Final Report

FHWA/IN/JTRP-2003/22

**EMERGENCY EARTHQUAKE ROUTES
FOR THE STATE OF INDIANA**

*PART I
CRITERIA FOR SELECTION OF PRIMARY ROUTES:
TRANSPORTATION ASPECTS*

by

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Emergency Earthquake Routes for the State of Indiana (A Two-Volume Report)

Introduction

The possibility of very strong earthquakes in the central Mississippi Valley has been known for 200 years in the New Madrid Seismic Zone. Awareness of the seismic risk in the State of Indiana has increased in the last decades. In the last two decades identification of an independent tectonic process occurring in the Wabash River Valley has implications for the seismic risk of the State of Indiana from what has recently been defined as the Wabash Valley Seismic Zone. The occurrence of a strong earthquake in these seismic zones or in Indiana or the neighboring states due to other not so well understood tectonic processes, as has occurred in the past, has serious implications for the State Transportation System. The definition of Earthquake Emergency Routes for the state is a priority for the Indiana Department of Transportation. The identification of Emergency Routes takes into account issues related to transportation including coverage of population and area and travel time along these routes.

This study formulates a multicommodity maximal covering network design problem (**MCNDP**) for identifying critical routes, for earthquake response and to seismically retrofit bridges. The **MCNDP** seeks routes that minimize the total travel time over the selected routes and maximizes the total population covered, subject to a budget constraint on bridge retrofitting costs on the selected routes. The problem is formulated as a two-objective

integer programming model and solved using the branch-and-cut module in the CPLEX optimizer. The model performance was analyzed using the transportation network of a seismically-prone region in southwest Indiana. Thereby, the search for the critical routes for an origin-destination (O-D) pair is confined to a limited geographical region around it.

A Geographical Information System (GIS) system was developed for evaluation of seismic vulnerability of the proposed Indiana Emergency Routes. The GIS based implementation of the technical information developed in this project through FEMA HAZUS-MH software was conducted in collaboration with the Polis Center in Indianapolis, nationally recognized and FEMA authorized HAZUS implementation facility. HAZUS software may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in southwestern Indiana. It uses information from the Indiana Department of Transportation (INDOT) maintenance database and selected structural and geotechnical information from the bridge drawings data base. Vulnerability assessment was performed using the general information from maintenance and final calibration was performed using a series of cases based on bridge drawings and soil exploration descriptions contained in these drawings.

Findings

Computational experiments were conducted to investigate the effects of varying the budget and the relative weights of the two objectives. A set of

critical routes was obtained. Seismic retrofit planning plays an important role in determining the effectiveness of the operational stage of earthquake

response management. At a strategic level, it is dependent on the routes critical for effective emergency response. However, past studies have not addressed the identification of critical routes under budget constraints. This is a very important practical problem for emergency response planners as it involves identifying effective transportation routes while accounting for bridge structural conditions and budget limitations. It implies the identification of bridges to retrofit under earthquake response planning. This study develops a network-level methodology to assist decision-makers in identifying an effective seismic retrofit scheme for the bridges of a network.

This study formally defines the Multicommodity Maximal Covering Network Design Problem (MCNDP) and formulates it. To our knowledge, it represents the first attempt to incorporate the coverage objective in a generic network model, leading to a new class of models with significant practical implications. Past work has used the coverage criterion in identifying a path, tree or tour on a network by considering either the routing or the fixed cost, while the proposed model simultaneously considers the routing and fixed costs subject to a budget constraint. That is, none of the existing models consider routing costs, fixed costs and coverage criterion in a single framework.

From a practical standpoint, the key contribution of this study is its ability to address planning problems faced by emergency response agencies vis-à-vis disaster management. The specific problem addressed here relates to earthquake response management. Under budget constraints, there is a need to determine an effective retrofit plan for the bridges in a seismically-prone region. Most studies in the literature have adopted a local perspective to solving this problem, and do not consider the effect of potential bridge failures on the transportation system performance. The few methodologies that use a systems approach do not consider the influence of retrofit decisions on the quality of the associated routes vis-à-vis effective earthquake response. The proposed model fills this critical gap by considering the total travel time for the key O-D pairs in the network and the total population covered by them as the criteria for determining the critical routes. The

budget limitation on the total bridge retrofit costs for the links constituting the critical routes is represented as a constraint in our model. Hence, this methodology simultaneously determines the set of bridges in a network requiring retrofit and the routes that serve as focal points for earthquake response.

The evolution of the earthquake resistant bridge design requirements contained in the AASHTO Specifications and the existence of a proposed draft seismic design specification being discussed have significant implications in the assessment of the existing vulnerability, its mitigation, and the design of new bridges along the Emergency Routes. Along with this development, the USGS assessment of the seismic risk of southwestern Indiana and a change of format of the definition of the earthquake design ground motion that is integral part of the proposed draft seismic design specification also has important implications on the assessment of the operational capabilities of the Earthquake Emergency Routes of Indiana. The effects of the 2001 Nisqually Earthquake that occurred in the State of Washington gave invaluable insight on the expected situation for the State of Indiana due to a comparable bridge inventory in age and bridge types and a parallel increase of the seismic risk awareness in both regions over the last decades.

Using the information collected in this study, the GIS implementation through FEMA-HAZUS-MH software may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in southwestern Indiana. Vulnerability was evaluated for different levels of ground acceleration in order to obtain behavior patterns for selected bridges. Using these results, seismic behavior patterns were obtained for bridges located in southwestern Indiana included in the INdot Vincennes District maintenance data base. Soil amplification and liquefaction potential were evaluated for the cases having reported SPT measurements from borings. Based on this process, information critical for the identification of upgrade needs for the transportation structures part of the Earthquake Emergency Routes is made available to the INdot.

Implementation

The findings and developments of this research project are presented next in the form of suggested implementation steps by the Indiana Department of Transportation.

1. Formal adoption of the selected emergency routes or some variation of the same by the state is a necessary step. Such routes adopted and maintained by INdot will be affected by issues and policies outside the scope of those considered in this study. For instance, policy decisions, budgetary constraints, new projects, i.e. continuation of I-69, will likely affect the formal adoption of a set of emergency routes and will continue to impose changes in the future. The methodology developed in this study and the information implemented in HAZUS-MH should be of assistance to state decision makers in the future.
2. Future implementation strategies can be studied through simulation studies using appropriately updated information for different earthquake scenarios including earthquake ground motion defined in current AASHTO Standard Specifications, proposed Draft AASHTO LRFD Seismic Provisions, or other. These simulations will provide corresponding bridge retrofit scope for the proposed routes or alternative definitions.
3. The information in the maintenance database should be periodically evaluated and should include information currently available only in the bridge drawings (see Section 7.5 of the final report). This will improve the vulnerability assessment of the bridges refining the mitigation policies to adopt in the future. This expansion of the maintenance database can be extended to other counties and districts not covered in this research project.
4. Establish a program for assessing the liquefaction, soil spread, and embankment failure potential along the routes at locations other than bridge sites by providing a consistent evaluation methodology supported with the information developed in this study. This implementation will require a soil exploration program to obtain geotechnical properties of sites where no information exists or where such information was obtained years ago with different objectives than liquefaction evaluation. In the research conducted in this study, liquefaction evaluations were made using solely boring information contained in selected bridge drawings.
5. To establish procedures for emergency response under different earthquake occurrence scenarios.
6. INdot should consider the formal adoption of HAZUS-MH software for scenario management, mitigation and vulnerability studies and to train appropriate personnel. An important feature in this implementation step is the engagement of the Polis Center in the training of INdot personnel on the use of HAZUS-MH software. The Polis Center is a nationally recognized HAZUS implementation facility.

