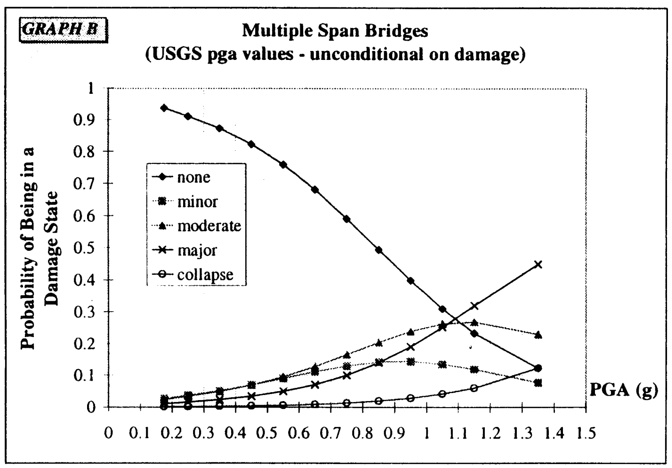
Number of bridges 9, 1
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
-93.432	1068.7	0.9303	0.00016

Number of bridges 715, 47, 52, 30,4
Chi-square with 3 DF 4.5136; p= 0.2111

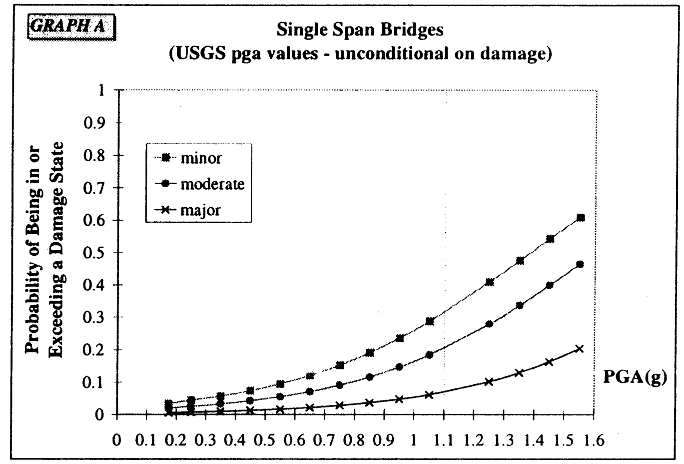
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.9028	0.3564	0.0001	49.543

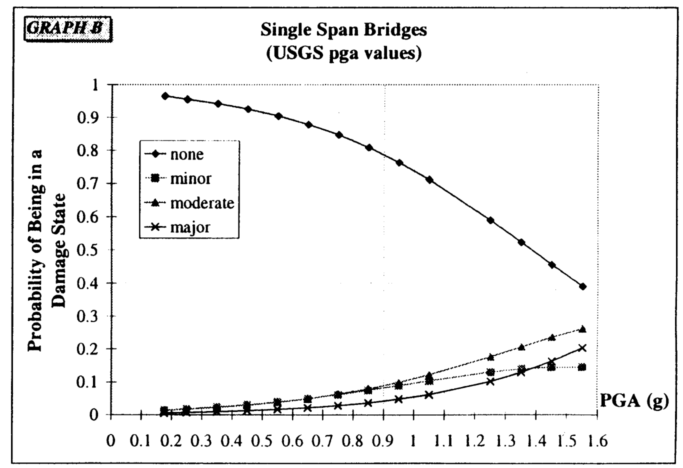




Number of bridges 306, 11, 11, 5
Chi-square with 2DF 4.1333; p = 0.1266

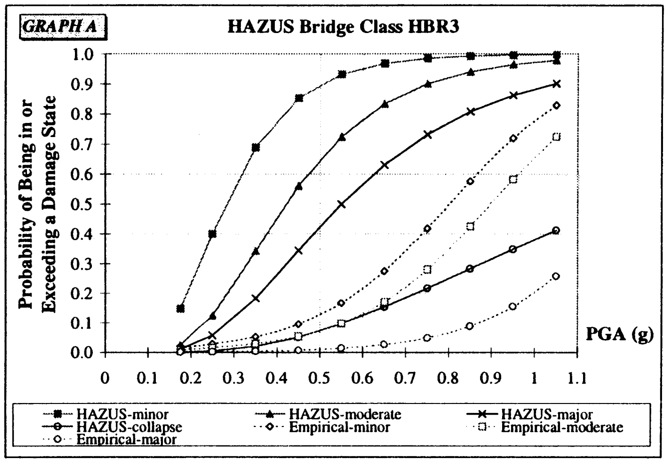
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.6972	0.5393	0.0001	14.838

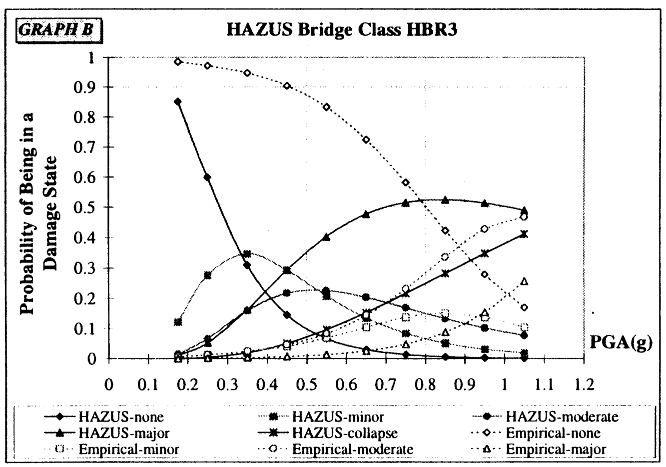




Number of bridges 213, 8, 15, 4
Chi-square with 2 DF 2.2965; p = 0.3172

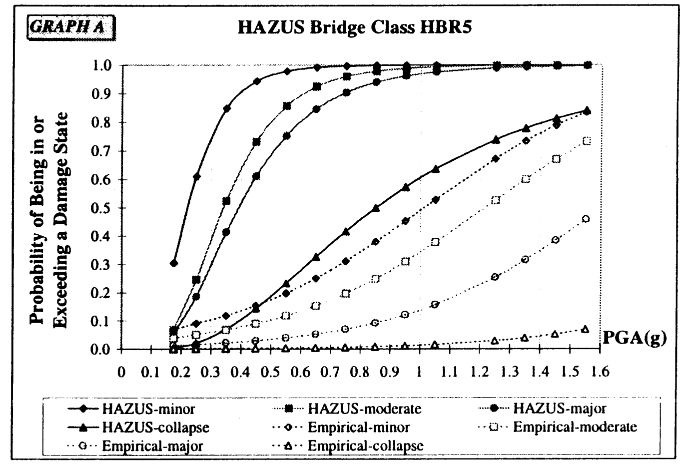
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
0.3866	0.9264	0.0001	593.821

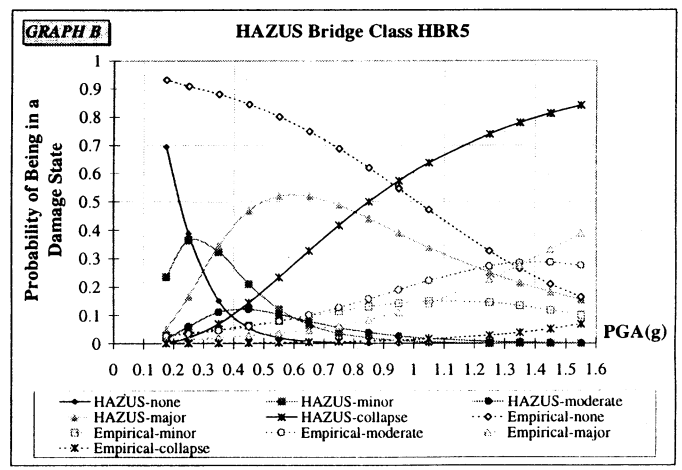




Number of bridges 539, 38, 39, 19, 2
Chi-square with 3 DF 7.0884; p = 0.0691

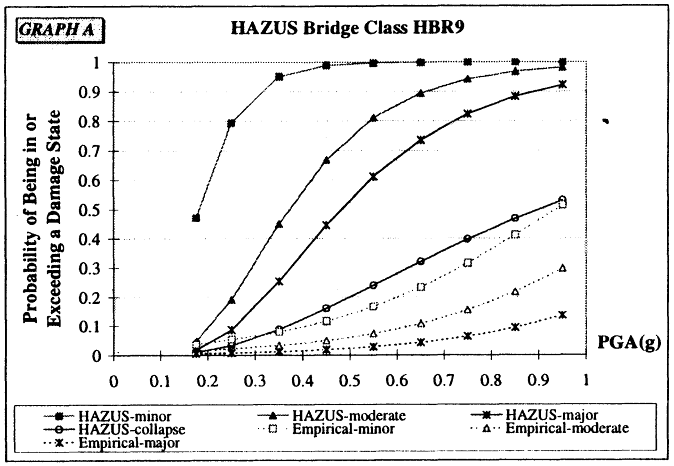
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.0228	0.3627	0.0001	20.5485