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<p>The occurrence of a strong earthquake in Indiana or neighboring states, as has occurred in the past, has serious implications for the State Transportation System with respect to adequate response. The definition of Earthquake Emergency Routes for the State of Indiana became a priority for the Indiana Department of Transportation. These Emergency Routes take into account various aspects related to transportation including coverage of population and area and travel time along these routes, and issues related to structural and geotechnical seismic vulnerability of the roads and bridges along these routes.</p> <p>Part I of the overall study report formulates a multi-commodity maximal covering network design problem (MCNDP) for identifying critical routes, for earthquake response and to seismically retrofit bridges. In the MCNDP, routes are sought that minimize the total travel time over the selected routes and maximize the total population covered, subject to a budget constraint on bridge retrofitting costs on the selected routes. The problem is formulated as a two-objective integer programming model and solved using the branch-and-cut module in the CPLEX optimizer. The model performance is analyzed using the transportation network in southwest Indiana. A problem reduction strategy is introduced to reduce computational times by recognizing that the critical routes are not usually circuitous. Thereby, the search for the critical routes for an origin-destination (O-D) pair is confined to a limited geographical region around it. To further reduce computational costs, the formulation is extended to incorporate valid inequalities that exploit the problem structure. Simulation studies are conducted to investigate the effects of varying the budget and the relative weights of the two objectives. Noninferior frontiers that illustrate the trade-offs between the conflicting objectives for different budgets are constructed to provide practical insights to decision-makers. The final selected routes in addition include constraints set by the Study Advisory Committee regarding specific roads to be included in the final set.</p> <p>Part II of the study report covers the structural and geotechnical seismic issues related to the definition of Earthquake Emergency Routes for the State of Indiana. Previous worldwide experience on bridge damage during strong earthquakes has influenced the bridge earthquake design specifications and mitigation procedures. The evolution of the relevant requirements in the AASHTO Specifications and the existence of a proposed draft seismic design specification under development as part of the NCHRP Project 12-49 have significant implications in the assessment of the existing vulnerability, its mitigation, and the design of new bridges along the Emergency Routes. Geographical Information System (GIS) was implemented to assist the evaluation of seismic vulnerability of the proposed Indiana Emergency Routes. Using the available information, the GIS implementation may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in Indiana incorporating information from the Indiana Department of Transportation (Indot) maintenance database and selected structural and geotechnical information from the relevant bridge drawings. Findings from a vulnerability assessment as well as final calibration performed using selected representative bridge data such as drawings and soil exploration descriptions are also reported. Vulnerability was evaluated for different levels of ground acceleration in order to obtain behavior patterns for selected bridges and to illustrate upgrade needs for the transportation structures on the Emergency Routes. The information contained in this report could be used to assist the engineers in conducting simulations that will help Indot to devise mitigation policies for different earthquake occurrence scenarios and to evaluate potential alternative routes.</p>			
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CHAPTER 1 INTRODUCTION

1.1 Background and Motivation

Earthquakes can significantly disrupt societal functioning, cause ecological damage, and lead to loss of human lives and property, warranting a coordinated and efficient response to mitigate their negative impacts. Past experience with earthquakes has demonstrated the vulnerability of the critical infrastructural lifelines and the need for mitigation strategies as well as emergency response planning. For example, the bridge failures under the 1989 Loma Prieta and the 1994 Northridge earthquakes resulted in substantial losses to the regional economies [Chang and Nojima, 2001].

Highway transportation systems are vital to the normal functioning of a society and can be even more so under disasters. Under disasters, emergency response agencies need to transport various commodities, including food, clothing, medicine, medical supplies, machinery and personnel from different points of origin to different locations in the disaster areas in an efficient manner [Haghani and Sei-Chang, 1996]. Damage to the highway system can seriously affect emergency response, recovery operations and disrupt the regional economy; hence the functionality of these systems is critical for post-disaster response. Gordon et al. [1998] estimate that twenty percent of the \$6.5 billion in losses due to the Northridge earthquake resulted from damage to the transportation system. This highlights the need for countermeasures to mitigate the possible risk of damage to the highways and the associated consequences.

Bridges typically represent the elements of a highway system that are most susceptible to failure under earthquakes. Hence, the seismic vulnerability of bridges has been extensively studied [Buckle et al. 1994]. As a consequence, seismic retrofitting of bridges has been the most preferred solution adopted in the pre-disaster strategic planning stage so as to minimize the risk of unacceptable damage to highways in earthquake-prone regions. Due to the significant effort and cost involved in strengthening existing bridges to seismic design standards it is not practical to retrofit every bridge in an earthquake-

prone region, and even if it were possible, retrofitting cannot be done simultaneously across all bridges. Also, seismic risk can vary across bridges due to their physical characteristics, age, and current structural condition. Hence, a key problem faced by earthquake response planners is the identification of the set of bridges for seismic retrofitting. This implies prioritization given budget limitations and the effort involved.

The bridge prioritization problem, that is, the selection of bridges for seismic retrofitting, can be viewed from structural and transportation domains vis-à-vis performance. From a structural perspective, the risk/vulnerability of a bridge to seismic failure is the predominant factor that influences prioritization. In the transportation context, the focus is on ensuring the survival of links (and the bridges on those links) that minimize emergency response times and maximize the population that can be reached, by retaining a network-level perspective. The existing literature predominantly addresses the prioritization problem from a structural perspective. The dominant approach is the use of multi-attribute utility theory [Nojima, 1999] to determine the risk for each bridge in the highway network. This is done by weighing the values of the various attributes to generate a measure of risk. For example, one study [Osaka Municipal Government Report, 1998] uses attributes on seismic capacity (or vulnerability) and importance (or criticality). Seismic capacity, an indicator of the structural sufficiency of a bridge with regard to an anticipated earthquake, is based on the bridge type, structural condition, service life, and geotechnical and seismic hazards. The importance criterion indicates the criticality of the bridge to post-disaster relief operations based on the traffic flow on the associated link, economic impacts due to its failure, and the connectivity with crossroads.

While the seismic capacity criterion has been well-studied in the literature, the importance criterion has not been adequately addressed [Chang and Nojima, 2001]. This is because bridge importance is not readily quantifiable, implying subjectivity in its determination. The issue of accessibility across the transportation network vis-à-vis earthquake response can be influenced by political, social and economic factors in addition to the engineering solutions. Hence, only a few retrofitting schemes exist in practice, and are primarily based on engineering judgment [Buckle, 1994], indicating methodological gaps in the state of knowledge to address this problem. The intuitive

solution to the earthquake response planning problem is the retrofitting of every bridge in the shortest path between the selected origin-destination (O-D) pairs in the network. However, this is typically cost prohibitive. This implies the need for a network level perspective to address the problem which considers the interactions between the failure of a bridge and its influence on the network-level system performance [Werner et al., 1997, Chang and Nojima, 2001]. The Multidisciplinary Center for Earthquake Engineering Research (MCEER) Bulletin (1994) suggests that the system performance considerations be addressed before performing the seismic capacity evaluation of bridges.

Few studies consider network-level transportation system performance in determining prioritization schemes. Basoz and Kiremidjian [1994] develop a network-level procedure to quantify the importance of bridges. It determines the bridges that constitute minimal cuts in a transportation network and ranks the individual bridges within the set. The minimal cut for a given O-D node pair in a network is the minimum number of links whose failure disconnects the two nodes. Performance was tied to the connectivity between critical destinations under disasters. Wakabayashi [1997] performs an importance analysis of the Kobe highway network based on several network realizations. The travel time between Osaka and Kobe represented the performance criterion. The structural vulnerability of the links was not considered.

Nojima [1999] proposed a performance based prioritization method for upgrading network components with limited resources. The performance measure was the reliability of system flow capacity, defined as the maximum flow between a specified O-D pair in the network. The study rank-orders links based on their ability to improve the reliability measure. Chang and Nojima [2001] develop performance measures based on the total length of the surviving network and the accessibility provided by it.

The overview of the past work indicates that several studies exist that address the bridge seismic retrofit aspects of the earthquake response planning problem. Hence, the problem has been addressed primarily in the structural engineering domain. Different criteria have been used to evaluate the bridges in the network. Some of them are ad-hoc in nature and involve considerable subjectivity. Only a few studies exist that address the seismic retrofit problem from a transportation systems perspective. Among those, there

are very few studies that analyze the problem at a network-level. Those that do are primarily concerned with the effect of retrofit decisions on vehicular flow performance measures for the surviving network. Studies addressed from the perspective of earthquake response are sparse, and typically consider a single criterion such as connectivity between O-D node pairs, travel time for a single O-D pair, or accessibility to population. None of these studies explicitly consider budget constraints in the planning problem. This significantly increases the problem complexity due to the dependence of the surviving network under an earthquake on the budget investment decisions. This key gap, significant from a practical standpoint, motivates the development of a new class of models that can aid planners in developing an effective budget allocation scheme to prioritize retrofit decisions. The proposed research addresses this by developing the **MCNDP** model.

1.2 Study Objectives

The primary study objective is to develop a methodology to address the strategic planning problem at a network-level for earthquake response. This methodology enables the decision-makers to identify an effective seismic retrofit scheme vis-à-vis earthquake response for the bridges of a network. The proposed methodology is motivated by the need to consider the key factors for effective earthquake response in a single framework. This is done by first defining the concept of “critical routes”. The critical routes of a transportation network are the set of routes whose functionality is critical to the effectiveness of earthquake response. The associated problem is labeled the critical routes problem. The specific tasks to address the study objective are:

1. Identification of criteria for selection of critical routes. These criteria are based on factors that directly affect the effectiveness of earthquake response.
2. Formally define the critical routes problem. This involves the description of the problem context, its constraints and the criteria used to solve the problem.
3. Development of a model to address the critical routes problem. A mathematical formulation is developed that seeks to optimize the response criteria under budget

limitations and other constraints. This generates the critical routes for the study region of interest.

4. Application of the model to a case study. Sensitivity analyses are conducted with respect to the model parameters to derive insights that assist decision-makers. This is done primarily by identifying a non-inferior frontier, which provides an intuitive practical tool for decision-making. The region of interest in this study is the earthquake-prone southwest region of Indiana. Figure 1.1 depicts the seismic map of the region.

1.3 Organization of the Report

This report is organized as follows. Chapter 2 provides an overview of the relevant literature in network design and coverage. Chapter 3 defines the critical routes problem and formulates it. It further proves that the problem is NP-hard, and identifies valid inequalities to enhance solution efficiency. Chapter 4 discusses the application of the formulation to a case study in the southwest Indiana region, and provides the implementation details. Chapter 5 reports the results from experiments conducted and derives insights. Chapter 6 reports the results from experiments conducted after incorporating the recommendations of the Study Advisory Committee in terms of “required” routes as part of the critical routes subnetwork. Chapter 7 provides some concluding remarks.

Figure 1.1 Seismic map of southwest Indiana

CHAPTER 2 LITERATURE REVIEW

This chapter provides a brief review of the methodological approaches relevant to the problem addressed in this research. As the focus is on network-level transportation-related planning for disaster response, especially under earthquakes, most of the relevant literature is in the urban operations research domain as applied to network models. For the sake of exposition, we first define the two types of link costs that are typically encountered in network design problems, namely fixed costs and routing costs. The fixed cost of a link is a one-time cost incurred with choosing that link for routing commodity flows and is independent of the quantity routed through it. The routing cost of a link is defined as the cost incurred in transporting a unit commodity across that link. For example, in an earthquake response planning context, bridge seismic retrofit costs would represent fixed costs for that link, and the time to traverse that link would correspond to the routing costs.

2.1 Network Design

A broad range of practical network optimization problems occur in the context of transportation, distribution planning, emergency response, telecommunications and computer networking. A specific class of these, labeled as network design problems, arises primarily in the capital investment phase of engineering problems. The network design problem generally seeks to determine a network configuration that minimizes the sum of the fixed costs of the links chosen and the cost of routing different commodities through the network defined by these links. Due to its usefulness in the aforementioned applications, the fixed charge network design problem has been well studied. Magnanti and Wong [1984] provide a comprehensive survey of integer programming-based methods to address network design problems. They develop a generic discrete choice network design formulation that unifies the different types of network design models. Well-known problems in combinatorial optimization, including the shortest path,

minimum spanning tree, Steiner tree and traveling salesman problem have been shown to be special cases of the generic network design model. However, these models have focused primarily on cost minimization and do not consider other important criteria like population coverage, environmental impacts and revenues.

2.2 Network Coverage

Current and Min [1986] emphasize the potential for multiple objectives in transportation planning problems and highlight the conflicting nature of such objectives in the context of public sector decision-making. For example, the shortest path network across O-D pairs may not maximize the accessibility to the population in the region. To overcome these limitations, Current et al. [1985] introduce the notion of coverage to the network design problem by formulating the maximum covering shortest path problem for a single O-D pair with two objectives: (i) to minimize the path cost for that O-D pair, and (ii) to maximize the total population covered by that path. They associate some fraction of the population with each node, which is labeled as the demand of that node. They define a demand node as covered if the path includes the node or passes through another node that is within a pre-specified distance from that node. They also formulate a variation of this problem for hazmat routing applications, called the minimum covering shortest path problem [Current et al., 1988], in which the demand covered is minimized.

Hutson and Revelle [1989] extend the concept of coverage to tree networks by considering two types of coverages: direct and indirect. A demand node is directly covered if there is a link in the tree incident upon it. Indirect coverage is assumed if the demand node is within a prescribed distance from a link in the tree. The maximal direct covering tree problem seeks to minimize the cost of the subtree and maximize the total demand covered by it. Hutson and Revelle [1993] extend the approach to develop the maximal indirect covering tree problem. They suggest the use of these models for problems involving road network construction in sparsely populated areas under resource constraints.

Kim et al. [1989] introduce the subtree r -cover problem which seeks a connected subgraph covering every demand node with minimum total length for a given network. A

cover radius is associated with each demand node. It is used to ensure that the distance between the demand node and the closest point in the subgraph does not exceed a threshold value. The formulation is applied to position troops in a war situation to ensure that troops protect/cover the various demand nodes within pre-specified threshold response times. Kim et al. [1990] consider a special case in which the given network is a tree.

Current and Schilling [1989] adapt the coverage concept to the traveling salesman problem by formulating the covering salesman problem. The problem aims to identify a minimum cost subtour in a network such that every demand node is within a specified distance from a node on the subtour. They extend this model to develop the maximal covering tour problem [1994], in which the objectives are to minimize the total tour length and maximize the demand covered by the tour, subject to a requirement on the number of nodes in the subtour. Suggested application domains include rural health care and overnight delivery systems. Gendreau et al. [1997] solve a variant of the maximal covering tour problem in which a given set of nodes must always be present in a tour, and propose its application to the location of post boxes and the design of collection routes. They also propose classes of valid inequalities for use in a branch-and-cut algorithm to solve large problem instances in a reasonable amount of time.

2.3 Discussion

The overview of the literature indicates that the coverage criterion has been successfully applied to identify a path, tree or tour, all of which are special network structures. This research extends the coverage criterion to a network problem with multiple O-D pairs, labeled the multicommodity maximal covering network design problem (**MCNDP**). Its objectives are to minimize the routing costs over all O-D pairs and maximize the total demand covered, subject to a budget constraint based on the fixed costs incurred on the chosen links. To our knowledge, there exist no prior studies that consider budgetary limitations, routing costs and coverage criteria in a single model with multiple O-D pairs. Such a model is essential for strategic planners to make long-term budget allocation decisions that factor in system-wide impacts of budgetary decisions

over time vis-à-vis performance. Potential applications of the **MCNDP** other than the problem addressed in this study include the design of regional transit systems and the planning of truck shipment routes. With modifications, the **MCNDP** can be used to design electric power networks to build high voltage lines for new power plants or to connect to existing power plants.