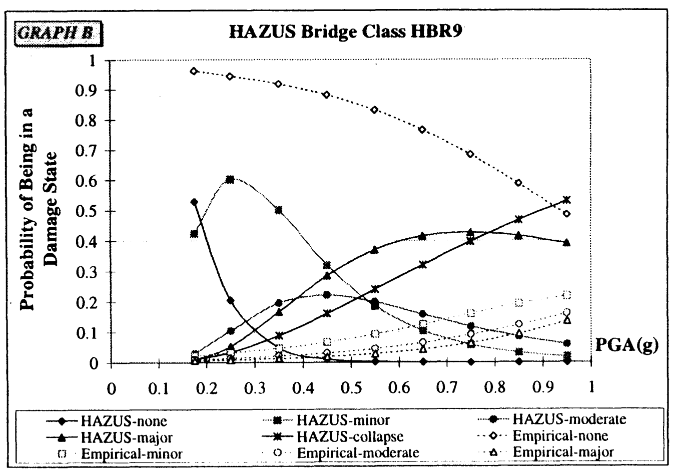




Number of bridges 95, 6, 3, 2
Chi-square with 2 DF 7.9886; p = 0.0184

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.1609	1.2202	0.0006	64.127

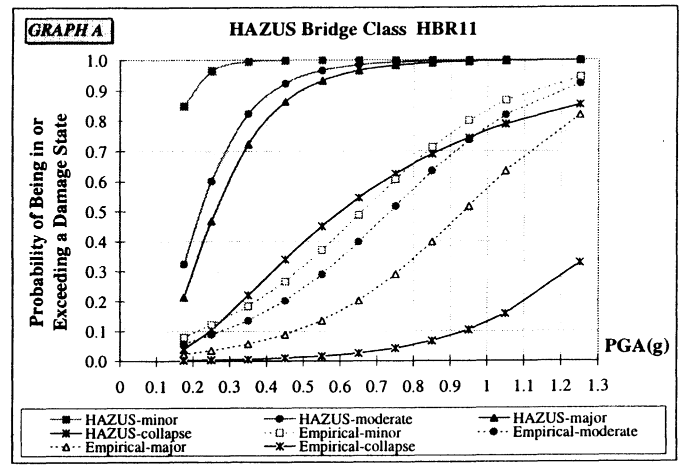


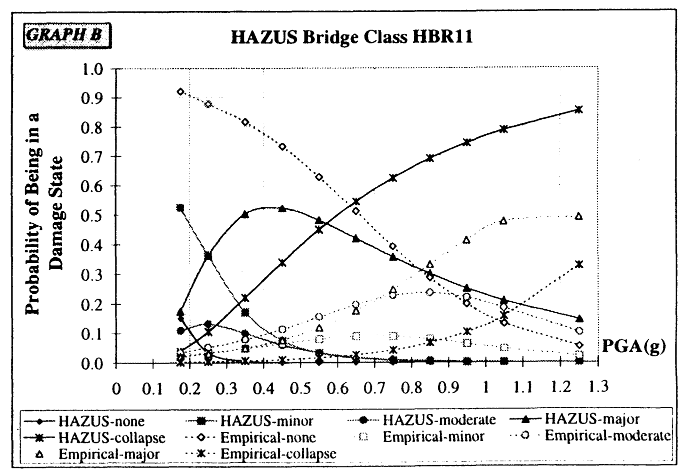


Northridge Earthquake *HAZUS Bridge Classes*

Number of bridges 405, 20, 34, 25, 4
Chi-square with 3 DF 5.1482; p = 0.1613

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.8128	0.5856	0.0001	123.079

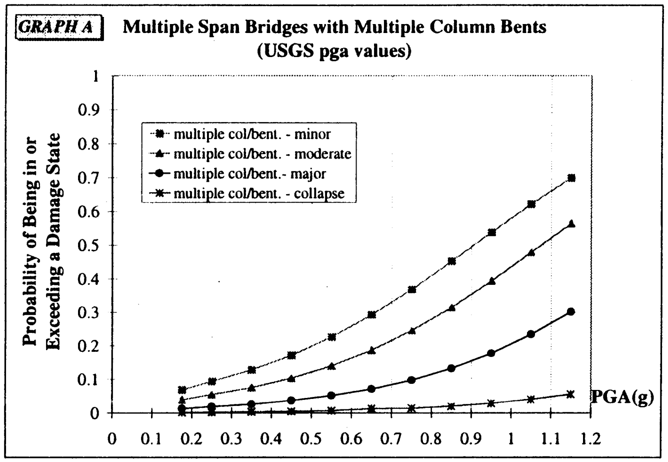


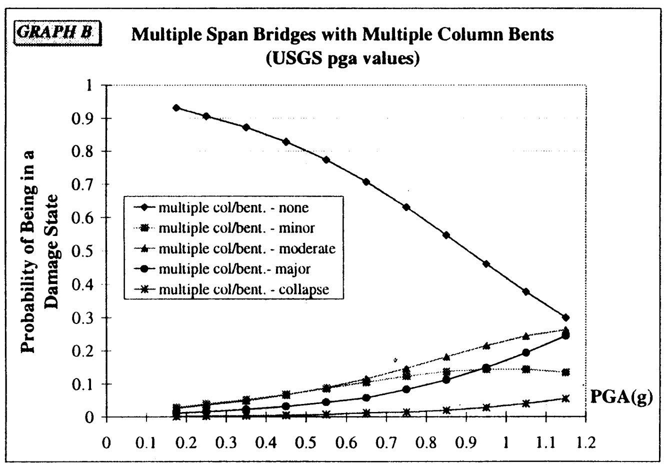


Northridge Earthquake — *Column Bent Type*

Number of bridges 452, 30, 32, 17, 3
Chi-square with 3 DF 2.9966; p = 0.39211

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.4584	0.4356	0.0001	31.766

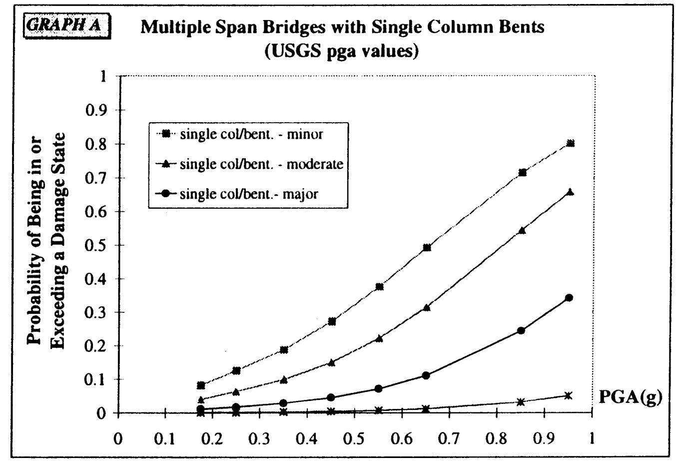


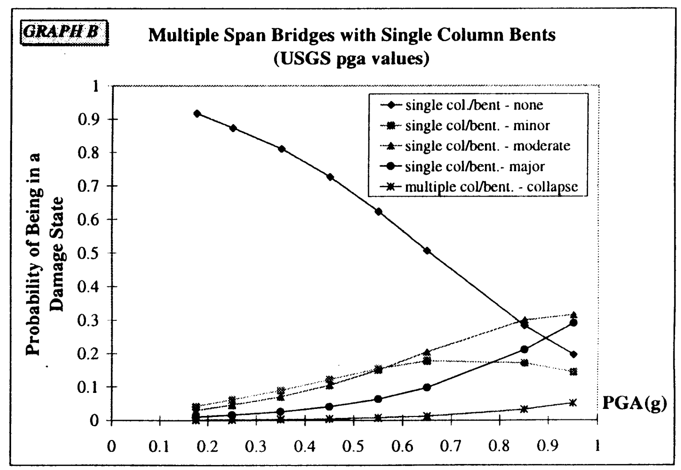


Northridge Earthquake — *Column Bent Type*

Number of bridges 94, 12, 13, 7, 1
Chi-square with 3 DF 1.7824; p = 0.6188

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.7701	0.8774	0.0001	117.933

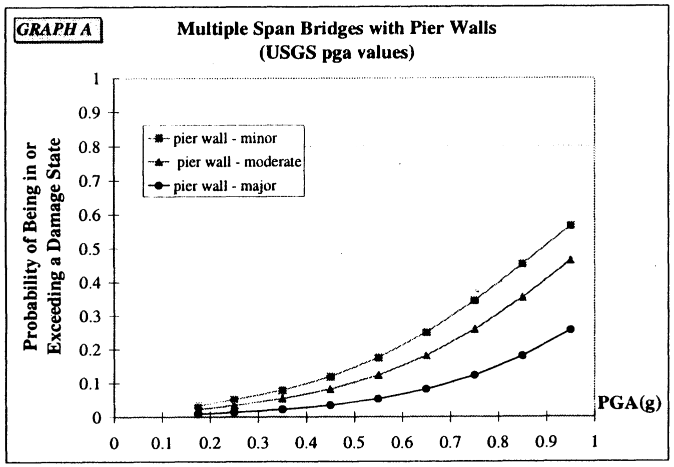


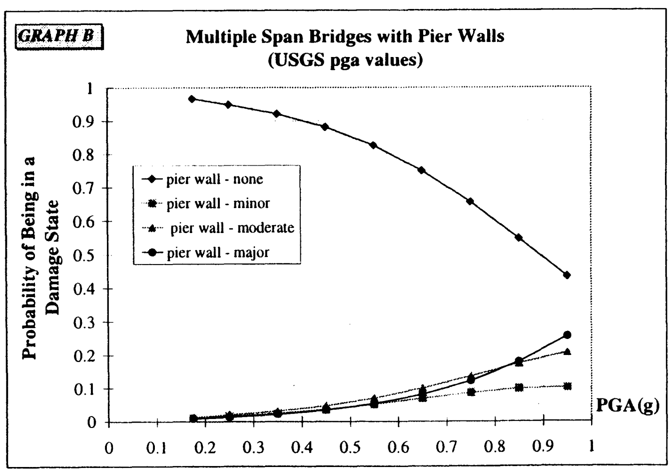


Northridge Earthquake *Column Bent Type*

Number of bridges 169, 5, 7, 6
Chi-square with 2 DF 2.6964; p = 0.2597

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.5109	0.9606	0.0001	93.779

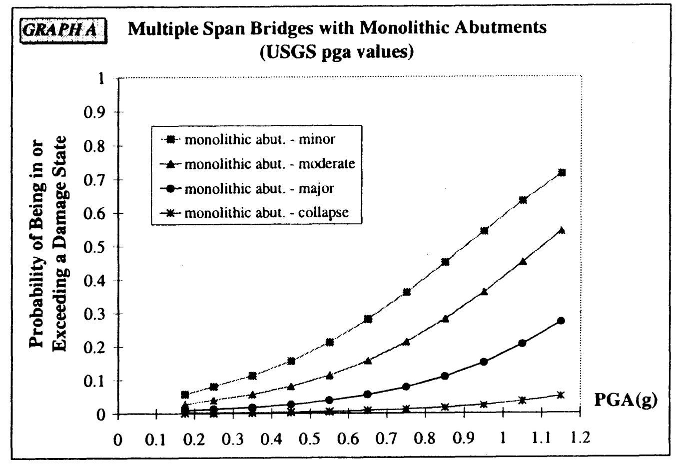


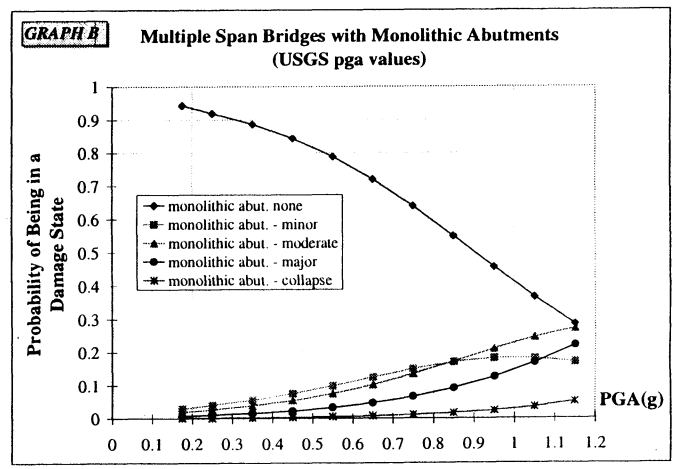


Northridge Earthquake *Abutment Type*

Number of bridges 341, 28, 23, 11, 2
Chi-square with 3 DF 1.3399; p = 0.7197

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.7219	0.4993	0.0001	41.342

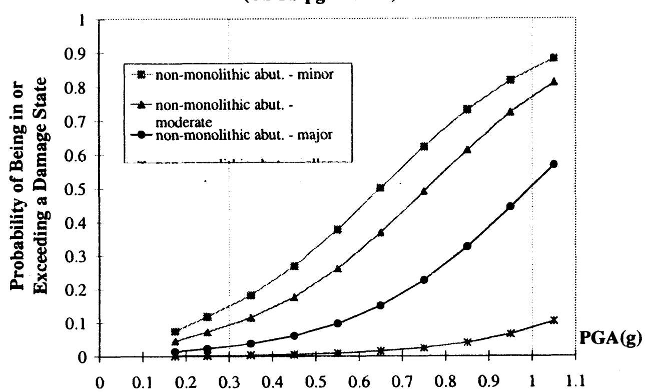




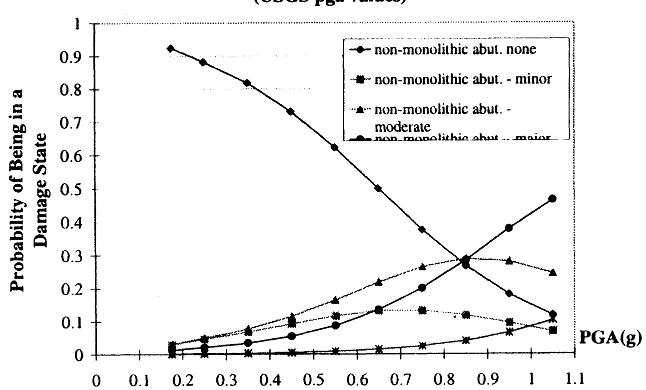
Number of bridges 259, 18, 25, 20, 3
Chi-square with 3 DF 3.5149; p = 0:3188

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.8862	0.4846	0.0001	48.725

GRAPH A **Multiple Span Bridges with Non-monolithic Abutments (USGS pga values)**



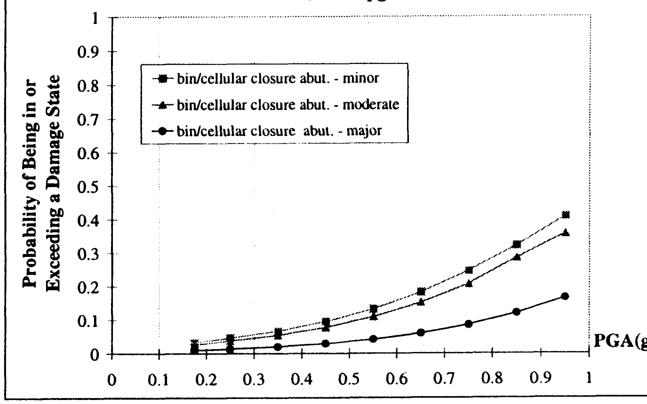
GRAPH B **Multiple Span Bridges with Non-monolithic Abutments (USGS pga values)**



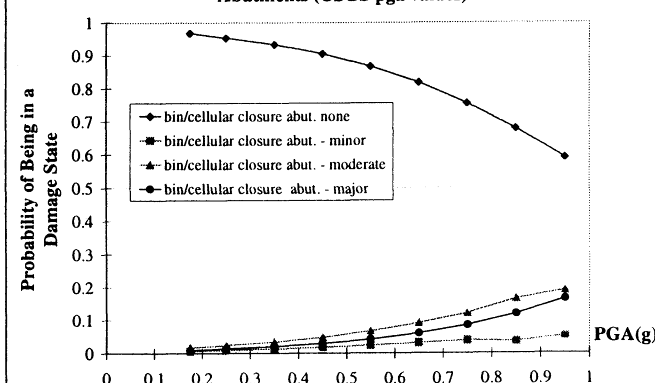
Number of bridges 124, 2, 6, 4
Chi-square with 2 DF 0.2012; p = 0.9043

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.7792	1.0837	0.0005	43.7802

GRAPH A **Multiple Span Bridges with Bin or Cellular Closure Type Abutments (USGS pga values)**

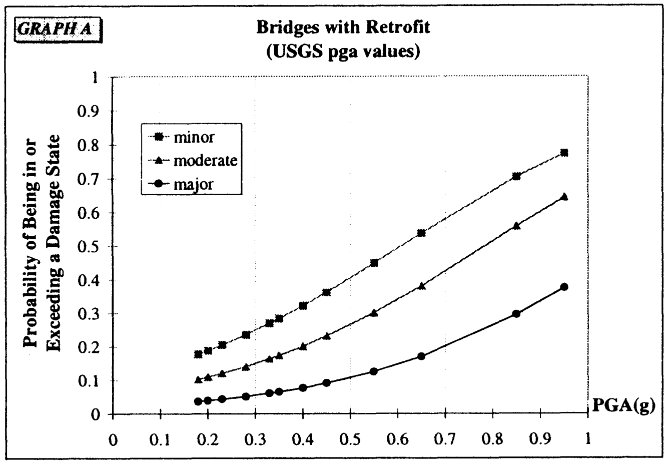


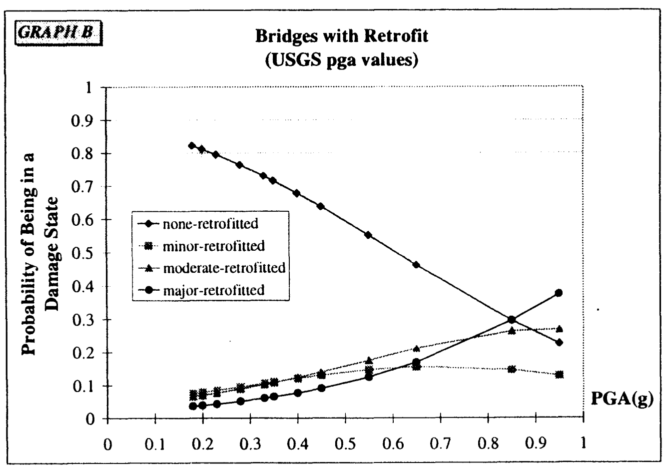
GRAPH B **Multiple Span Bridges with Bin or Cellular Closure Type Abutments (USGS pga values)**



Number of bridges 56, 8, 8, 5
Chi-square with 2 DF 0.8578; p = 0.6512

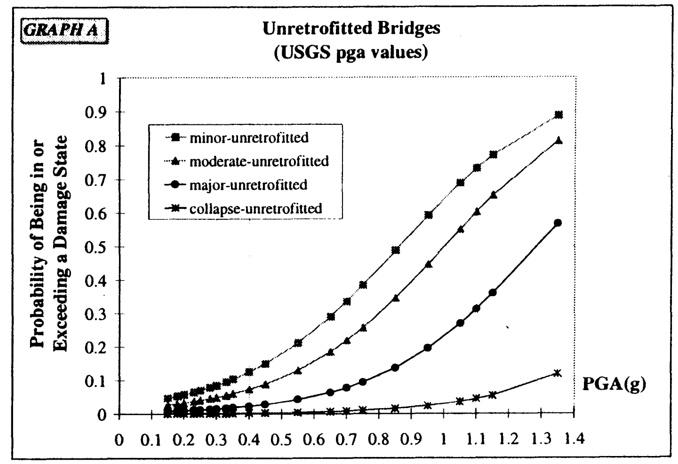
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.5876	1.7837	0.0443	36.148

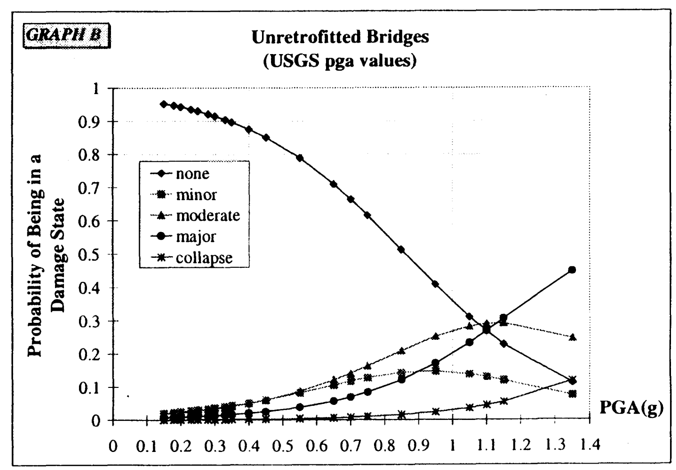




Number of bridges 660, 39, 45, 23, 3
Chi-square with 3 DF 3.2957; p = 0.3482

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.2241	0.3846	0.0001	68.316

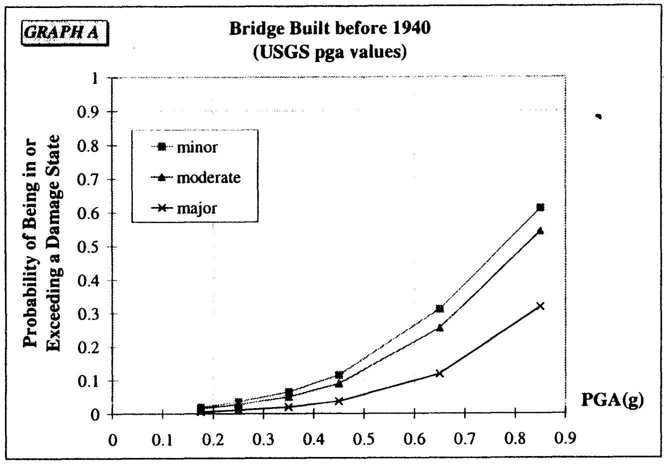


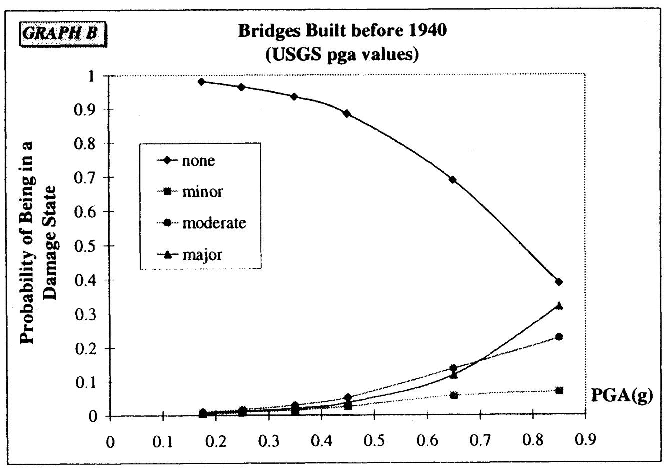


Northridge Earthquake *Design Year*

Number of bridges 55, 1, 2, 2
Chi-square with 2 DF 12.4432; p = 0.002

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.2569	2.0975	0.0029	521.58

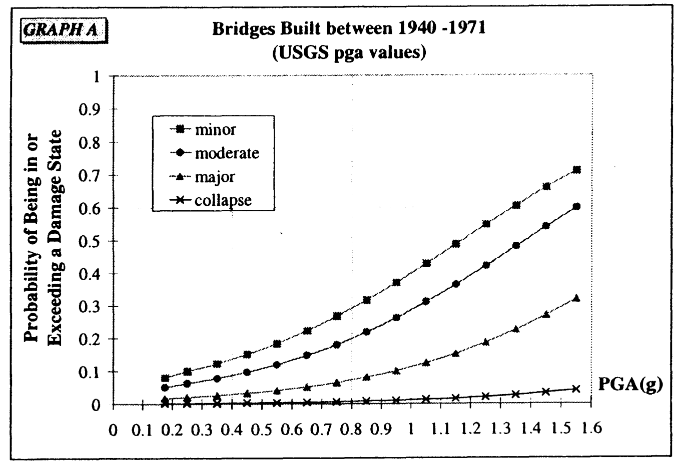


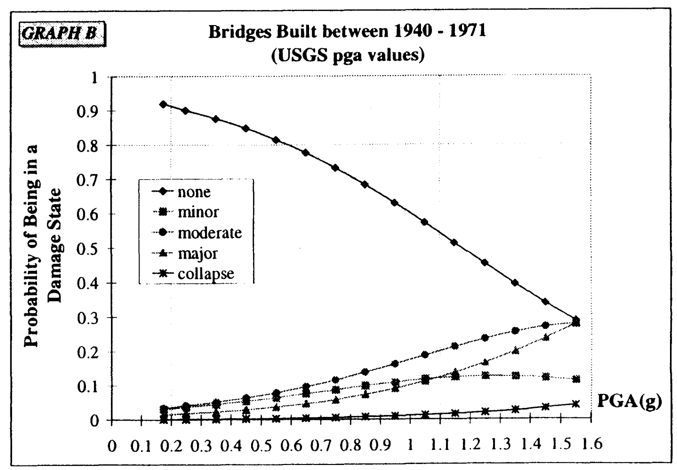


Northridge Earthquake *Design Year*

Number of bridges 887, 49, 59, 28, 3
Chi-square with 3 DF 9.5481; p = 0.0228

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.3905	0.3152	0.0001	10.9195

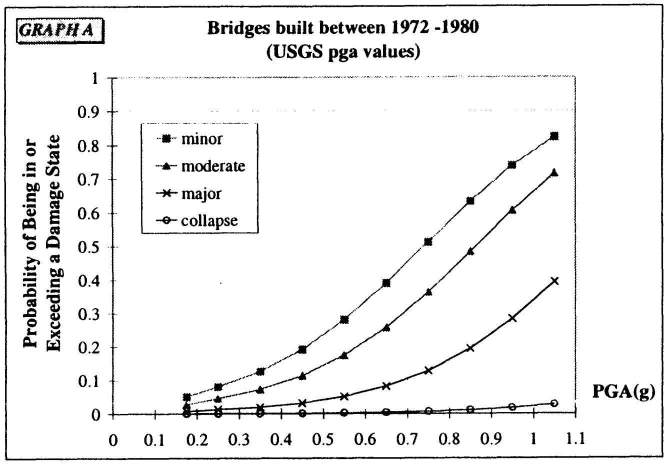


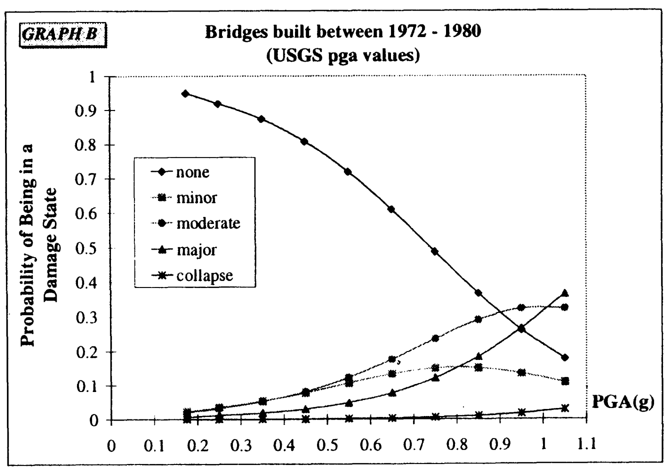


Northridge Earthquake *Design Year*

Number of bridges 206, 20, 30, 17, 1
Chi-square with 3DF 2.2139; p = 0.5292

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.9623	0.5877	0.0001	142.916

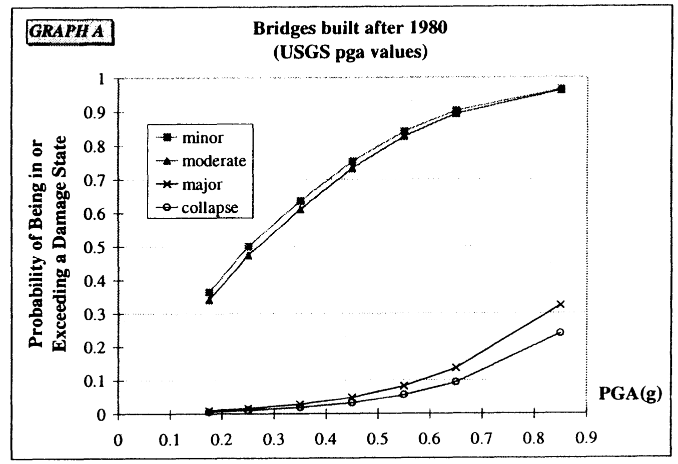


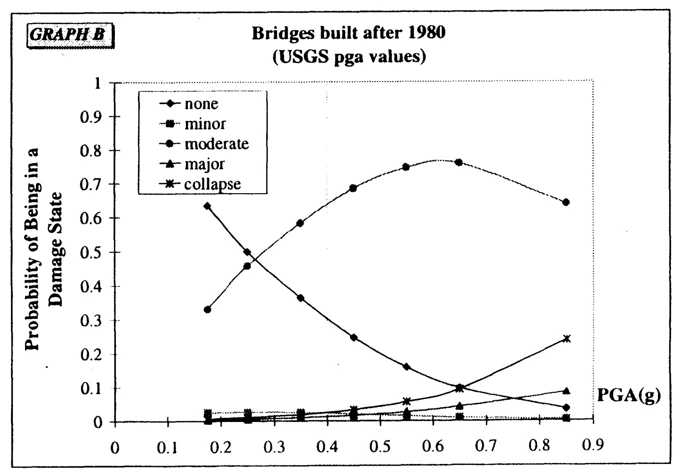


Northridge Earthquake *Design Year*

Number of bridges 64, 3, 55, 1, 2
Chi-square with 3DF 21.108; p = 0.0001

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
5.5719	1.6403	0.0007	262.928