CHAPTER 3 METHODOLOGY

This chapter develops the methodology to address the critical routes problem using the **MCNDP**. Section 3.1 illustrates the applicability of the **MCNDP** to the “critical routes problem”. Section 3.2 states the **MCNDP**. Section 3.3 presents the mathematical model. Notation and parameters are introduced in Section 3.3.1, the decision variables are defined in Section 3.3.2, and the formulation is illustrated in Section 3.3.3. Section 3.4 examines the computational complexity of the **MCNDP**. Section 3.5 presents valid inequalities for the formulation.

3.1 The Critical Routes Problem

Network-level disaster management planning is vital for effectively responding to natural calamities and security-related problems. For example, the availability of the transportation network is critical to emergency response [Haghani and Oh, 1996] under earthquakes. It entails the identification of “critical routes” in a planning context that remain functional following an earthquake, to enable the response operators to access as much population as possible in a minimum amount of time. This implies two objectives for the selection of the critical routes: (i) minimizing the total travel time on the routes between the O-D pairs, and (ii) maximizing the total population that can be covered by these routes. The functionality or survivability of a route, implying its availability under an earthquake, is governed significantly by its weakest elements, the bridges. The seismic retrofitting [Cooper et al., 2001] of bridges can enhance the survivability of the associated routes under earthquakes. However, as discussed earlier, due to the significant cost and effort involved in retrofitting, it is impractical to retrofit every bridge in an earthquake-prone region, especially with limited budgets. Hence, the budget serves as a constraint in the determination of the critical routes. Therefore, the critical routes problem

can be viewed as a budget-constrained covering network design problem which seeks low cost routes that cover the maximum population while satisfying budget constraints introduced by the need to retrofit bridges on the critical routes.

3.2 Problem Statement

We formulate the **MCNDP** generically here, and apply it to the critical routes problem in the next chapter. We are given the locations of demand centers of a region and its associated undirected network. The network links have a fixed cost for their usage and a routing cost. There is a budget constraint on the total fixed cost incurred. The **MCNDP** seeks to allocate a limited budget to links such that the total routing costs for a set of O-D pairs is minimized and the total demand covered by the routes connecting them is maximized. The demand of a center is covered if a link in one of the selected routes provides access to it.

3.3 Mathematical Model

This section first introduces the notation, parameters and decision variables, and then proposes an integer programming formulation for the **MCNDP**.

3.3.1 Notation and Parameters

Let $G(N, E)$ denote an undirected network with node set N and link set E . The indices i and j denote a node in the network, $i, j \in N$ and $E \subseteq N \times N$, where $[i, j]$ denotes an undirected link between nodes i and j with a nonnegative fixed cost f_{ij} . Let B to denote the available budget. Each O-D pair in the network is represented as a unique commodity type. Let k represent the commodity type index, $k \in K$, where K denotes the set of all commodities. One unit of flow of commodity k must be transported over the network from its origin $O(k)$ to its destination $D(k)$. To differentiate the direction of flow of a commodity, we consider two directed links (i, j) and (j, i) corresponding to each original undirected link $[i, j]$. Let A denote the set of the directed links; all links are uncapacitated. Let c_{ij}^k be the nonnegative routing cost for a unit of commodity k on link (i, j) , and m

demand the demand center index, $m \in M$, the set of demand centers in the region. Let r_m represent the demand associated with demand center m , and E_m the set of links that are eligible to cover it. We treat the demand centers separately from the nodes in the network, whereas past studies assign demand centers to a node in the network which is a more restrictive approach.

3.3.2 Decision Variables

The formulation contains three types of variables: (i) the arc flow variables denoted by the vector $x = \{ x_{ij}^k \}$, which define the flow of different commodities in each of the selected links, (ii) the design variables denoted by the vector $y = \{ y_{ij} \}$, which define the links selected for the network design, and (iii) the coverage variables denoted by the vector $z = \{ z_m \}$, which define whether or not a demand center is covered. They are defined as follows:

$$x_{ij}^k = \begin{cases} 1, & \text{if there is a unit flow of commodity } k \text{ on link } (i, j) \\ 0, & \text{otherwise} \end{cases}$$

$$y_{ij} = \begin{cases} 1, & \text{if link } [i, j] \text{ is used in a flow path} \\ 0, & \text{otherwise} \end{cases}$$

$$z_m = \begin{cases} 1, & \text{if demand center } m \text{ is accessible from a link of a flow path} \\ 0, & \text{otherwise} \end{cases}$$

3.3.3 The MCNDP Formulation

The **MCNDP** formulation has two objectives: Z_1 , the total routing cost, and Z_2 , the total demand covered where,

$$Z_1 = \sum_{k \in K} \sum_{[i,j] \in E} (c_{ij}^k x_{ij}^k + c_{ji}^k x_{ji}^k) \quad \text{and} \quad Z_2 = - \sum_{m \in M} r_m z_m.$$

The integer programming formulation for the **MCNDP** is expressed as follows:

MCNDP:

$$\text{Minimize } Z = [Z_1, Z_2] \tag{1}$$

subject to

$$\sum_{(i,j) \in A} x_{ij}^k - \sum_{(j,i) \in A} x_{ji}^k = \begin{cases} 1, & \text{if } i = O(k) \\ -1, & \text{if } i = D(k) \\ 0, & \text{otherwise} \end{cases} \quad \forall i, k \quad (2)$$

$$z_m \leq \sum_{k \in K} \sum_{[i,j] \in E_m} (x_{ij}^k + x_{ji}^k) \quad \forall m \quad (3)$$

$$x_{ij}^k \leq y_{ij} \quad \forall k, [i, j] \in E \quad (4)$$

$$x_{ji}^k \leq y_{ij} \quad \forall k, [i, j] \in E \quad (5)$$

$$\sum_{[i,j] \in E} f_{ij} y_{ij} \leq B \quad (6)$$

$$\sum_{(i,j) \in A} x_{ij}^k \leq 1 \quad \forall i, k \quad (7)$$

$$\sum_{(i,j) \in A} x_{ij}^k \leq 1 \quad \forall k, j = D(k) \quad (8)$$

$$\sum_{\{(i,j) \in A / \forall i, j \in Q\}} x_{ij}^k \leq |Q| - 1 \quad \forall k, Q \subseteq N, 2 \leq |Q| \leq |N| - 2 \quad (9)$$

$$x_{ij}^k, x_{ji}^k = 0 \text{ or } 1 \quad \forall k, [i, j] \in E \quad (10)$$

$$y_{ij} = 0 \text{ or } 1 \quad \forall [i, j] \in E \quad (11)$$

$$z_m = 0 \text{ or } 1 \quad \forall m \in M \quad (12)$$

$$x \in S \quad (13)$$

In multiobjective programming, there may never exist a single solution that optimizes all the objectives. Thus, the notion of an optimal solution is not relevant here and is replaced by the concept of a noninferior solution set from which the decision-maker

selects the most preferred solution. A solution to the above formulation is said to be noninferior if there exists no other feasible solution that improves one objective without degrading the other. There are two popular solution approaches to multiobjective programming: preference based techniques and generating techniques [Cohon, 1978]. The latter approach, which has been commonly employed due to its simplicity, generates the entire noninferior solution set or an approximation of it. We adopt this approach. An approximation of the set can be obtained by using a single objective function formed by different convex combinations of the objective functions. Our objective function becomes:

$$Z(w_1, w_2) = w_1 Z_1 + w_2 Z_2 \quad (14)$$

where $w_1 + w_2 = 1$, and $w_1, w_2 \geq 0$.

The use of the weight pair (w_1, w_2) reflects the explicit tradeoffs between the total routing cost and the demand that can be covered. Due to the discrete nature of the MCNDP solution set, the above weighting method is incapable of identifying noninferior solutions that lie in the duality gap of the convex hull of the set. Figure 3.1 illustrates an example of a noninferior frontier which is a discrete set of points. The boundary of its convex hull is obtained by joining its exterior points. However, there can be non-dominated solutions that lie within the convex hull. For example, point B represents such an instance in the figure. Hence, point B is said to lie in the duality gap of the convex hull. Solving a constrained version to enable such identification entails a substantial increase in the computational effort. Also, the number of noninferior solutions is large, though finite, for even small-size discrete optimization problems and in the worst case increases exponentially with problem size. Hence, it is not practical to generate the entire noninferior set. Instead, it is appropriate to focus on generating an approximation to the noninferior solution set.

Constraints 2 denote the network flow conservation constraints that require x to describe a simple path from the origin to the destination for all commodities. Constraints 3 represent the coverage constraints, which imply that a demand center is covered only when at least one of the links providing accessibility to the center is in a flow path.

Constraint sets 4 and 5 are the forcing constraints; they state that no flow is allowed in either direction of link $[i, j]$ unless the associated fixed cost is incurred. The **MCNDP** can be formulated in an alternative way by expressing the forcing constraints in an aggregated form:

$$\sum_{k \in K} x_{ij}^k \leq |K| y_{ij} \quad \forall [i, j] \in E \quad (15)$$

$$\sum_{k \in K} x_{ji}^k \leq |K| y_{ji} \quad \forall [i, j] \in E \quad (16)$$

Using constraint sets 15 and 16 instead of 4 and 5, respectively, significantly reduces the number of constraints in the formulation, and consequently the problem size. However, past efforts suggest that the generic network design models with the less efficient disaggregate formulation perform computationally much better than the more efficient aggregate formulation [Magnanti and Wong, 1984]. This is because the disaggregate version of the forcing constraints better approximates the convex hull of the set of feasible integer solutions of the **MCNDP**, and hence, yields tighter lowerbounds for the linear programming (LP) relaxation of the formulation.

Constraint 6 is the budget constraint; it states that the sum of the fixed costs of the links in any solution should not exceed B .

Due to constraint set 3, any solution can entail looping paths of the kind C1 and C2, as shown in Figure 3.2, for one or more commodities as loops help to achieve extra population coverage. We seek loopless paths and this is enabled through constraint sets 7 and 8. Constraints 7 state that the maximum flow of any commodity type exiting any node in the network should not exceed unity. Constraints 8 prevent the occurrence of loops at destination nodes (similar to C2), which is not precluded by 7.

The coverage constraints can also lead to isolated subtours such as ST (as shown in Figure 3.2) that do not share any link with the corresponding commodity flow paths in the solution. These subtours are prevented by adding subtour elimination constraints 9, in which Q denotes the nodes in a subtour. Since, $|Q|$ can potentially take several values between 2 and $|N|-2$, there are potentially an exponential number of such constraints.

Constraint sets 10, 11 and 12 restrict the flow, design, and coverage variables, respectively, to 0-1 values.

Constraint 13 restricts the flow variables to the set S which represents a restricted solution domain S in which some flow variables are fixed *a priori*. As discussed in the next chapter, this is necessary to restrict the geographical region within which the search is performed to enable computational time savings. Presumably, the commodity paths to be determined would be confined to a subnetwork surrounding the O-D pair, and not circuitous. Hence, it models the topological restrictions upon the commodity flows in the network.

The **MCNDP** is a linear integer program with $(2|K||E|+|E|+|M|)$ integer variables and comprises of $(|K||N|+|M|+2|K||E|+1+|N||K|+|K|)$ constraints and an exponential number ($O(2^{|E|})$) of subtour elimination constraints.

3.4 Computational Complexity of MCNDP

Lemma: The **MCNDP** is NP-hard.

Proof: We prove it by restriction. Consider the instance **MCNDP_R** of problem **MCNDP**: $|M| = 0$, that is, there are no demand centers and the objective function is just the total routing cost of the various commodities. The coverage constraints 3 cannot be imposed as there are no demand centers. Also, the constraints sets 7, 8 and 9 are redundant in the new formulation and can be ignored. Solving this instance, which has only one objective, is equivalent to solving a budget design problem which is known to be NP-hard [Johnson et al., 1978]. Therefore, the **MCNDP** generalizes the budget design problem. It follows that the **MCNDP** is NP-hard.

Hence, the **MCNDP** is rather intractable, implying that the solution approach is typically an enumeration-type procedure.

3.5 Valid Inequalities

Typically, integer programming formulations are solved using branch-and-bound type methods that solve linear programs at the nodes of the branch-and-bound tree. The computational time for such procedures is dependent on the number of tree nodes

enumerated to obtain the solution, as well as the time required to solve the formulation at each enumerated node. The number of nodes to be enumerated is dependent on the quality of the bounds generated by associated LP relaxations. Valid inequalities, which are constraints based on the problem characteristics, are useful in this context. They are redundant in an integer programming formulation but can eliminate non-integer solutions that are optimal for the LP relaxations. Hence, they improve the lower bounds computed by the solution algorithms, thereby generating computational time savings. Valid inequalities can be appended to the **MCNDP** to enhance the formulation.

Proposition 1. The constraint

$$x_{ij}^k + x_{ji}^k \leq y_{ij} \quad \forall k, [i, j] \in E \quad (17)$$

is a valid inequality for the **MCNDP**.

Proof: In any feasible design, a link $[i, j]$ is either chosen or not. Consider the case where link $[i, j]$ is selected, which implies $y_{ij} = 1$. Then, the corresponding set of inequalities for that link in constraints 17 are its subtour elimination constraints for $|Q| = 2$. If that link is not selected, it implies $y_{ij} = 0$. Then, the forcing constraints imply that $x_{ij}^k = 0, \forall k$, and the corresponding constraints in 17 are valid. This completes the proof.

This valid inequality is a virtual constraint vis-à-vis the problem as it is not a direct representation of a physical reality or a logical characteristic. However, it can replace constraints 4 and 5, and part of constraints 9 (that is, for $|Q| = 2$). Hence, it can be used to generate some computational time savings by reducing the problem size.

Proposition 2. The following constraints are valid for the **MCNDP**.

$$y_{ij} \leq z_m \quad \forall m, [i, j] \in E_m \quad (18)$$

$$x_{ij}^k \leq z_m \quad \forall m, k, [i, j] \in E_m \quad (19)$$

$$x_{ij}^k + x_{ji}^k \leq z_m \quad \forall m, k, [i, j] \in E_m \quad (20)$$

Proof: If $x_{ij}^k = 1$ for some k and $[i, j] \in E_m$ then y_{ij} assumes the value 1, and the demand center m is covered implying $z_m = 1$. If $x_{ij}^k = 0 \forall k, [i, j] \in E_m$ then no fixed cost is incurred for the $\forall [i, j] \in E_m$ and the demand center m is not covered. This completes the proof of constraints 18. The proof of constraints 19 follows directly from the forcing constraints, and the result just proved. Constraint set 18 is redundant in the **MCNDP** formulation and directly follows from constraints 17 and 18.

These valid inequalities imply that if a demand center m is not chosen, then links in E_m do not appear in the solution. This simplifies the budget constraint and network flow conservation constraints, leading to potential computational efficiencies.

Proposition 3. Let $m1$ and $m2$ be two demand centers such that $E_{m1} \subseteq E_{m2}$, then the following relation holds.

$$z_{m1} \leq z_{m2} \quad (21)$$

Proof: Suppose demand center $m1$ is covered by flow of some commodity k in arc (i, j) , this flow also covers demand center $m2$. If demand center $m1$ is not covered, constraint 21 reduces to the nonnegativity constraint for the variable z_{m2} . The proof is complete.