APPENDIX F

EMPIRICAL FRAGILITY CURVES

Event: *Northridge earthquake.*

Dependent variable: *Damage state.*

Independent variable: *Peak ground acceleration based on WCFS [1995].*

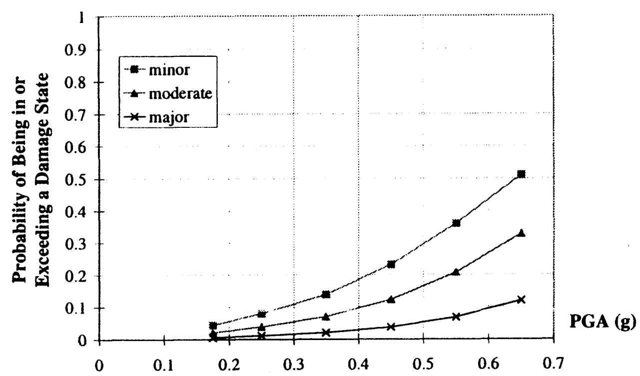
Grouped by	Data set
Bridge sub-categories	*correlation data set*
Number of spans	*correlation data set*
NIBS bridge classes	*highway bridge data set*
Design year	*highway bridge data set*

Number of bridges 204, 16, 13, 6
Chi-square with 2DF 1.4793; p = 0.4773

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.2109	1.2995	0.0001	498.14

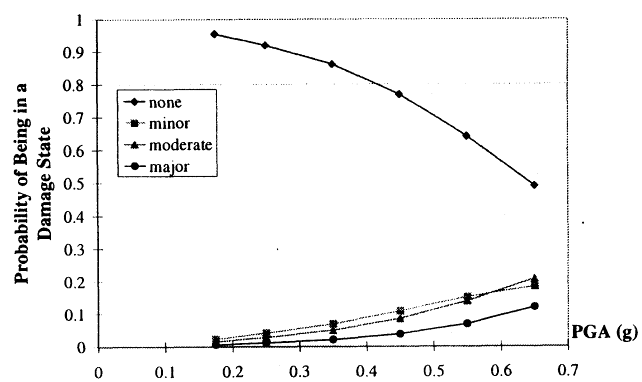
GRAPH A

Bridge Sub-category C1M1
(WCFS pga values - unconditional on damage)



GRAPH B

Bridge Sub-category C1M1
(WCFS pga values - unconditional on damage)

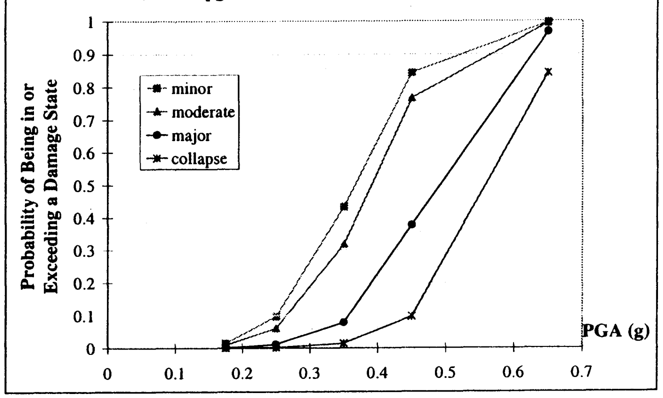


Number of bridges 20, 1, 2, 1, 2
Chi-square with 3DF 6.2316; p = 0.1009

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
19.55	5.9534	0.001	999

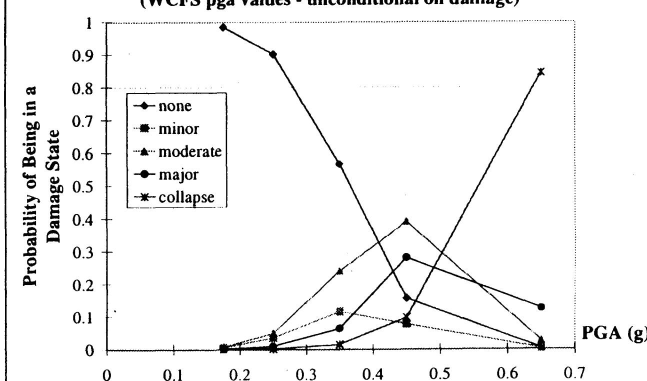
GRAPH A

Bridge Sub-category C1M2
(WCFS pga values - unconditional on damage)



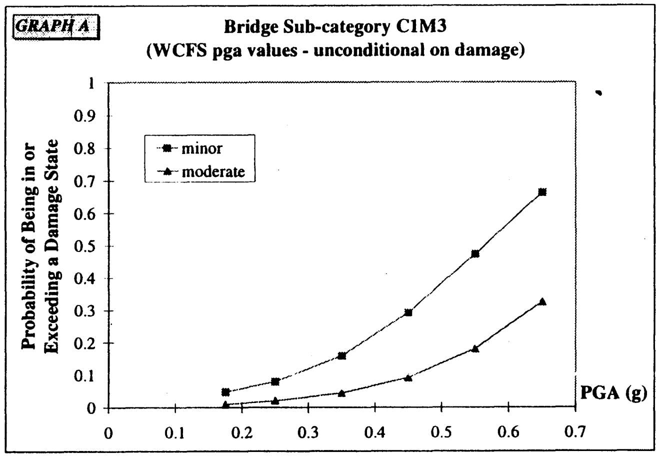
GRAPH B

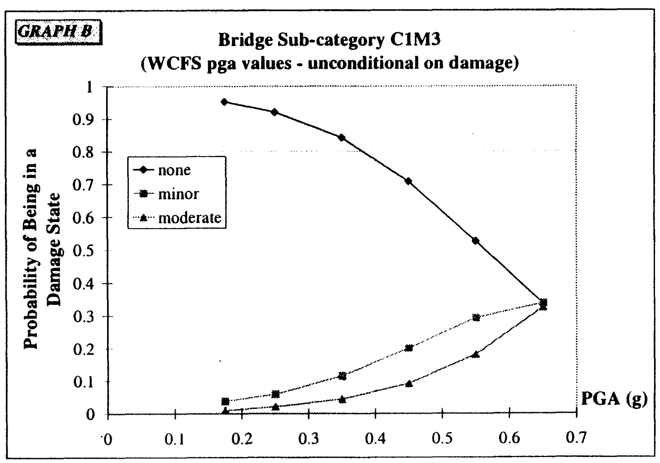
Bridge Sub-category C1M2
(WCFS pga values - unconditional on damage)



Number of bridges 30, 5, 3
Chi-square with 1DF 0.0264; p = 0.871

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
7.8111	2.8565	0.0062	999

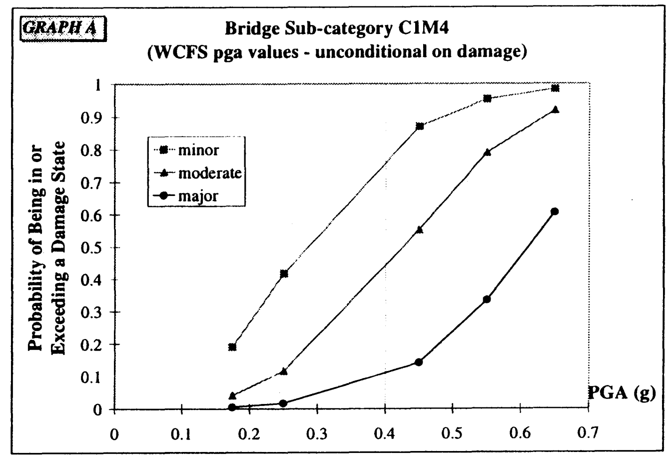


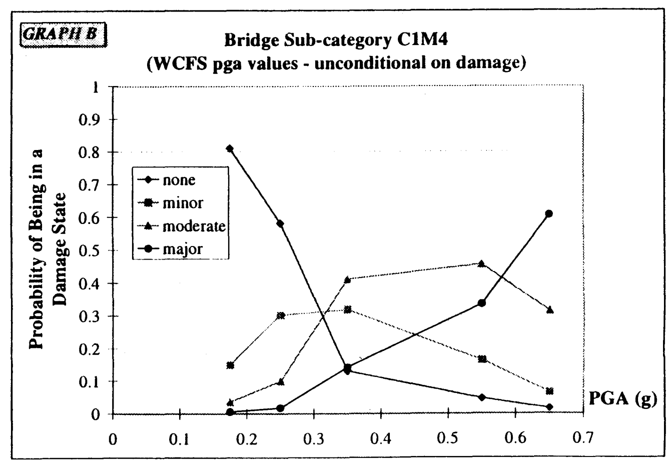


Northridge Earthquake *Bridge Sub-categories*

Number of bridges 8, 4, 4,2
Chi-square with 2DF 3.8478; p = 0.146

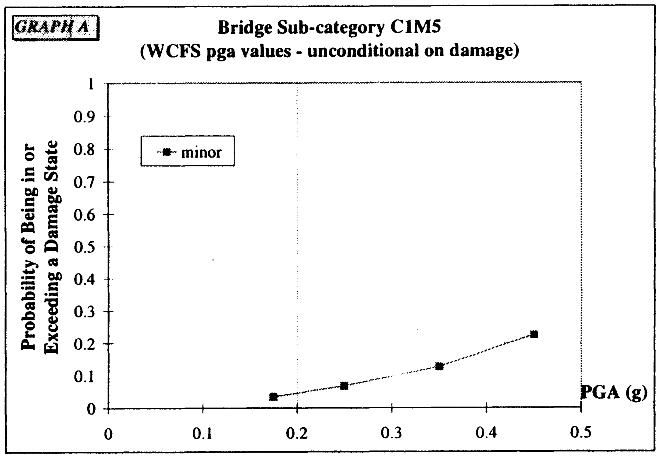
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
11.1559	3.9589	0.0048	999

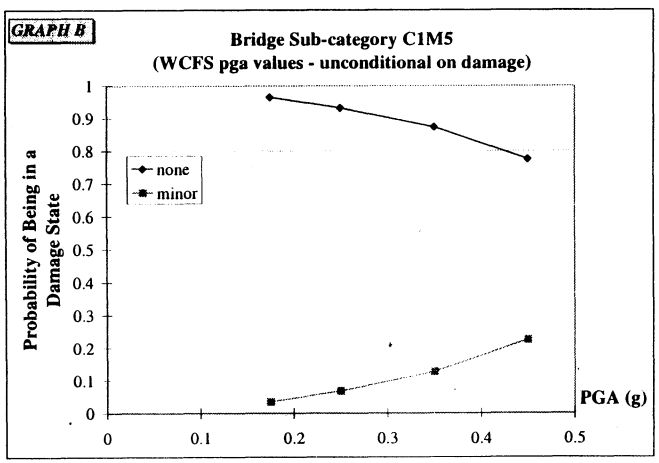




Number of bridges 33, 2
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.9151	7.9218	0.3827	999

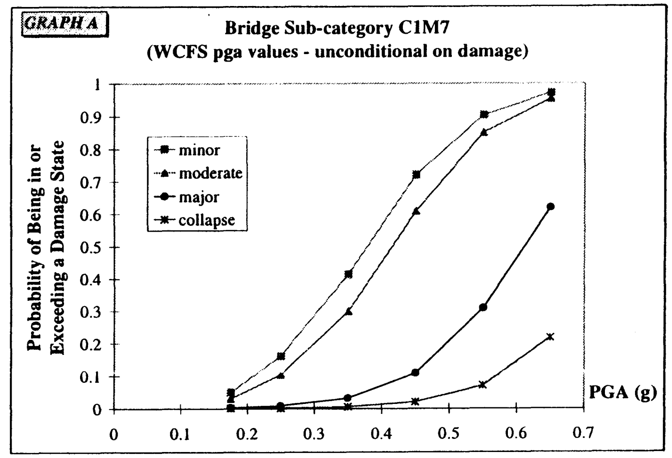


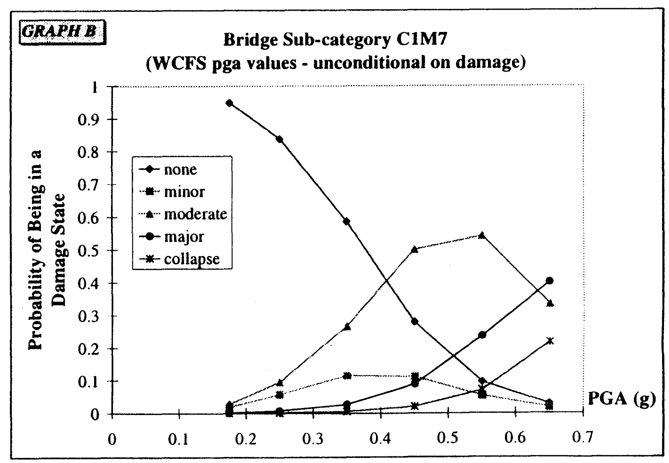


Northridge Earthquake *Bridge Sub-categories*

Number of bridges 134, 8, 17, 3,1
Chi-square with 3DF 1.9193, p = 0.5893

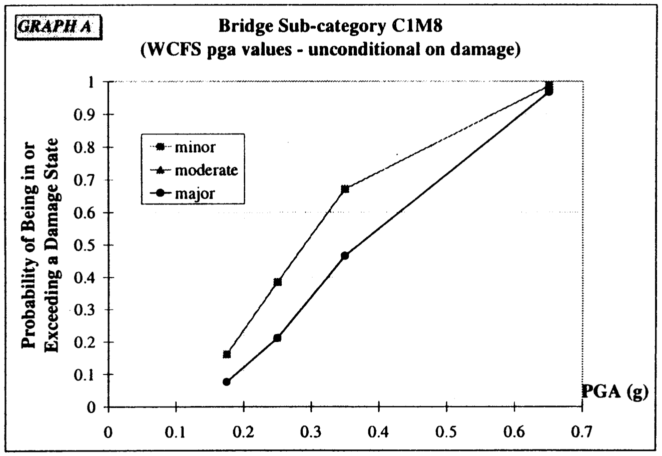
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
12.942	2.2732	0.0001	999

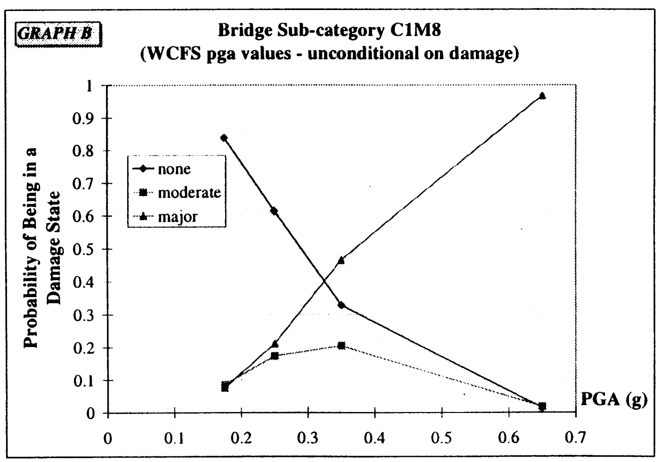




Number of bridges 16, 3, 0, 7
Chi-square with 1DF 9.2694, p = 0.0023

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
11.7992	4.7936	0.0138	999

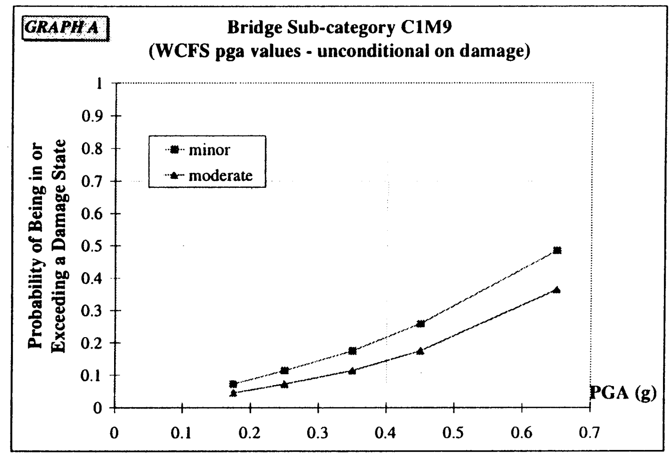


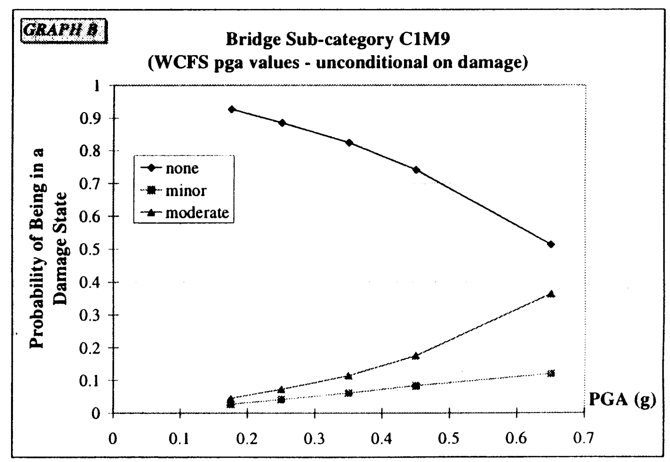


Northridge Earthquake | Bridge Sub-categories

Number of bridges 37, 2, 4
Chi-square with 1DF 0.27; p = 0.6034

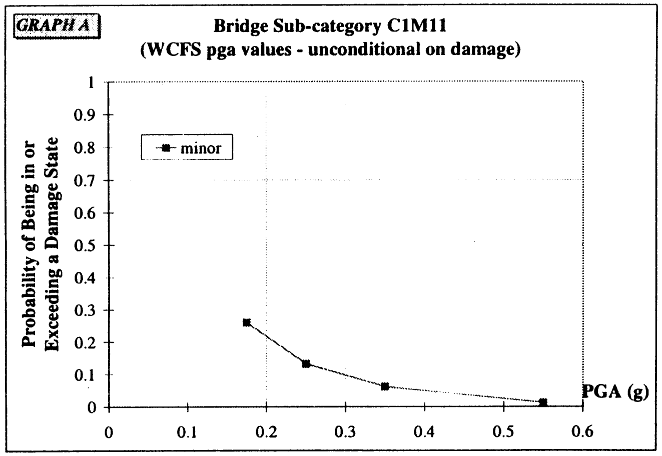
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.9732	2.5952	0.0553	144.49