This valid inequality can potentially reduce computations by exploiting the problem characteristic that if a demand center is covered by a link (i, j) , it is redundant to search other links to cover this demand center.

In summary, the **MCNDP** is a NP-hard integer programming formulation. It is exacerbated by the need for subtour and looping elimination constraints. In addition, valid inequalities can be proposed to increase computational efficiency by exploiting the problem structure. The next chapter discusses the application of the **MCNDP** to the determination of critical routes for earthquake response planning.

Figure 3.1 Example of noninferior solution in the duality gap

Figure 3.2 Example of a solution with looping paths and a subtour

CHAPTER 4 CASE STUDY: CRITICAL ROUTES FOR EARTHQUAKE RESPONSE PLANNING

This chapter discusses the application of the **MCNDP** formulation to the determination of critical routes for earthquake response in southwest Indiana. It first describes the details of the case study, followed by a summary of the solution procedure and its implementation. A problem reduction strategy is proposed to reduce computational times.

4.1 Case Study

The **MCNDP** is used to determine the critical routes under earthquakes for a network representing southwest Indiana, as shown in Figure 4.1. It consists of 184 nodes, 307 links, and 93 population centers. The demand of a center is set equal to its population as obtained from the US Census 2000 Data [US Census Bureau, 2000]. Fifteen centers have a population greater than 5000 and are denoted as major population centers. The remaining 78 have a population between 3000-5000, and are labeled minor population centers. O-D node pairs, which represent the commodities (that is each O-D pair represents one commodity), are chosen so as to ensure connectivity between the major population centers. From Figure 4.1, the 15 major population centers can be viewed to lie in five different layers from top to bottom: 3 centers each in the first three levels, 2 in the fourth level, and 4 in the fifth level. To enable connectivity across the region it is reasonable to choose O-D node pairs that connect major population centers between adjacent layers, as also those within each layer. Note that the network topology constituted by any feasible set of critical routes is connected. This approach generated 33 O-D node pairs. Using engineering judgment, the paths of 5 O-D pairs are pre-determined. The fixed cost of a link is the total retrofit area of all the bridges on that link. The link routing costs are their free-flow travel times, implying that link capacity is

ignored. This is because the problem addresses the planning stage of earthquake emergency response. The link travel times are assumed to be symmetric. The links that can potentially cover a population center are those that are within some pre-specified threshold distance of it. They were identified by observing its location and the proximity of roads (interstates, US roads and state roads) to it using the geographical map of the region.

4.2 Solving the **MCNDP**

4.2.1 Solution Procedure

The **MCNDP** is solved using the branch-and-cut algorithm in CPLEX [ILOG CPLEX 7.1, 2001]. It is a search technique that uses the tree structure in which CPLEX dynamically adds cut constraints at the tree nodes to reduce the tree size. The solution procedure has two primary computational aspects. The first is the number of the tree nodes searched in the branch-and-cut method since each requires the solution of a linear program (that is, the LP relaxation of the **MCNDP** formulation). The LP relaxation represents the formulation obtained by allowing the integer decision variables to assume continuous values in the range within their upper and lower limits. The optimal value of this formulation provides a lower bound for the **MCNDP** formulation as it is less restrictive than the original formulation. Thereby, larger the number of tree nodes searched, the greater the computational time. This highlights the critical importance of the second computational aspect, the **MCNDP** formulation size, which depends on the number of constraints. In this context, computational savings can be generated by using valid inequalities, and/or by including or excluding the subtour elimination constraints. The solution procedure first excludes most of the subtour elimination constraints to make the problem computationally tractable. The only ones included are those for $|Q|=2$ implying link subtours. This is because there are only a few constraints with $|Q|=2$, and preliminary analysis indicates that the associated subtours can occur frequently in the solution if they are not precluded. Next, the solution procedure identifies subtours in the solution obtained from CPLEX using this relaxed formulation. If subtours exist, it adds

the necessary subtour elimination constraint(s) and reoptimizes the new formulation. This step is repeated until no more subtours are encountered in the solution.

The best-known integer solution, which is the upper bound to the optimal solution, is the final solution obtained from CPLEX. The lower bound to the optimal solution is the best objective function value across all LP relaxation solutions of the branch-and-cut nodes. They are used to determine the percentage optimality gap, which is a measure of the quality of the solution. The percentage optimality gap is defined as $[(\text{upper bound} - \text{lower bound}) * 100] / \text{lower bound}$, where the bounds are obtained from CPLEX. It indicates how close the best-known integer solution is to the optimal solution.

4.2.2 Implementation Details

The computing environment consists of a Sun Ultra Enterprise server E6500 with 26 400Mhz UltraSparc II processors under the multi-user Solaris 7 operating environment with 23GB RAM, 131GB swap space and 8MB cache. A C++ program was implemented to solve the problem and was compiled using GNU, g++ v2.95.3. It invokes subroutines from the CPLEX Callable Library version 7.1 to construct the formulation and solve it with the in-built mixed integer programming (MIP) optimizer that provides a customized branch-and-cut procedure. Experiments are performed by excluding and including the valid inequalities in the formulation.

A preliminary analysis was performed to calibrate the CPLEX parameter settings for different experimental scenarios in terms of the budgets and the relative weights of the two objectives, and for different % optimality gaps. A trial and error approach was adopted in order to determine a good choice for the optimality tolerance level to solve an instance to the maximal permissible closeness to optimality as possible within a reasonable computational time. Our test runs indicated that for very low values w_2 (≤ 0.005), default settings yielded good computational performance. However, some analysis was required to arrive at a good setting for higher values of w_2 . It was based on aggressive probing, best estimate search for the node selection strategy, and variable branching using pseudo reduced costs. The CPLEX preprocessor was set to on, as it was able to achieve reduction in the MIP problem size leading to faster solution times. Also,

based on our preliminary analysis a priority order was assigned to the variables in which branching would be performed first on the x variables, then on the y variables and finally on the z variables. Each instance was solved under exactly identical parameter settings for the set of experiments with or without valid inequalities to compare the performance of the two cases.

4.2.3 Problem Reduction Strategy

The preliminary runs indicated the intractable nature of the **MCNDP**; an instance with budget $B = 3.33$ million sqft and weight pair $(w_1, w_2) = (0.99, 0.01)$ required over 5 days to solve to a 1% optimality gap. To generate significant computational time savings, a strategy to reduce the search domain was incorporated based on the notion that the solution for an O-D pair would lie within some restricted geographical area around it. Therefore, it is reasonable to expect that the path for an O-D pair is confined to a subnetwork around it rather than including circuitous components that are geographically further away. This enables us to set the commodity flow variables to zero for the rest of the network, which provides the restricted solution domain S for the formulation.

Figure 4.1 Road network of southwest Indiana

CHAPTER 5 RESULTS FROM COMPUTATIONAL EXPERIMENTS

This chapter reports the results and derives insights from the computational experiments conducted using the case study. The primary objective of the experiments is to analyze model sensitivity for different budgets and relative weights for the two objectives. We make a few comments on the computational performance. Subsequently we seek to develop noninferior frontiers that illustrate the trade-offs between the conflicting objectives for different budgets to provide practical insights to decision-makers.

5.1 Model Sensitivity and Computational Performance

An effective problem reduction strategy was sought to be identified based on computational efficiency of the test runs conducted. In the runs all the valid inequalities were included for computational efficiency. A strategy which sets 83.46% of the total number of flow variables to zero *a priori*, performed in a robust manner. Hence, further experiments were conducted using this strategy. The formulation was solved for the following weight pairs: (0.999,0.001), (0.99,0.01), (0.95,0.05) and (0.9,0.1) for the budget range 1.43 million sqft to 3.33 million sqft. The value 1.43 million sqft is the minimum budget required for a feasible solution to the formulation. Table 5.1 illustrates the total travel time and the total population covered for the instances solved. The computational times were higher for larger w_2 values because of the need to search more routes due to the increased importance of population coverage in the objective function. When w_2 is smaller, the problem approaches the budget design problem where the network structure can be favorably exploited to yield better computational performance. For low budgets, the problems are more difficult to solve due to interactions between commodities caused by the need to share links in the final solution. This was also observed by Dionne and Florian [1979]. In our experiments, subtours occur in the

intermediate solutions for all budgets with weight pairs (0.95,0.05) and (0.9,0.1), and only for two instances in (0.99,0.01). This is because a larger weight for population coverage leads to physically meaningless isolated subtours in an effort to improve the objective function value.

5.2 Noninferior Frontiers: Travel Time-Population Coverage Trade-offs

Figure 5.1 displays the noninferior solution set for the budget range 1.43 million sqft to 3.33 million sqft. In the figure, the solutions towards the left of the graph for any budget correspond to lower w_2 values; for example, the leftmost points correspond to $w_2 = 0.001$.

In general, for a budget, higher values of w_2 lead to greater population coverage at the expense of increased system travel time. This plot provides flexibility [Hall, 1985] to the decision-maker by generating a range of solutions. Noninferior solution sets, like the ones shown here, can be very useful to the decision-maker as they highlight the tradeoffs between conflicting objectives. They also aid decision-makers to compare the additional benefits accrued in terms of system travel time savings and extra population coverage due to additional budget. For example, the marginal benefits that could be realized from a budget of 2 million sqft instead of 1.67 million sqft would be higher than those under 2.33 million sqft instead of 2 million sqft.

Figures 5.2a-g depict the critical routes for seven of the instances solved above. For the lowest budget, 1.43 million sqft., the subnetwork formed by the critical routes is sparse, but still covers approximately 90% of the total population of the region. For the critical route networks shown in these figures, the travel times and the population covered are higher as the weight for population is increased, keeping the budget fixed. This is because the routes are more circuitous.

Figure 5.1 Noninferior solution set for different budget values

Figure 5.2b Critical routes for the weight pair (0.99,0.01) and budget 1.67 million sqft

Figure 5.2c Critical routes for the weight pair (0.9,0.1) and budget 1.67 million sqft

Figure 5.2d Critical routes for the weight pair (0.99,0.01) and budget 2 million sqft

Figure 5.2e Critical routes for the weight pair (0.95,0.05) and budget 2 million sqft

Figure 5.2f Critical routes for the weight pair (0.99,0.01) and budget 2.33 million sqft

Figure 5.2g Critical routes for the weight pair (0.95,0.05) and budget 2.33 million sqft

Table 5.1 Objective function values for the instances solved

Budget (million sqft)	(w_1, w_2)	Total travel time (minutes)	Total population covered	% of Total population covered
1.43	(0.999,0.001)	1656.67	409572	89.46
	(0.99,0.01)	1670.96	412062	90.00
	(0.95,0.05)	1670.96	412062	90.00
	(0.9,0.1)	1686.42	412354	90.06
1.67	(0.999,0.001)	1442.33	415148	90.68
	(0.99,0.01)	1510.70	430861	94.11
	(0.95,0.05)	1758.24	437885	95.64
	(0.9,0.1)	1766.11	438561	95.79
2.00	(0.999,0.001)	1350.84	423192	92.44
	(0.99,0.01)	1454.16	441712	96.48
	(0.95,0.05)	1639.79	450129	98.32
	(0.9,0.1)	1903.84	449489	98.18
2.33	(0.999,0.001)	1342.47	433975	94.79
	(0.99,0.01)	1428.82	448727	98.01
	(0.95,0.05)	1601.52	452715	98.88
	(0.9,0.1)	1809.39	455482	99.49
2.67	(0.999,0.001)	1335.77	436682	95.38
	(0.99,0.01)	1407.88	450804	98.47
	(0.95,0.05)	1620.63	456039	99.61
	(0.9,0.1)	1786.13	455953	99.59
3.00	(0.999,0.001)	1335.77	436682	95.38
	(0.99,0.01)	1409.15	452018	98.73
	(0.95,0.05)	1561.31	455904	99.581
	(0.9,0.1)	1713.74	457111	99.85
3.33	(0.999,0.001)	1335.77	436682	95.38
	(0.99,0.01)	1409.15	452018	98.73
	(0.95,0.05)	1584.22	456184	99.64
	(0.9,0.1)	1635.3	456448	99.70

CHAPTER 6 RECOMMENDED CRITICAL ROUTES CONSISTENT WITH INDOT (SAC) NEEDS

This chapter reports the results from the computational experiments conducted using the case study after incorporating the recommendations of the Study Advisory Committee based on the critical route subnetworks identified in Chapter 5. Section 6.1 states the recommendations briefly and discusses them. Section 6.2 discusses the critical routes obtained by incorporating the recommendations.

6.1 Feedback from Study Advisory Committee and Emergency Response Operators

Based on the feedback from the SAC, the following recommendations were proposed for the critical routes:

- 1) Inclusion of the section of I-64 that passes through the study region.
- 2) Provision of connectivity of the critical routes to adjacent states, namely, Illinois and Kentucky.

The first recommendation attempts to address the omission of I-64 in the critical routes shown in Chapter 5. Since I-64 is a freeway, it would potentially provide an efficient critical route subnetwork component due to the superior structural and maintenance characteristics of freeways; that is, it would need minimal budgetary investments to ensure seismic tolerance. Hence, it would seem natural for it to be a part of at least some of the critical routes. To analyze the reasons for its omission in the critical route subnetworks suggested in Chapter 5, we revisit the model presented in chapter 3 that determines the critical routes. This model trades off three factors, namely total travel time, population covered and budget. Observing the figures (figures 5.2a-g) shown in Chapter 5, we note that the population centers close to I-64 that are not covered in most critical routes are Griffin, Elberfield and Lynnville. Their populations are 160,

636 and 781, respectively. Hence, together they constitute only 0.34% of the total population of the study region. Due to the budgetary constraints, the benefits of higher population coverage achievable through the routes other than I-64 are greater than the lower total travel time achieved due to its inclusion. Hence, the model excludes I-64 from the critical routes shown in Chapter 5.

Routes that provide connectivity to adjacent states are important. They aid in the coordination of response plans with adjacent states and also enable the response operators in Indiana to utilize the assistance offered by their counterparts in the adjacent states in the event of an earthquake primarily causing damage in Indiana. This potentially increases response effectiveness.

Both sets of recommendations were incorporated into the determination of the critical routes by adding the associated constraints into the original **MCNDP** formulation. This also means that the new formulation has a more restricted solution set compared to the original one. Therefore, the new objective function value (denoting the optimal solution for the more-constrained problem) incorporating these additional constraints can never be better than that of the original formulation corresponding to the results in Chapter 5.

6.2 Characteristics of the Recommended Critical Routes

After the addition of constraints to incorporate the SAC recommendations, the minimum budget required for a feasible solution increased to 1.67 million sqft. The critical routes were solved for budgets ranging from 1.67 million sqft. to 3.33 million sqft. Table 6.1 shows the total travel time and the total population covered for the various instances solved. Figure 6.1 depicts the noninferior solution frontier obtained. It illustrates that the marginal benefits realizable from a 2 million sqft. budget compared to the 1.67 million sqft. case are higher than those corresponding to the noninferior frontiers illustrated in Chapter 5 for the same budgets. The results (Figures 6.2a-f) suggest that the critical routes identified in Chapter 5 do not differ perceptibly from those computed here. For identical problem parameters, in most cases, the total travel time and the total population covered have worsened due to the recommendations. This is because the addition of new constraints leads to a more restricted solution set.