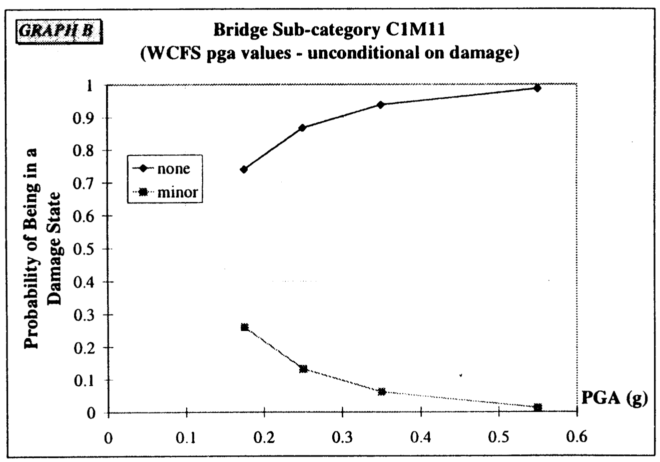




Number of bridges 9, 1
Chi-square N/A

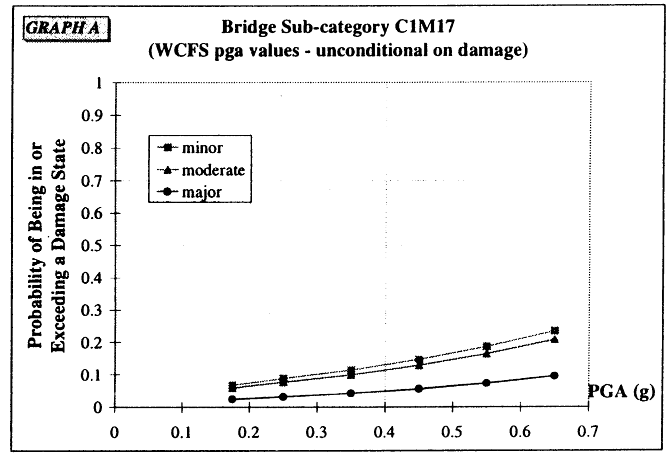
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
-8.3485	13.1055	0.5241	0

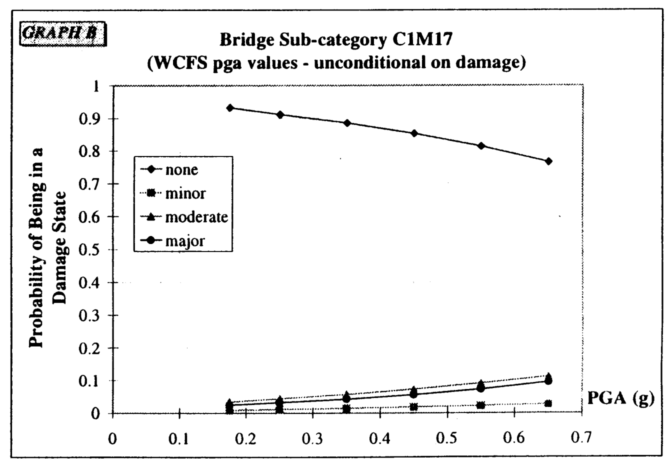




Number of bridges 71, 1, 4, 3
Chi-square with 2DF 8.6392; p = 0.0133

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.878	2.43	0.2363	17.778

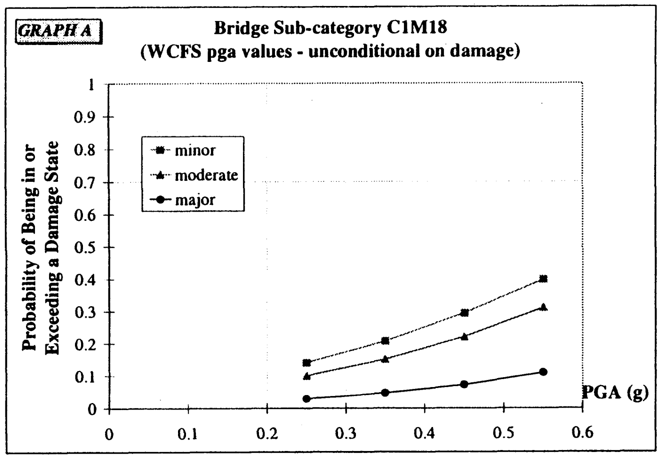


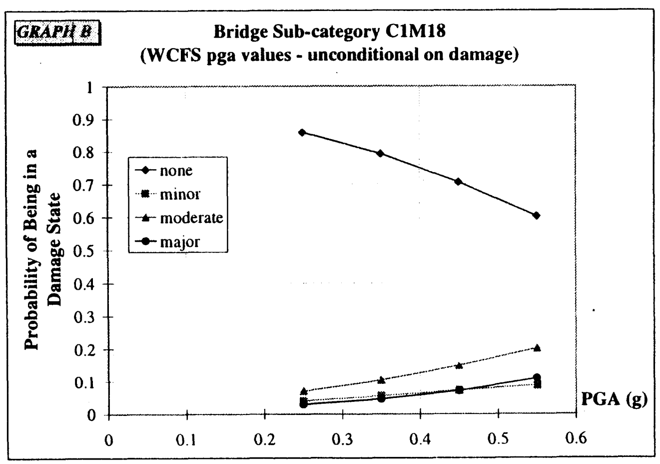


Northridge Earthquake *Bridge Sub-categories*

Number of bridges 11, 1, 2, 1
Chi-square with 2DF 0.7771; p = 0.6781

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.6576	5.2	0.3704	105.382

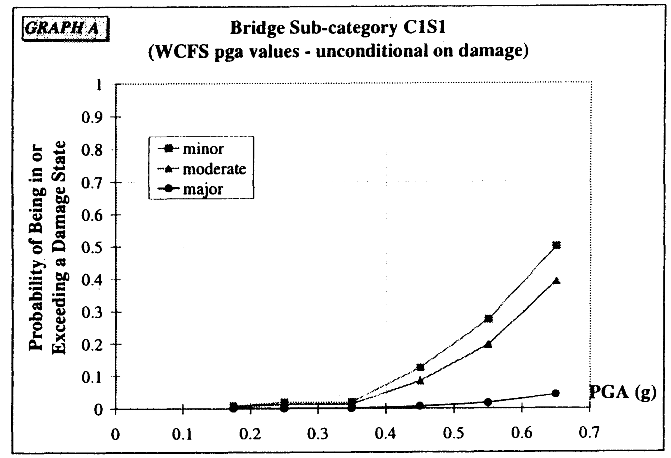


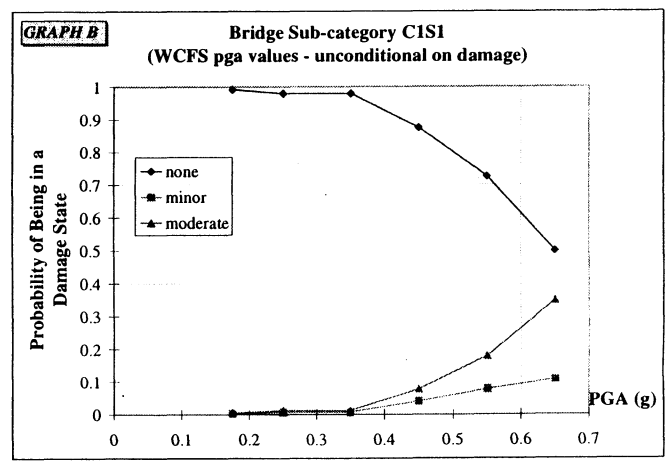


Northridge Earthquake *Bridge Sub-categories*

Number of bridges 175, 5, 11, 1
Chi-square with 2DF 2.257; p = 0.3235

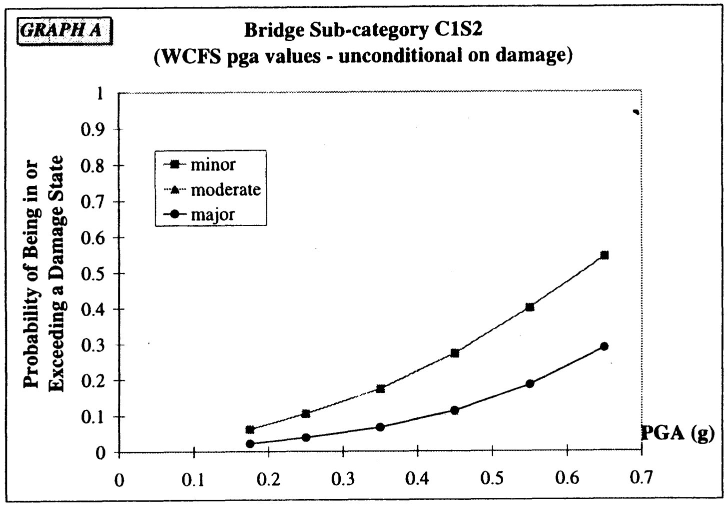
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
9.7236	2.088	0.0001	999

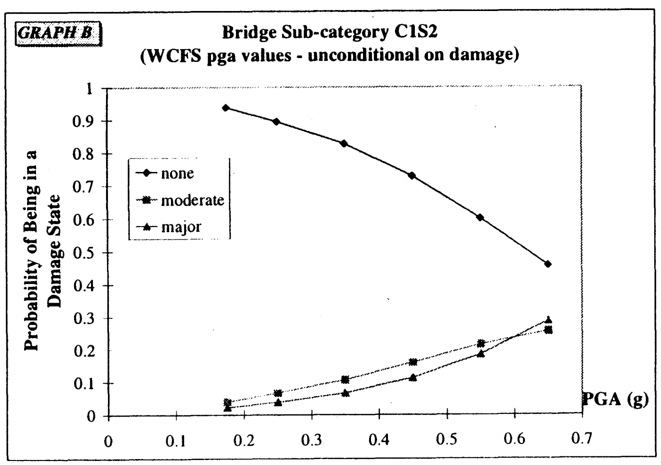




Number of bridges 66, 6, 4
Chi-square with 1DF 0.5126; p = 0.474

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
5.7724	2.1416	0.007	321.318





Validity of the model fit is questionable, C1M6.

Number of bridges 2, 1, 2
Chi-square with 1DF 0.0005, p = 0.982

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
89.514	455.4	0.8442	999

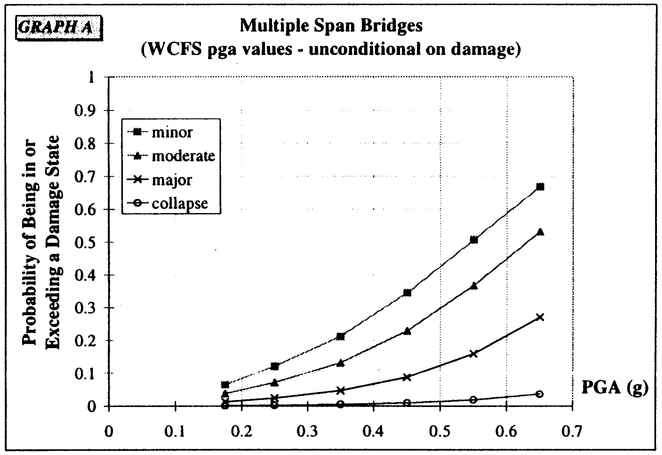
Validity of the model fit is questionable, C1M10.

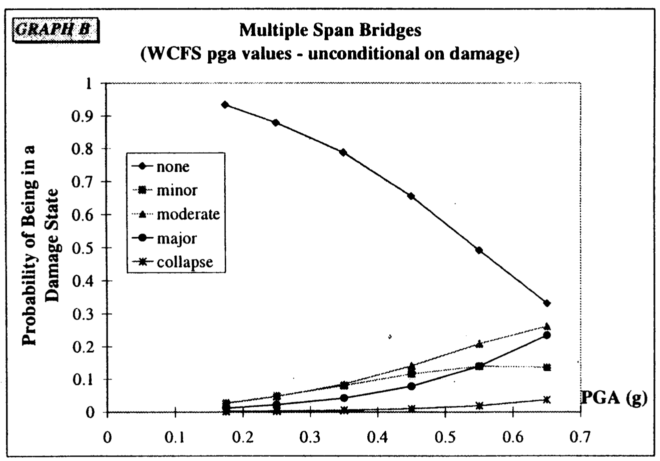
Number of bridges 9, 1, 2, 5, 1
Chi-square with 3DF 0.0005; p = 1.0

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
100.5	470.9	0.8311	999

Number of bridges 593, 45, 52, 30, 4
Chi-square with 3DF 7.0473; p = 0.0704

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.7163	0.6602	0.0001	825.735

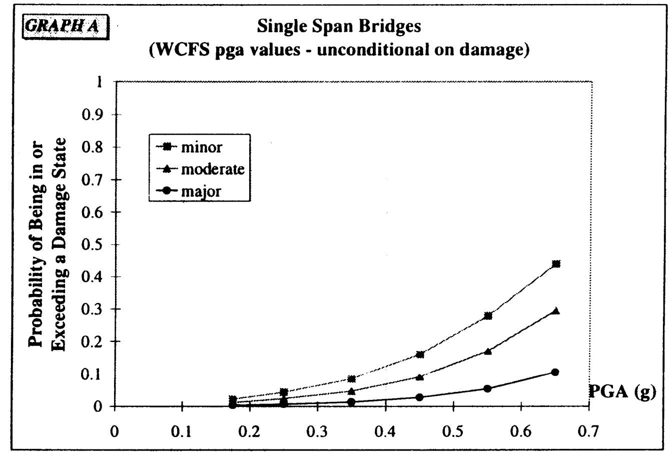


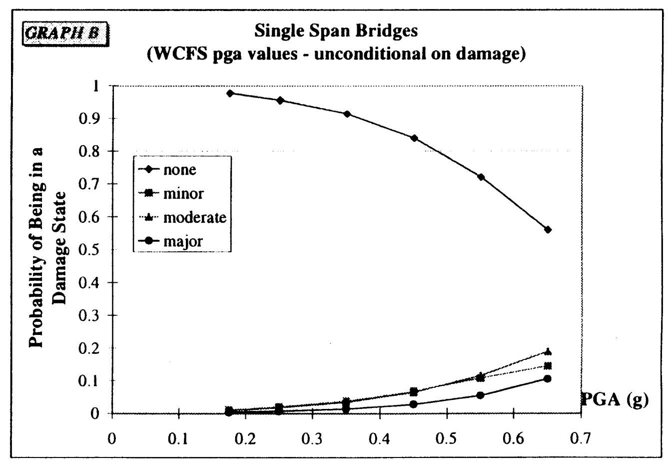


Northridge Earthquake *Number of Spans*

Number of bridges 252, 11, 11, 5
Chi-square with 2DF 1.6306; p = 0.4425

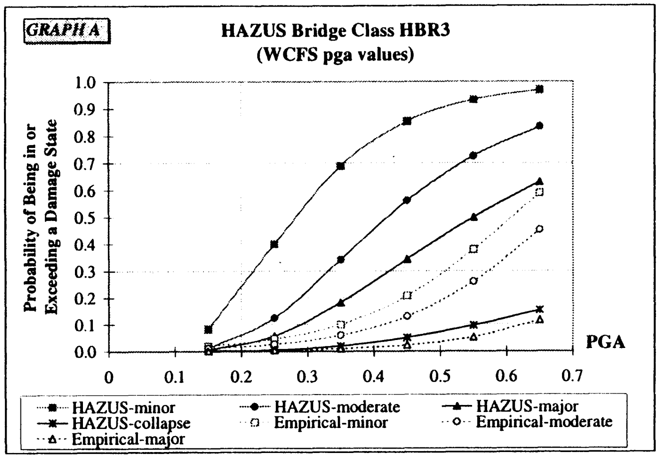
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
7.0749	1.3492	0.0001	999

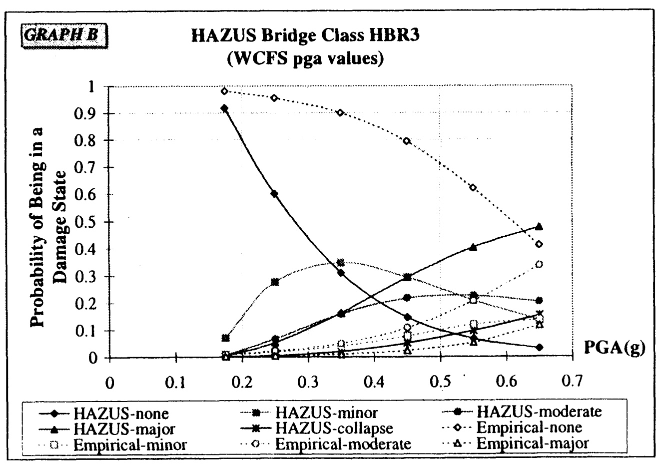




Number of bridges 181, 9, 15 ,4
Chi-square with 2 DF 0.334; p = 0.8462

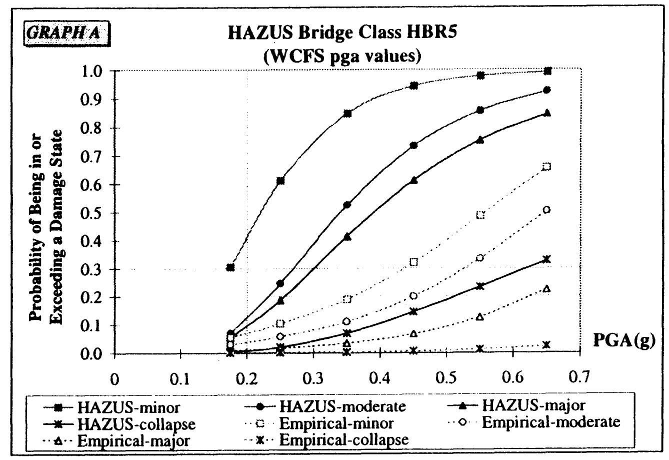
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
8.5247	1.5197	0.0001	999

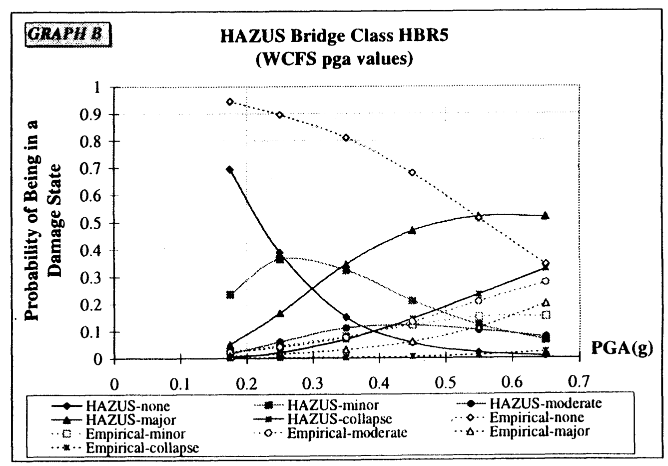




Number of bridges 463, 36, 39, 19, 2
Chi-square with 3 DF 7.3775; p = 0.0608

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.9848	0.7831	0.0001	999

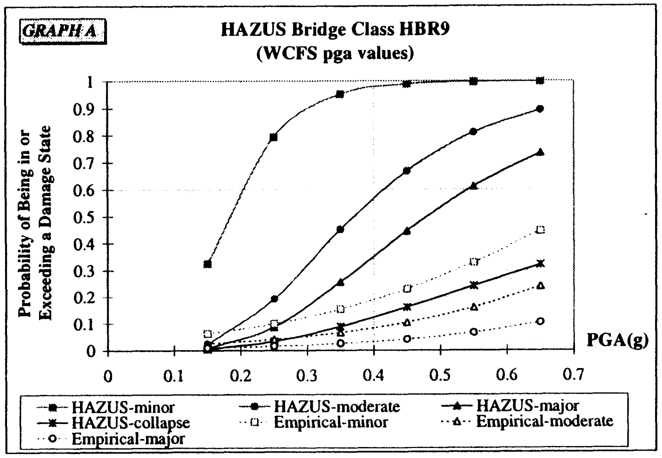


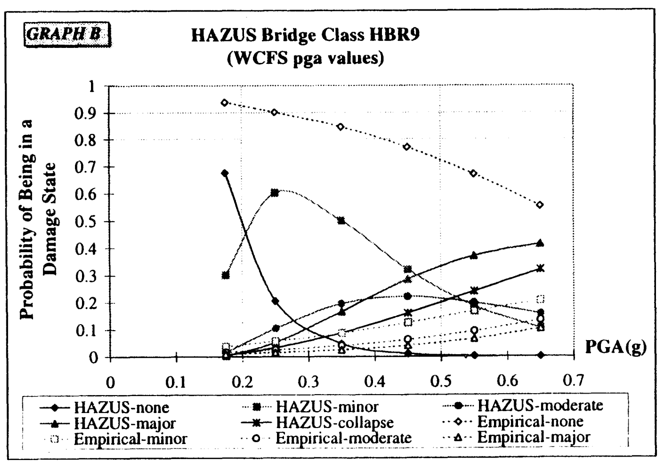


Northridge Earthquake *HAZUS Bridge Classes*

Number of bridges 62, 6, 3, 2
Chi-square with 2DF 0.2788; p = 0.8699

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.9693	2.0296	0.0144	143.925

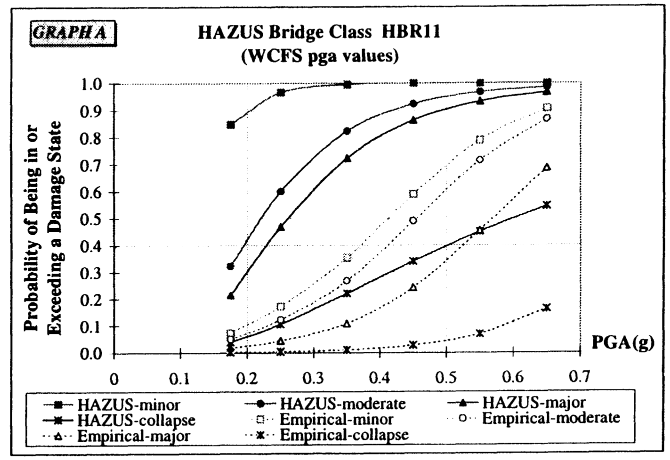


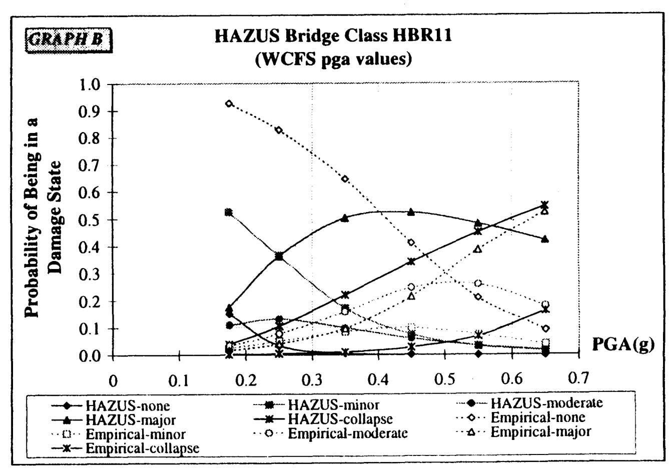


Northridge Earthquake *HAZUS Bridge Classes*

Number of bridges 341, 20, 34, 25, 4
Chi-square with 3DF 10.9637; p = 0.0119

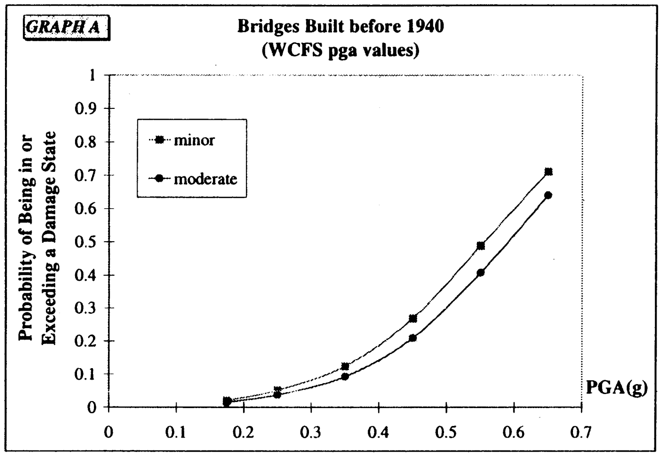
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
0.6448	1.0649	0.0001	999

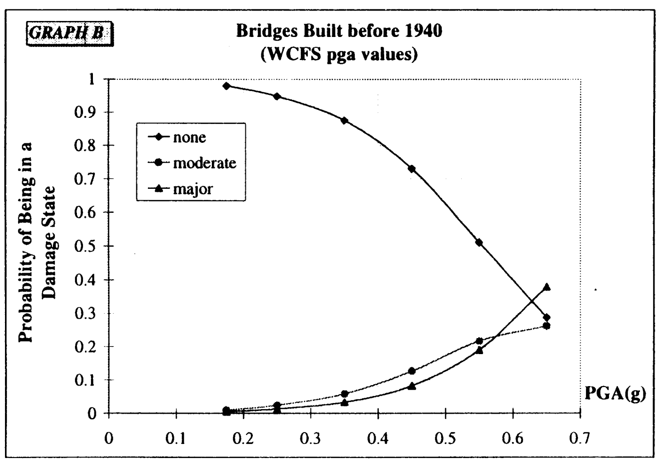




Number of bridges 49, 1, 2, 2
Chi-square with 2 DF 11.8379; $p = 0.0027$

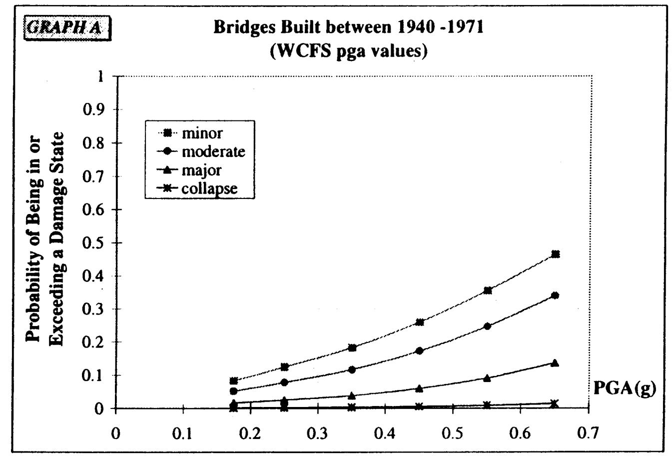
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
9.5429	2.9618	0.0013	999

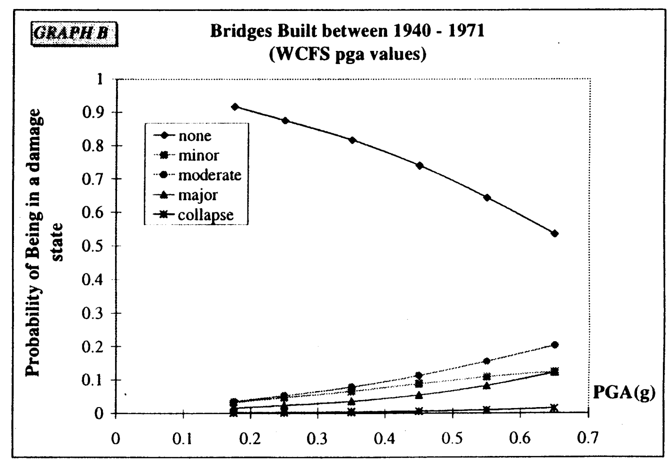




Number of bridges 768, 49, 59, 28, 3
Chi-square with 3 DF 7.546; $p = 0.0564$

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.5165	0.6046	0.0001	91.5153

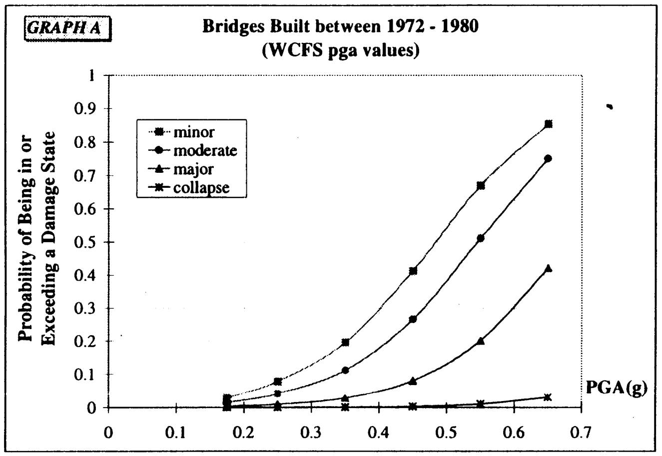


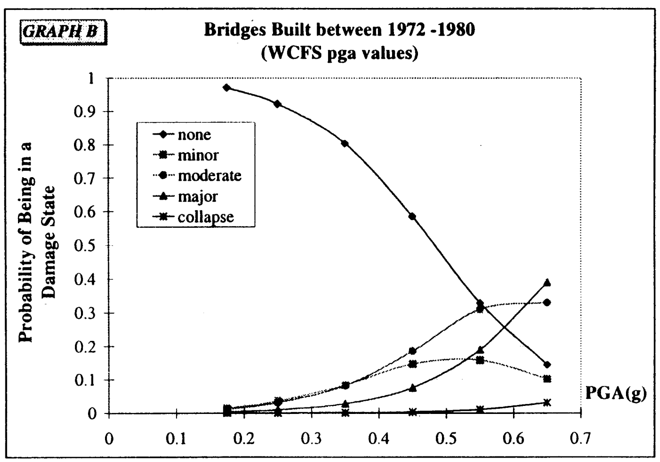


Northridge Earthquake — *Design Year*

Number of bridges 202, 20, 30, 17, 1
Chi-square with 3DF 1.047; p = 0.7899

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
10.6047	1.2555	0.0001	999

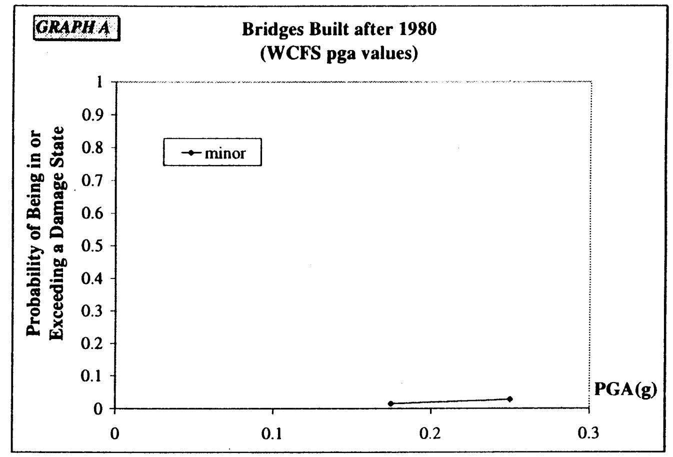


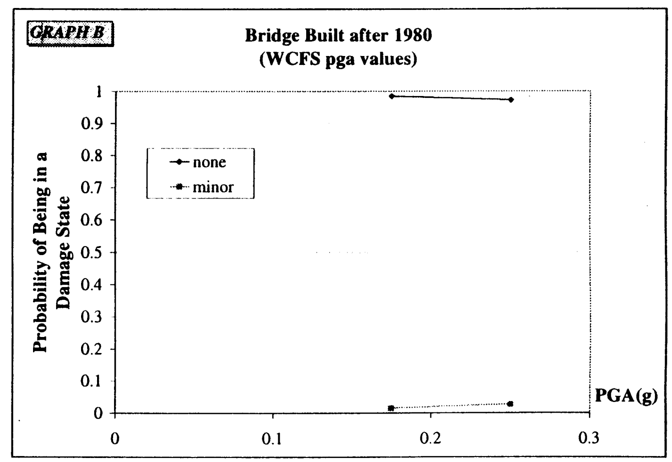


Northridge Earthquake — *Design Year*

Number of bridges 43, 1
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
5.8307	8.1824	0.4761	340.614





APPENDIX G

EMPIRICAL FRAGILITY CURVES

Event: *Northridge earthquake.*

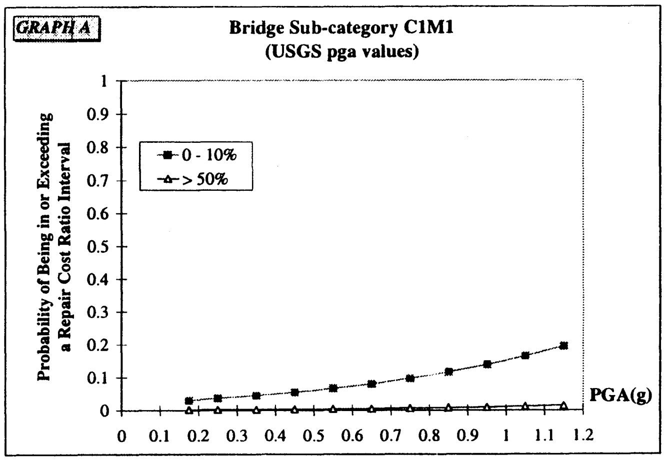
Dependent variable: *Repair cost ratio.*

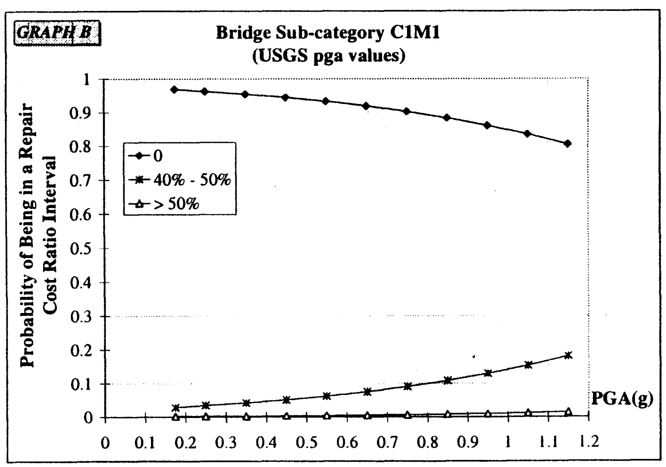
Independent variable: *Peak ground acceleration based on USGS [1994].*

Grouped by	Data set
Bridge sub-categories	*correlation data set*
Number of spans	*correlation data set*
Design year	*highway bridge data set*

Number of bridges 248, 14, 0, 0, 0, 0, 1
Chi-square with 1DF 4.2087; p = 0.0402

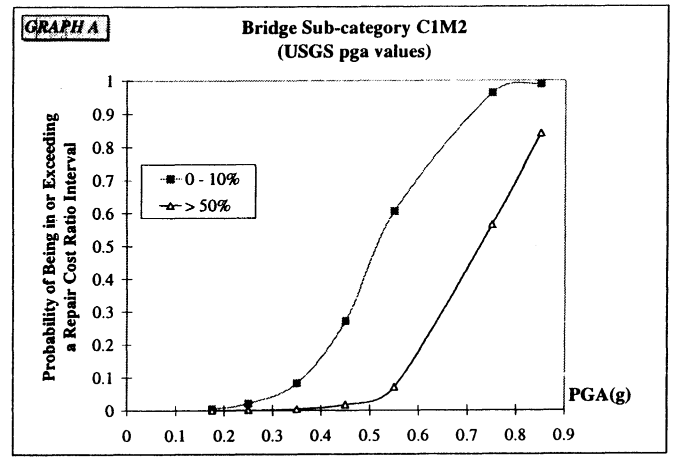
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.0366	0.8679	0.019	7.666

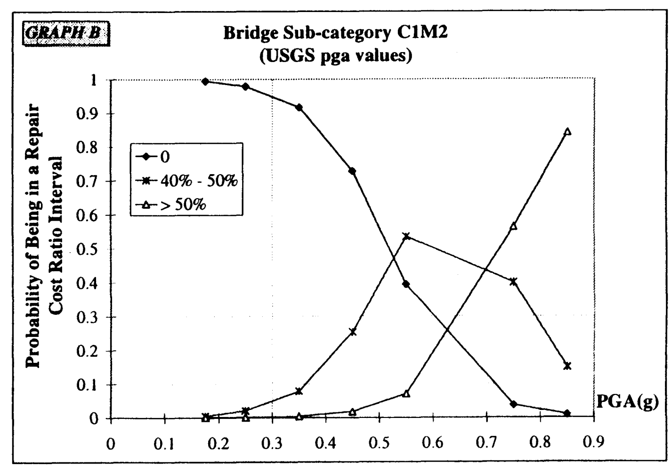




Number of bridges 25, 3, 0, 0, 0, 0, 2
Chi-square with 1DF 1.2325; p = 0.2669

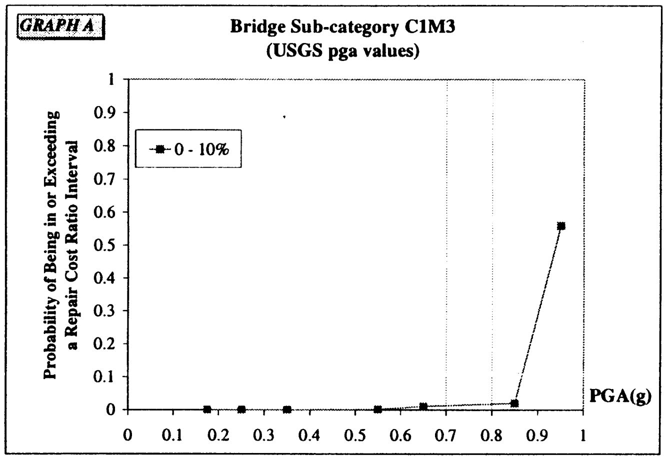
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
14.1502	4.9897	0.0046	999

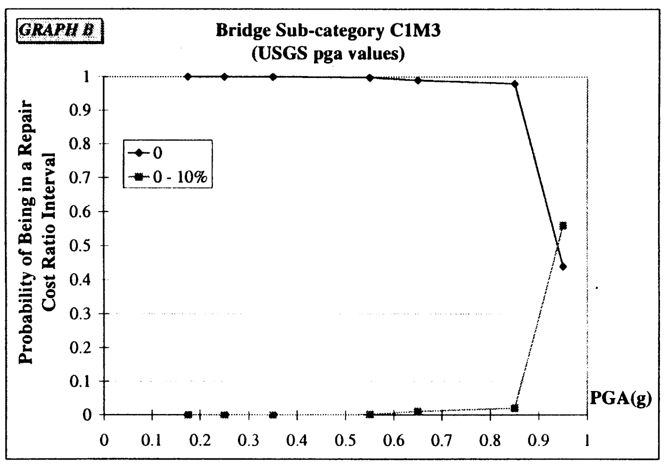




Number of bridges 41, 2
Chi-square N/A

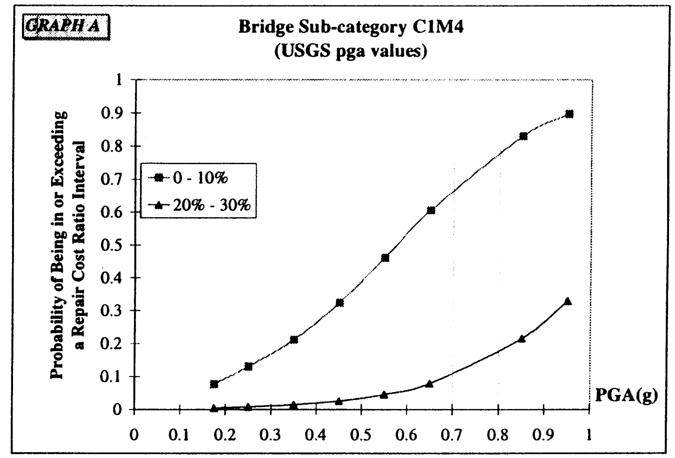
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
15.8729	12.2571	0.1953	999

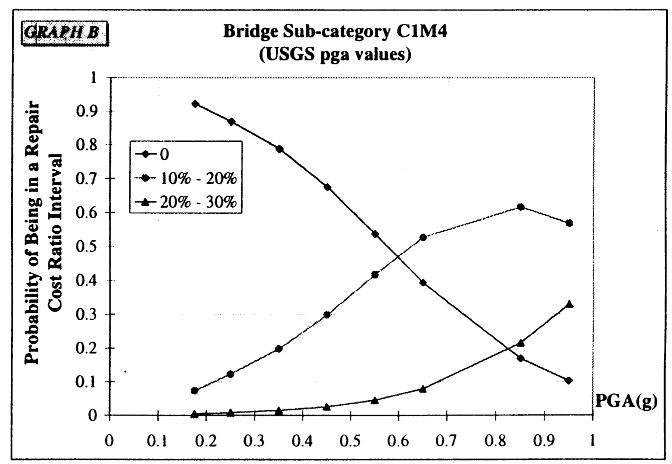




Number of bridges 12, 5, 0, 1
Chi-square with 1DF 1.2358; p = 0.2663

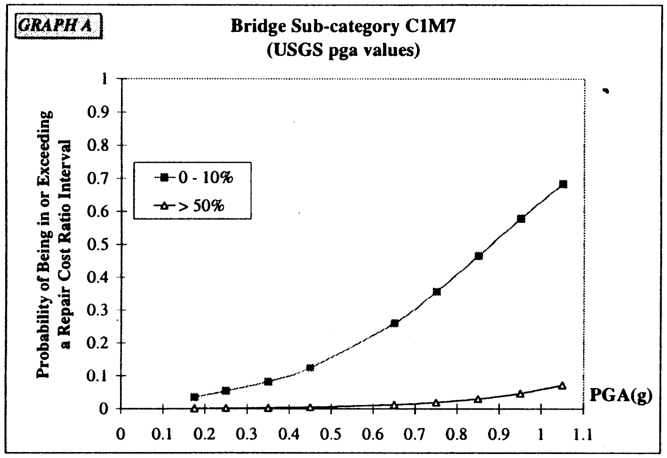
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
5.8143	2.5652	0.0234	335.044

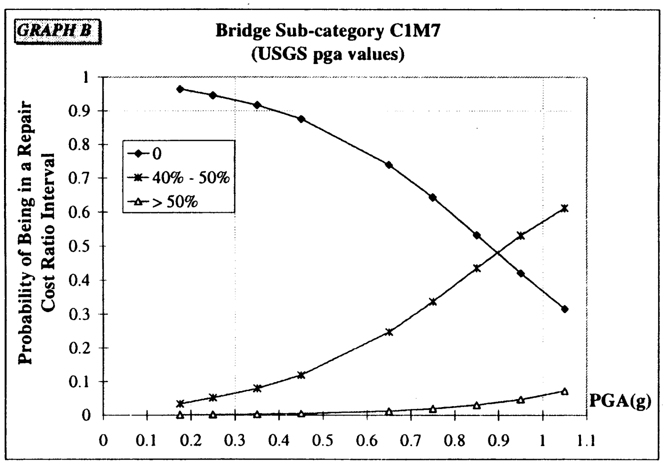




Number of bridges 182, 17, 0, 0, 0, 0, 1
Chi-square with 1DF 1.2726; p = 0.2593

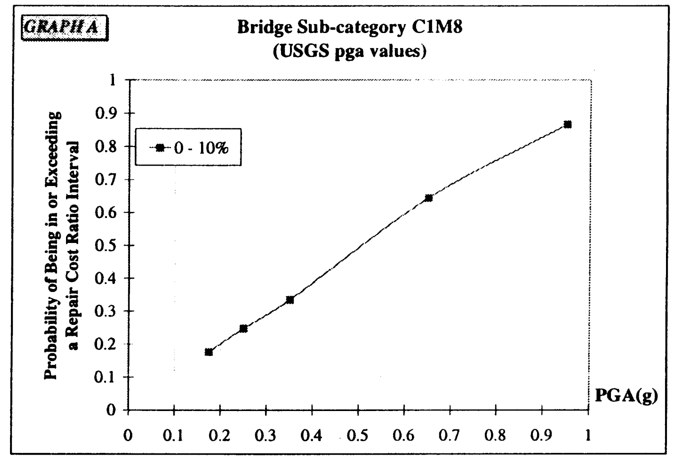
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.5425	0.9689	0.0001	93.923

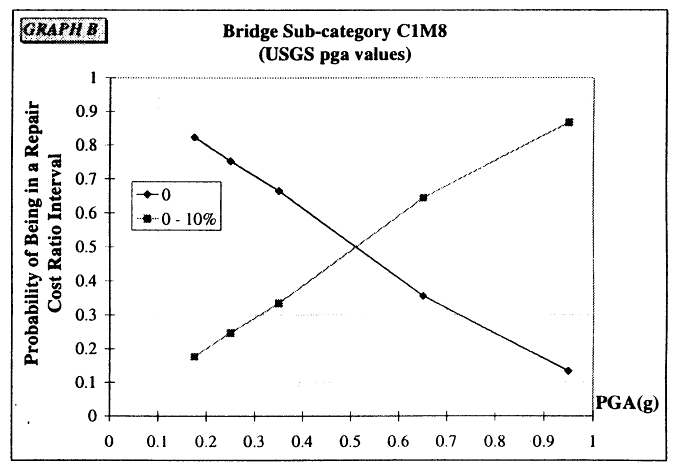




Number of bridges 20, 9
Chi-square N/A

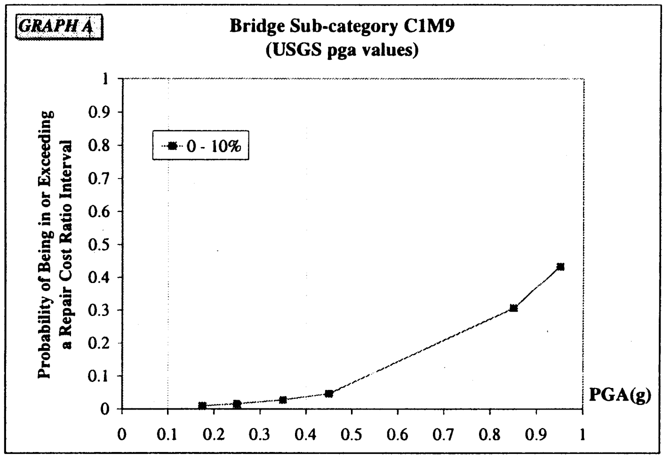
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.2668	2.5439	0.0935	71.295

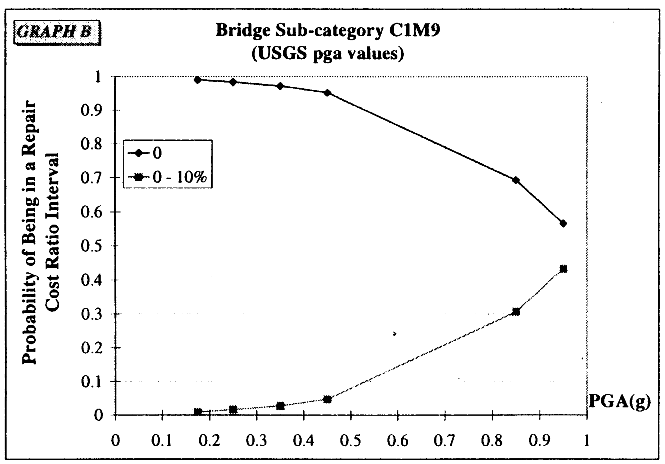




Number of bridges 43, 3
Chi-square N/A

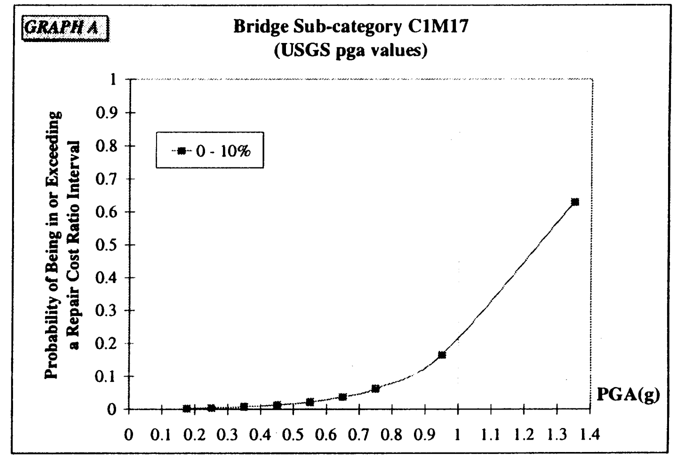
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
5.48	2.3089	0.0176	239.855

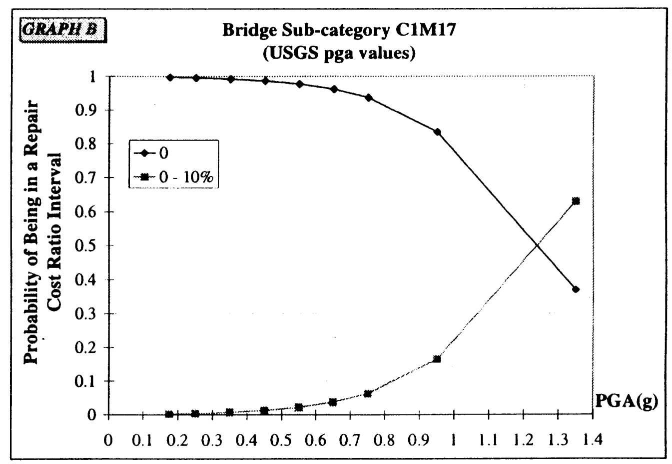




Number of bridges 106, 2
Chi-square N/A

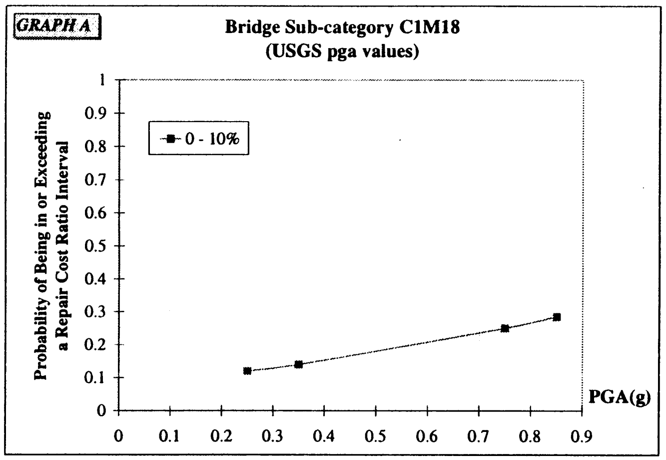
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
5.3866	2.2555	0.0169	218.466

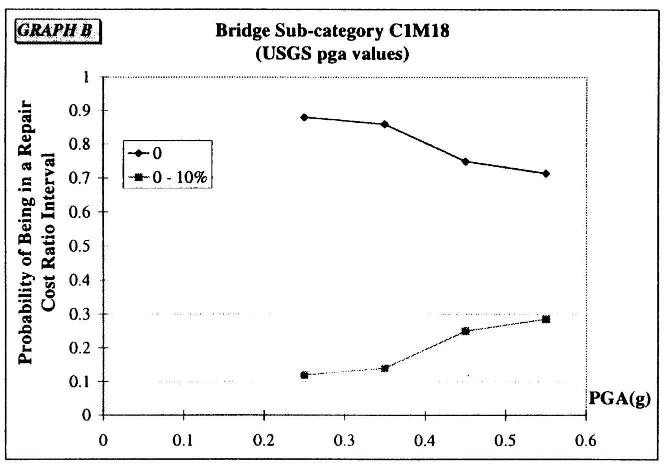




Number of bridges 12, 3
Chi-square N/A

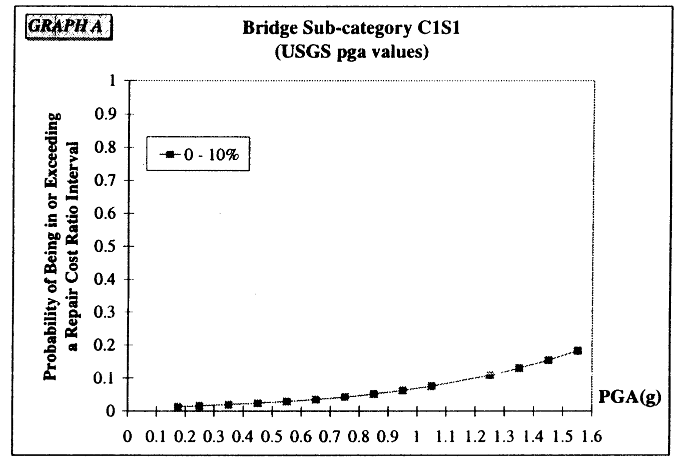
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
1.8013	2.5747	0.4842	6.058

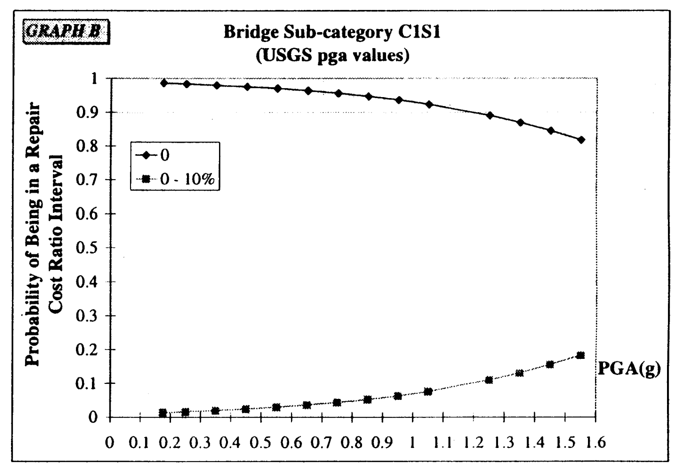




Number of bridges 199, 6
Chi-square N/A

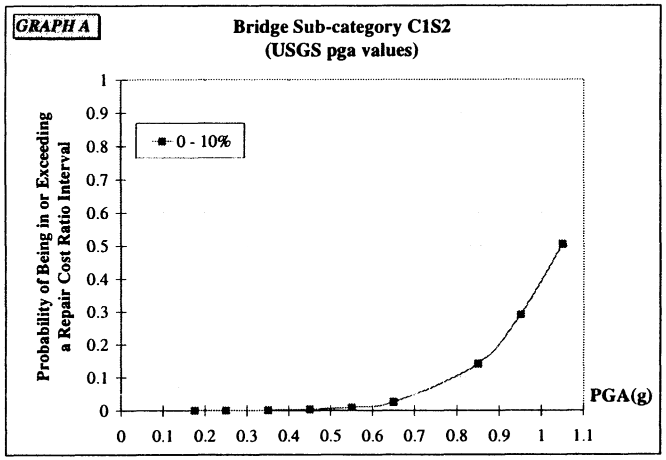
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
1.993	0.9765	0.0413	7.337

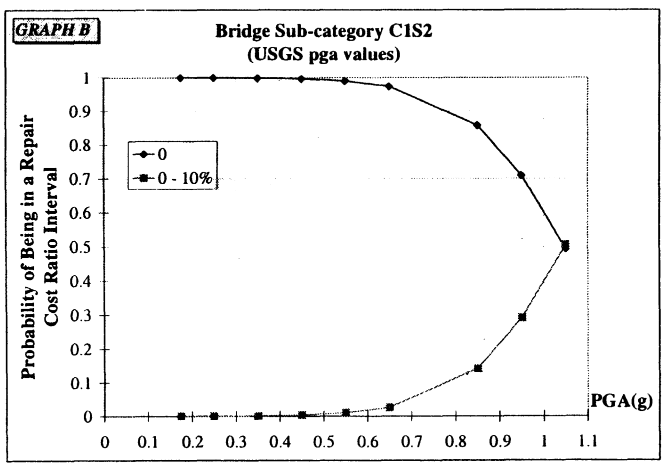




Number of bridges 90, 3
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
9.1284	4.883	0.0616	999





Validity of the model fit is questionable for C1M6.

Number of bridges 8, 1
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
66.1641	164.1	0.6869	999

Validity of the model fit is questionable for C1M10.

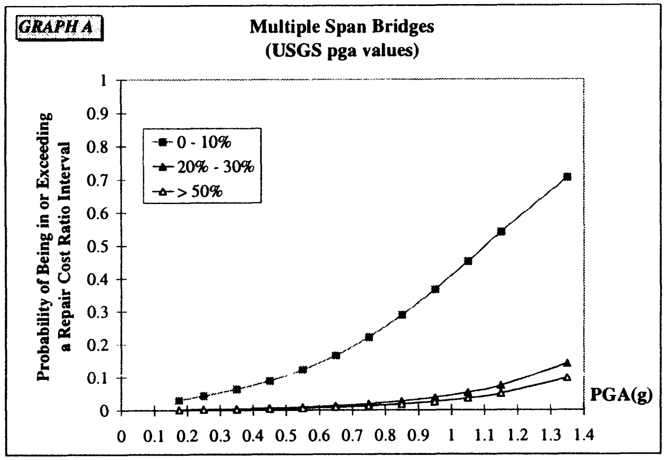
Number of bridges 11, 6, 0, 1
Chi-square with 1DF 0.0006; p = 0.9804

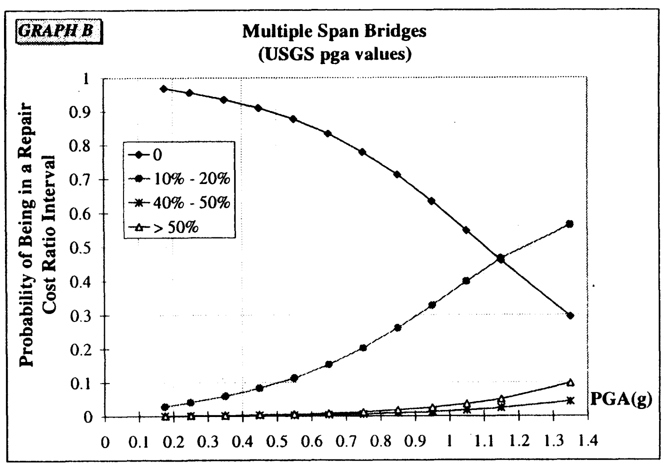
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
92.0092	406.3	0.8208	999

Northridge Earthquake *Number of Spans*

Number of bridges 777, 65, 0, 2, 0, 0, 4
Chi-square with 2 DF 22.306; p= 0.0001

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.6441	0.4301	0.0001	38.248

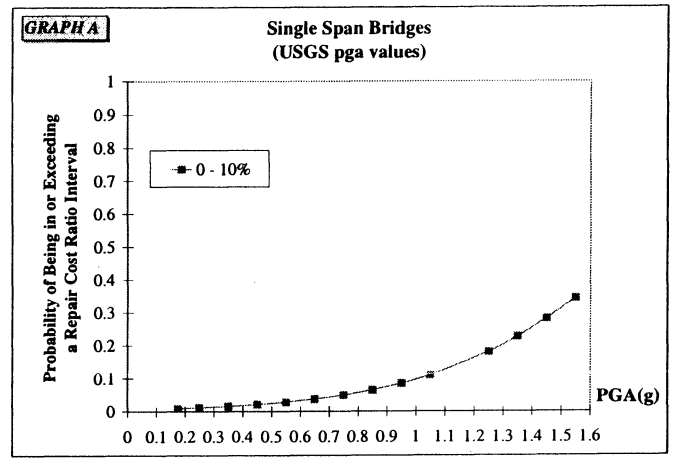


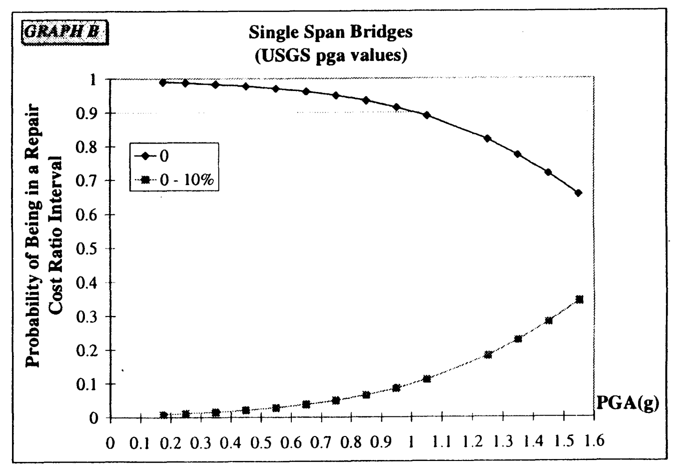


Northridge Earthquake *Number of Spans*

Number of bridges 315, 9
Chi-square N/A

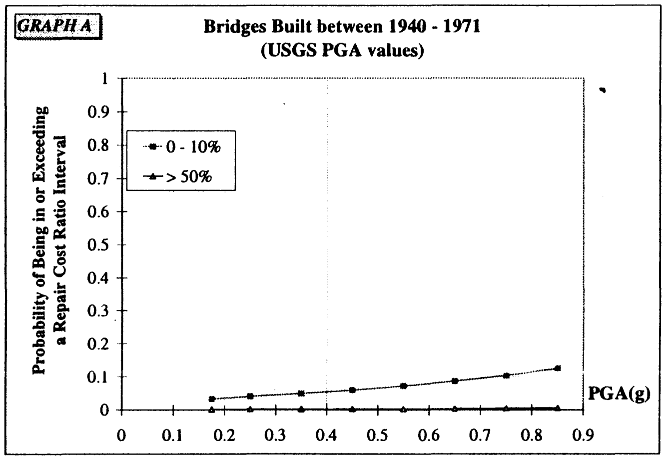
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.9131	0.8059	0.0003	18.414

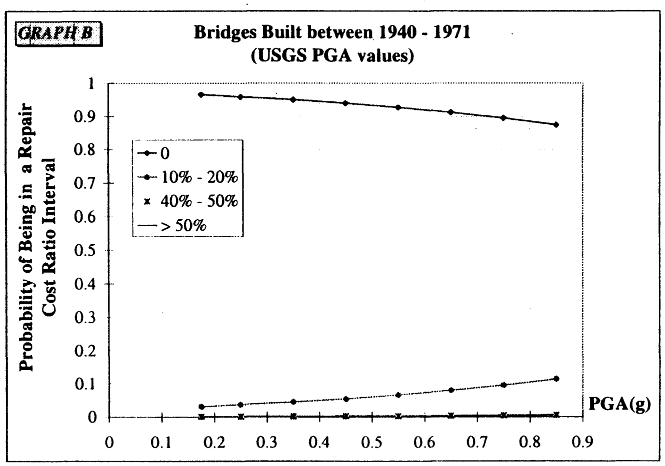




Number of bridges 743, 41, 0, 2, 0, 0, 2
Chi-square with 2 DF 346.4903; p = 0.0001

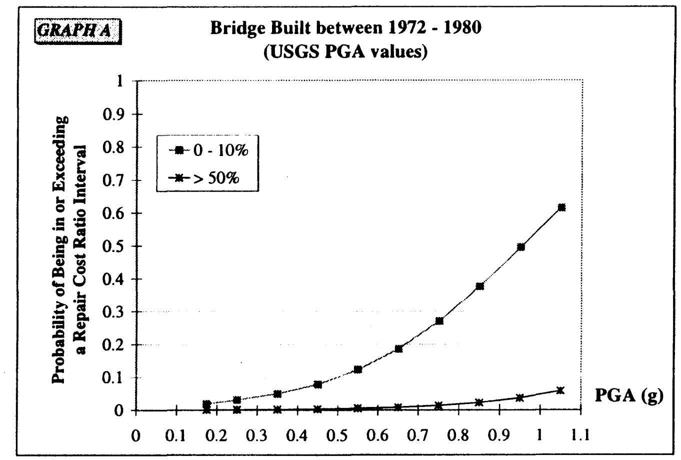
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
2.0205	0.4602	0.0001	7.5425

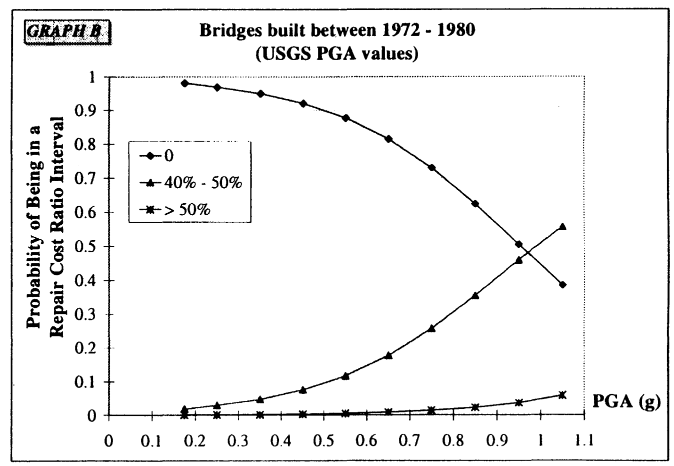




Number of bridges 217, 31, 0, 0, 0, 0, 2
Chi-square with 1 DF 7.767; p = 0.1837

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
4.8737	0.7628	0.0001	130.807





APPENDIX H

EMPIRICAL FRAGILITY CURVES

Event: *Northridge earthquake.*

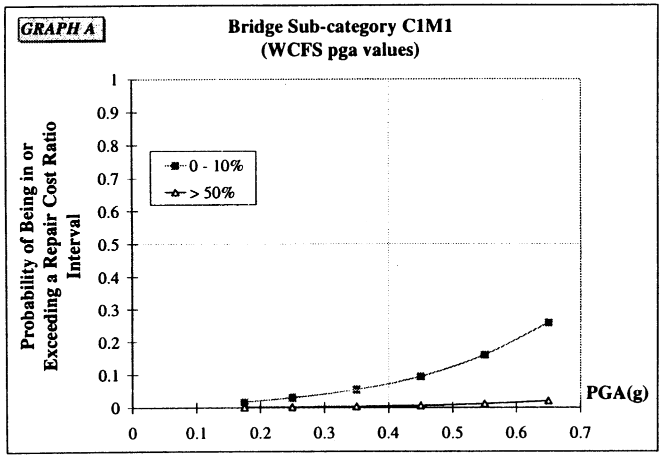
Dependent variable: *Repair cost ratio.*

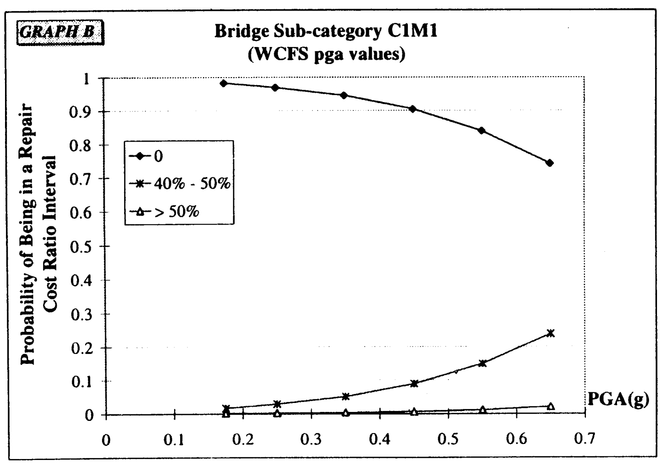
Independent variable: *Peak ground acceleration based on WCFS [1995].*

Grouped by	Data set
Bridge sub-categories	*correlation data set*
Number of spans	*correlation data set*

Number of bridges 234, 14, 0, 0, 0, 0, 1
Chi-square with 1DF 1.0888; p = 0.2967

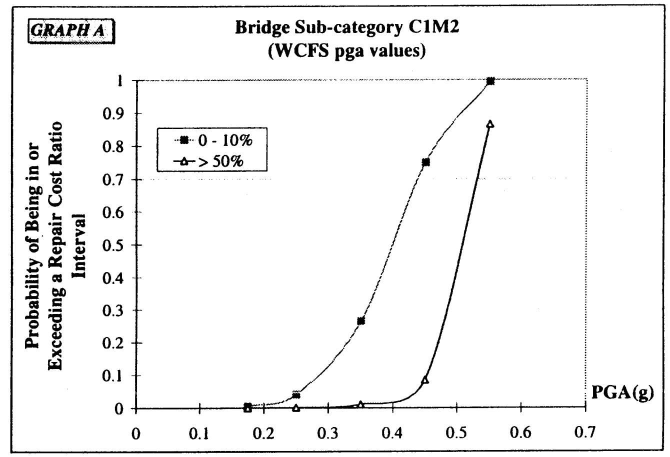
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
5.9901	1.8173	0.001	399.458

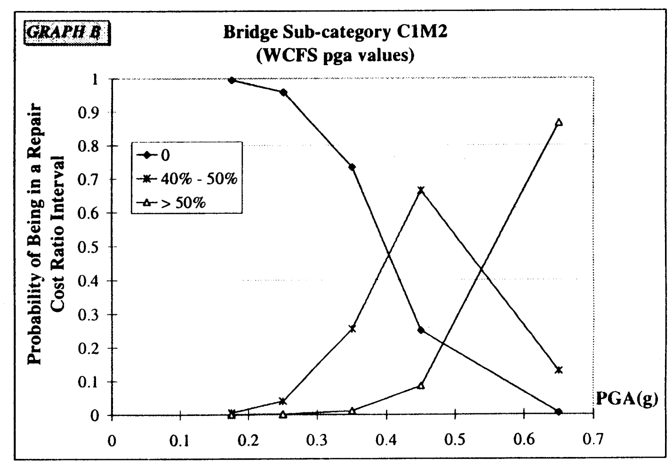




Number of bridges 21, 3, 0, 0, 0, 0, 2
Chi-square with 1DF 0.7453; p = 0.388

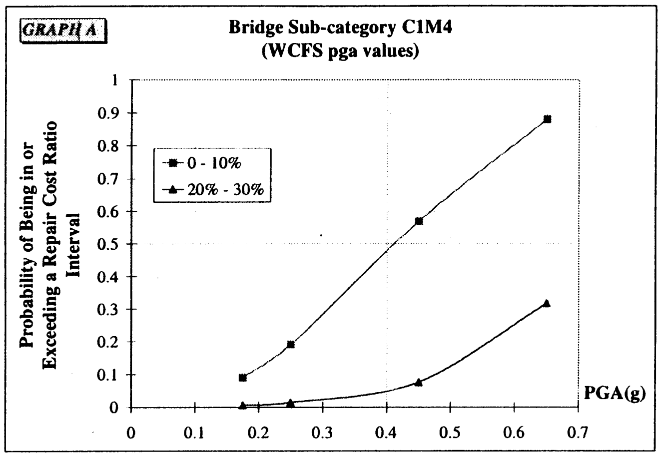
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
21.1736	7.1296	0.003	999

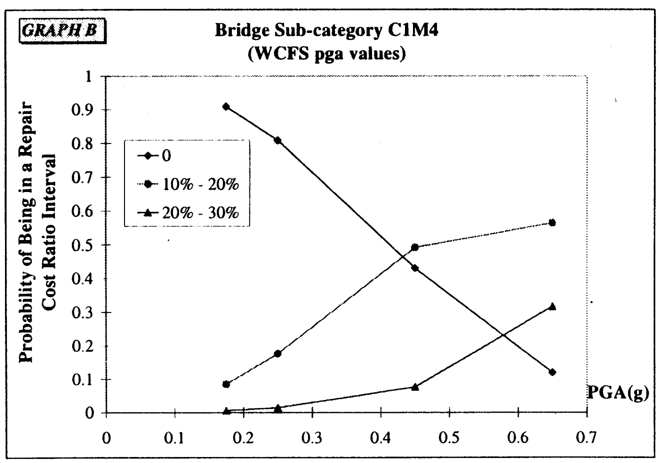




Number of bridges 12, 5, 0, 1
Chi-square with 1DF 1.5724; p = 0.2099

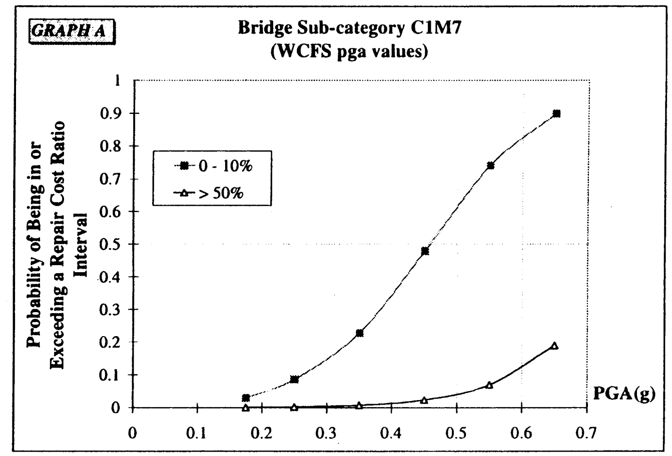
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
8.5994	3.8228	0.0245	999

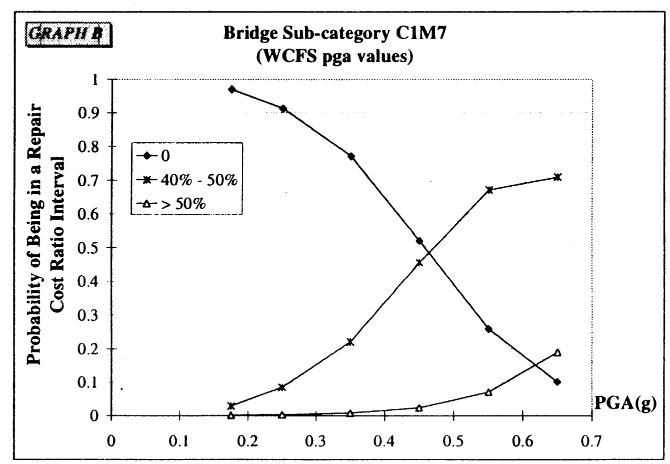




Number of bridges 145, 17, 0, 0, 0, 0, 1
Chi-square with 1DF 0.3317; p = 0.5647

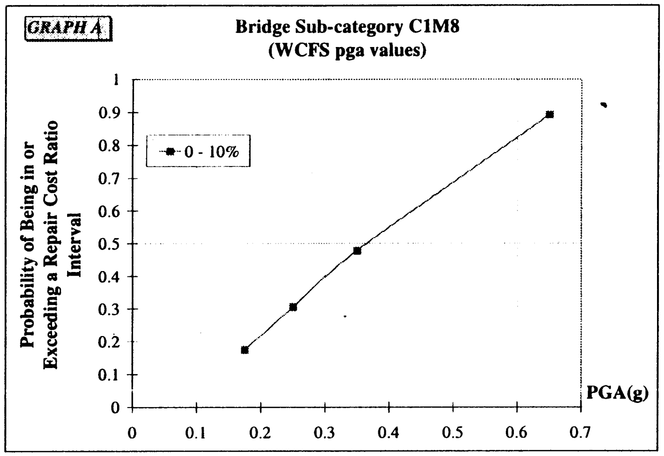
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
11.3501	2.412	0.0001	999

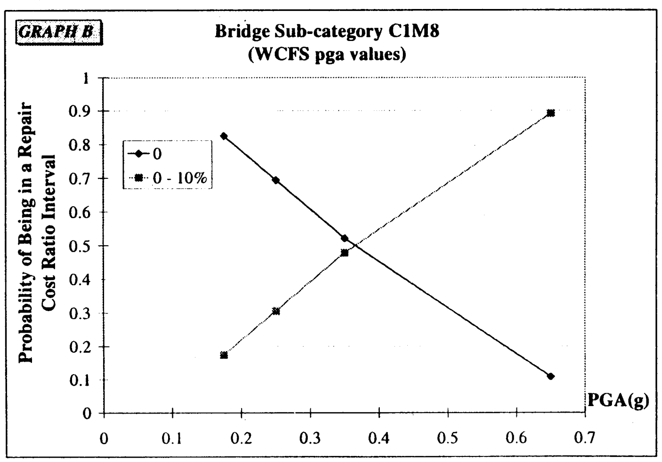




Number of bridges 17, 9
Chi-square N/A

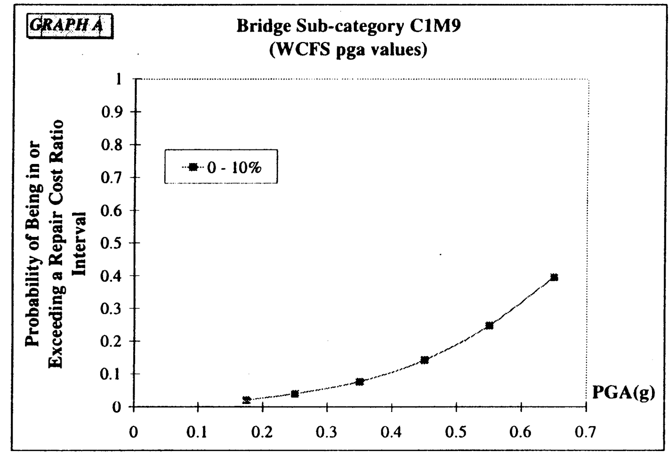
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
7.3293	3.6514	0.0447	999

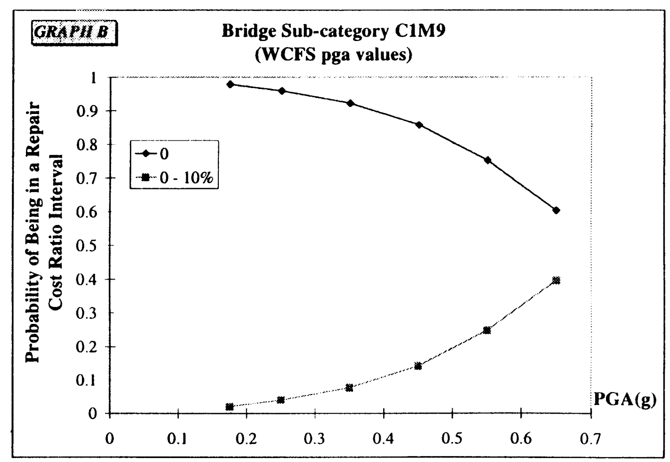




Number of bridges 40, 3
Chi-square N/A

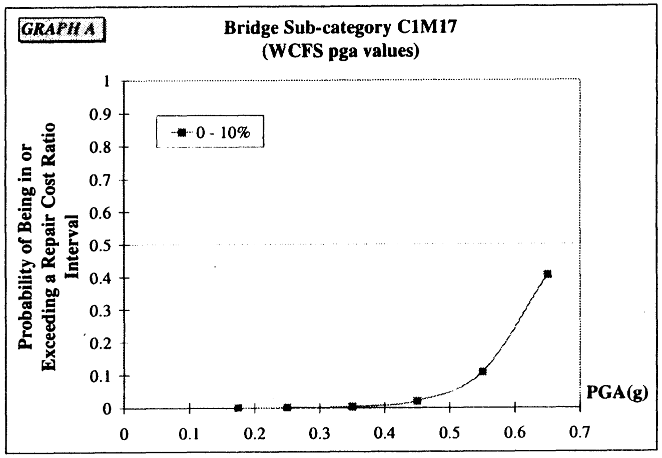
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.8797	3.4154	0.044	972.354

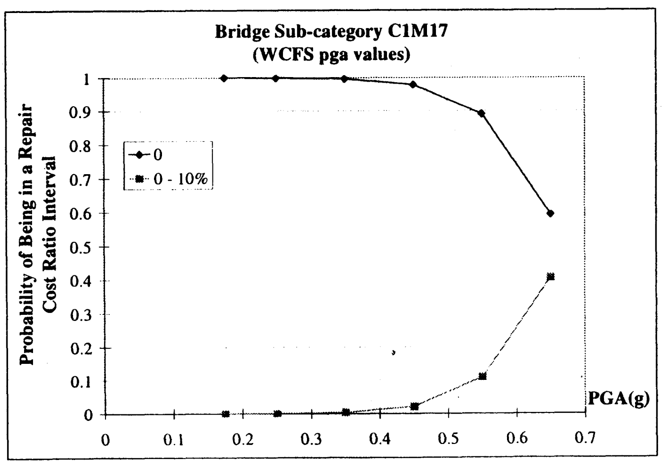




Number of bridges 77, 2
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
17.235	9.9249	0.0825	999

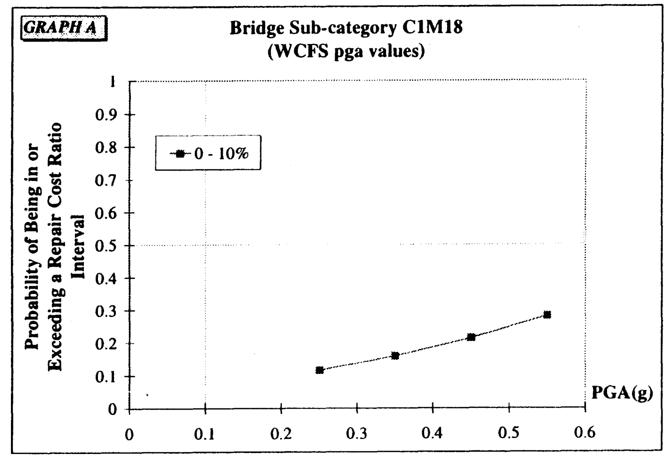


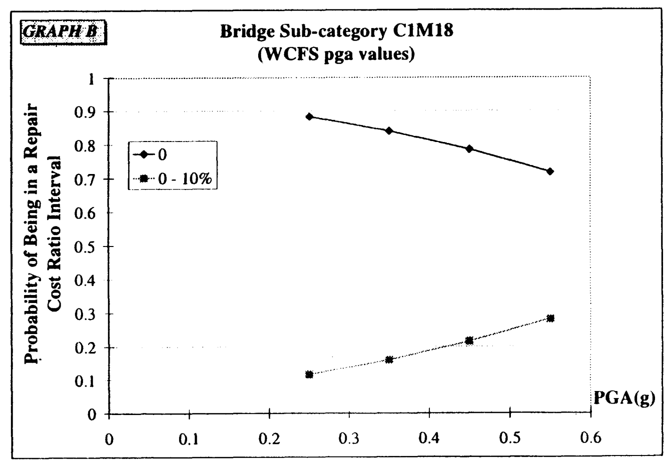


Northridge Earthquake *Bridge Sub-categories*

Number of bridges 12, 3
Chi-square N/A

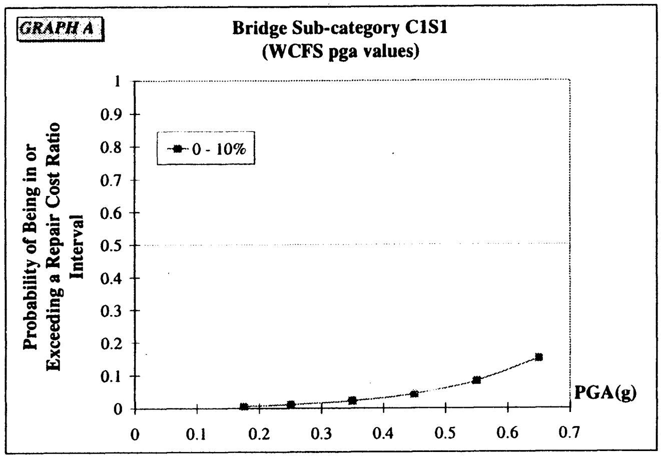
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
3.6401	5.7322	0.5254	38.097

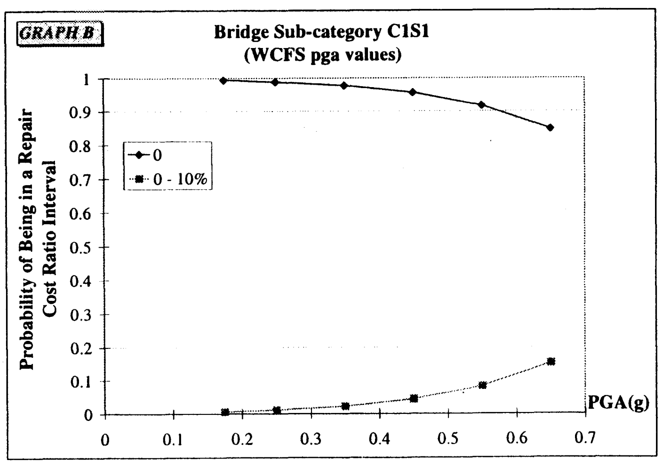




Number of bridges 186, 6
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
6.8493	2.8434	0.016	943.251

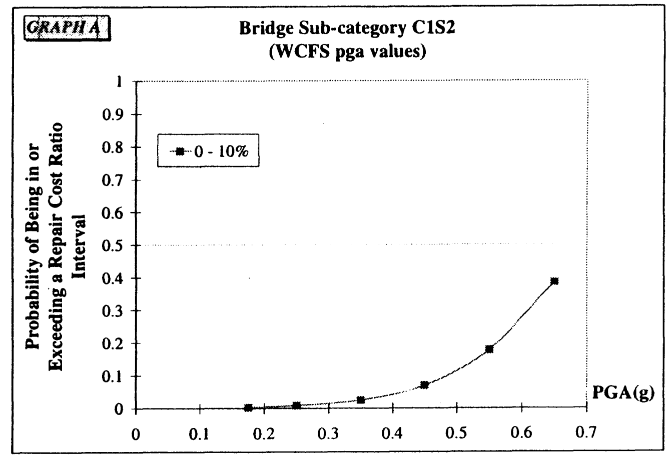


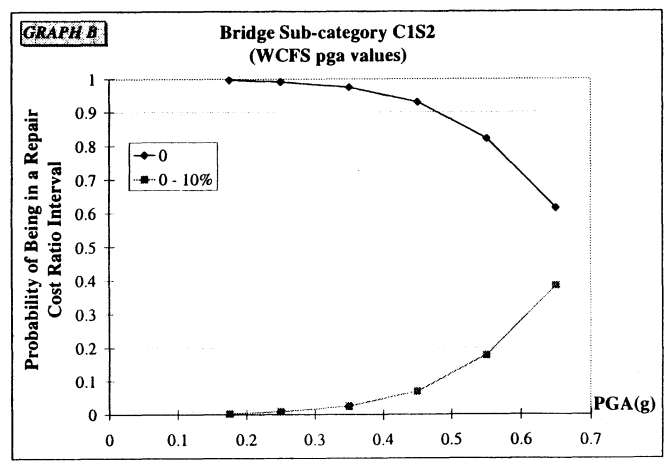


Northridge Earthquake *Bridge Sub-categories*

Number of bridges 73, 3
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
10.6387	4.3189	0.0138	999





Validity of the model fit is questionable for C1M3.

Number of bridges 36, 2
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
93.9342	548	0.8639	999

Validity of the model fit is questionable for C1M6.

Number of bridges 4, 1
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
144.3	444.7	0.7456	999

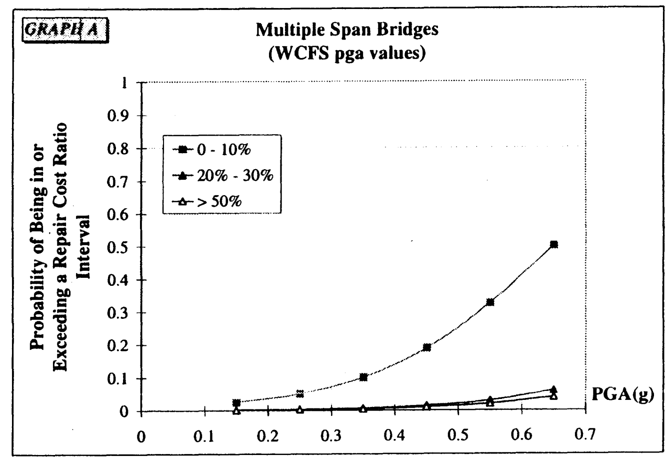
Validity of the model fit is questionable for C1M10.

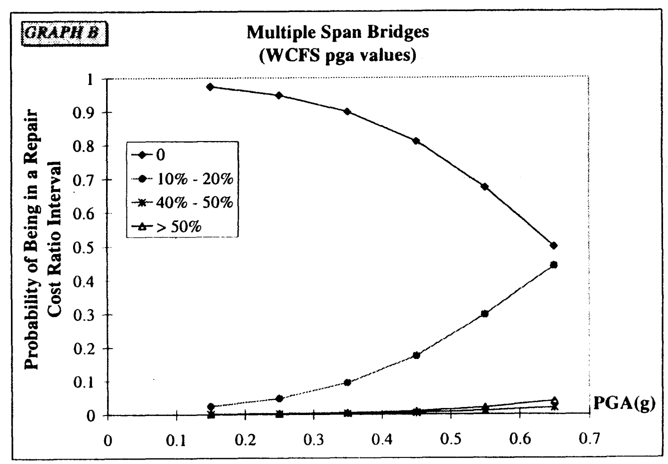
Number of bridges 11, 6, 0, 1
Chi-square with 1DF 0.0004; p = 0.985

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
91.7074	533.6	0.8635	999

Number of bridges 663, 65, 0, 2, 0, 0, 4
Chi-square with 2 DF 104.0838; p= 0.0001

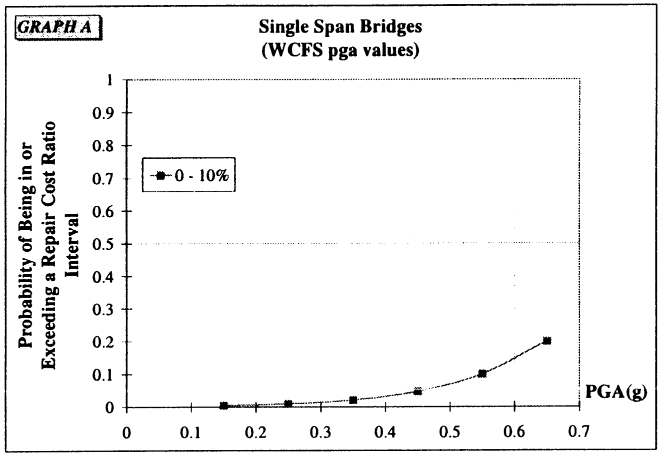
Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
7.3069	0.8356	0.0001	999

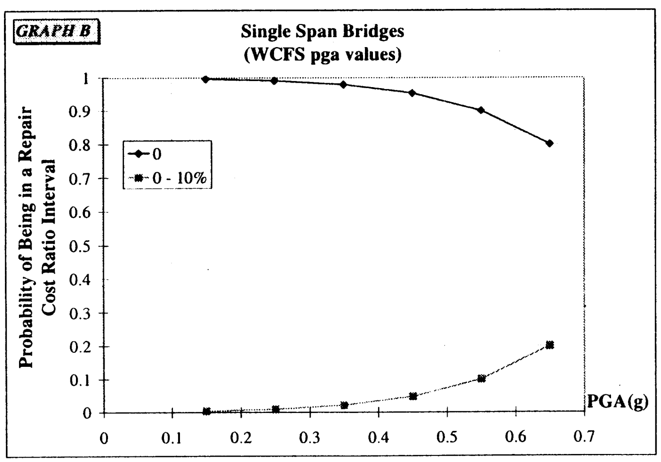




Number of bridges 270, 9
Chi-square N/A

Parameter Estimate	Standard Error	Prob. > Chi-square	Odds Ratio
8.086	2.3496	0.0006	999





MCEER
MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
A National Center of Excellence in Advanced Technology Applications

State University of New York at Buffalo
Red Jacket Quadrangle ■ Buffalo, New York 14261-0025
Phone: 716/645-3391 ■ Fax: 716/645-3399
E-mail: mceer@acsu.buffalo.edu ■ WWW Site: http://mceer.buffalo.edu

ISSN 1520-295X