It may be noted that, in some instances, the results in Table 6.1 seem better than those shown in Table 5.1. This is not because the solution is inherently better in the restricted case. As just stated, the restricted *optimal* solution will always be worse. However, due to the problem complexity (NP-hard problem) discussed in Chapter 3, neither of these two cases is solved to optimality, but only to a certain optimality tolerance level. Hence, when more variables are fixed (as is done in Chapter 6), the problem can in many instances be solved much closer to optimality though the problem is more constrained.

Connectivity was provided at three sections with Illinois and two sections with Kentucky as desired by the SAC.

Figure 6.1 Noninferior solution set for different budget values

Figure 6.2b Recommended critical routes for the weight pair (0.9,0.1) and budget 1.67 million sqft

Figure 6.2c Recommended critical routes for the weight pair (0.99,0.01) and budget 2.00 million sqft

Figure 6.2d Recommended critical routes for the weight pair (0.95,0.05) and budget 2.00 million sqft

Figure 6.2e Recommended critical routes for the weight pair (0.99,0.01) and budget 2.33 million sqft

Figure 6.2f Recommended critical routes for the weight pair (0.95,0.05) and budget 2.33 million sqft

Table 6.1 Objective function values for the instances solved

Budget (million sqft)	(w_1, w_2)	Total travel time (minutes)	Total population covered	% of Total population covered
1.67	(0.999,0.001)	1662.63	408422	89.21
	(0.99,0.01)	1685.59	410802	89.73
	(0.95,0.05)	1686.69	410802	89.73
	(0.9,0.1)	1715.19	411855	89.96
2.00	(0.999,0.001)	1391.75	413091	90.23
	(0.99,0.01)	1466.27	429665	93.85
	(0.95,0.05)	1646.53	434609	94.93
	(0.9,0.1)	1755.28	434461	94.90
2.33	(0.999,0.001)	1346.02	427829	93.45
	(0.99,0.01)	1398.75	445473	97.30
	(0.95,0.05)	1584.67	449826	98.25
	(0.9,0.1)	1778.19	450736	98.45
2.67	(0.999,0.001)	1337.33	435301	95.08
	(0.99,0.01)	1425.29	450589	98.42
	(0.95,0.05)	1599.52	453614	99.08
	(0.9,0.1)	1695.34	455026	99.39
3.00	(0.999,0.001)	1328.15	436562	95.36
	(0.99,0.01)	1411.76	451015	98.51
	(0.95,0.05)	1592.24	455519	99.50
	(0.9,0.1)	1689.16	456991	99.82
3.33	(0.999,0.001)	1323.77	436682	95.38
	(0.99,0.01)	1407.76	452018	98.73
	(0.95,0.05)	1516.12	456048	99.61
	(0.9,0.1)	1614.49	457661	99.97

CHAPTER 7 CONCLUSIONS

This chapter summarizes the study and highlights its contributions.

7.1 Summary

Seismic retrofit planning plays an important role in determining the effectiveness of the operational stage of earthquake response management. At a strategic level, it is dependent on the routes critical for effective emergency response. However, past studies have not addressed the identification of critical routes under budget constraints. This is a very important practical problem for emergency response planners as it involves identifying effective transportation routes while accounting for bridge structural conditions and budget limitations. It implies the identification of bridges to retrofit under earthquake response planning. This study develops a network-level methodology to assist decision-makers in identifying an effective seismic retrofit scheme for the bridges of a network. It introduces the **MCNDP**, which seeks routes that minimize the total routing costs over the selected routes and maximizes the total demand covered, subject to a budget constraint. An integer programming formulation of the problem is presented, and is shown to be NP-hard. The problem characteristics are exploited to develop some valid inequalities.

In general, the **MCNDP** model is useful in addressing network-level disaster management planning. The **MCNDP** is applied to generate critical routes for earthquake response planning in southwest Indiana, which is part of a seismically active region that includes parts of Illinois, Kentucky and Missouri as well. The branch-and-cut procedure of the CPLEX MIP optimizer is used to solve the problem. A problem reduction strategy is proposed to reduce computational time while ensuring that the resulting solution is close to or equal to the optimal solution. Noninferior frontiers are generated using convex

combinations of the two objectives so as to provide practical insights for decision-makers. These frontiers assist the decision-maker in analyzing the trade-offs among various objectives and making budget allocation decisions.

A new set of critical routes were developed based on the SAC feedback for the first set of critical routes identified. The SAC recommendations were to include the section of I-64 passing through the study region, and to provide connectivity to the adjacent states (Illinois and Kentucky) to ensure seamless inter-state coordination and cooperation in the event of an actual earthquake. These recommendations were incorporated to generate a new set of critical routes through the addition of constraints to the **MCNDP** formulation. The updated critical routes do not differ perceptibly from those obtained from the experiments prior to the SAC recommendations.

7.2 Contributions of the Study

This study formally defines the Multicommodity Maximal Covering Network Design Problem (**MCNDP**) and formulates it. To our knowledge, it represents the first attempt to incorporate the coverage objective in a generic network model, leading to a new class of models with significant practical implications. Past work has used the coverage criterion in identifying a path, tree or tour on a network by considering either the routing or the fixed cost, while the proposed model simultaneously considers the routing and fixed costs subject to a budget constraint. That is, none of the existing models consider routing costs, fixed costs and coverage criterion in a single framework.

From a practical standpoint, the key contribution of this study is its ability to address planning problems faced by emergency response agencies vis-à-vis disaster management. The specific problem addressed here relates to earthquake response management. Under budget constraints, there is a need to determine an effective retrofit plan for the bridges in a seismically-prone region. Most studies in the literature have adopted a local perspective to solving this problem, and do not consider the effect of potential bridge failures on the transportation system performance. The few methodologies that use a systems approach do not consider the influence of retrofit decisions on the quality of the associated routes vis-à-vis effective earthquake response. The proposed model fills this critical gap by

considering the total travel time for the key O-D pairs in the network and the total population covered by them as the criteria for determining the critical routes. The budget limitation on the total bridge retrofit costs for the links constituting the critical routes is represented as a constraint in our model. Hence, this methodology simultaneously determines the set of bridges in a network requiring retrofit and the routes that serve as focal points for earthquake response.

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Final Report: Part II

FHWA/IN/JTRP-2003/22

EARTHQUAKE EMERGENCY ROUTES FOR THE STATE OF INDIANA
ROUTE SEISMIC VULNERABILITY ASPECTS

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INDOT Research

TECHNICAL *Summary*

Technology Transfer and Project Implementation Information

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Final Report

Emergency Earthquake Routes for the State of Indiana (A Two-Volume Report)

Introduction

The possibility of very strong earthquakes in the central Mississippi Valley has been known for 200 years in the New Madrid Seismic Zone. Awareness of the seismic risk in the State of Indiana has increased in the last decades. In the last two decades identification of an independent tectonic process occurring in the Wabash River Valley has implications for the seismic risk of the State of Indiana from what has recently been defined as the Wabash Valley Seismic Zone. The occurrence of a strong earthquake in these seismic zones or in Indiana or the neighboring states due to other not so well understood tectonic processes, as has occurred in the past, has serious implications for the State Transportation System. The definition of Earthquake Emergency Routes for the state is a priority for the Indiana Department of Transportation. The identification of Emergency Routes takes into account issues related to transportation including coverage of population and area and travel time along these routes.

This study formulates a multicommodity maximal covering network design problem (**MCNDP**) for identifying critical routes, for earthquake response and to seismically retrofit bridges. The **MCNDP** seeks routes that minimize the total travel time over the selected routes and maximizes the total population covered, subject to a budget constraint on bridge retrofitting costs on the selected routes. The problem is formulated as a two-objective

integer programming model and solved using the branch-and-cut module in the CPLEX optimizer. The model performance was analyzed using the transportation network of a seismically-prone region in southwest Indiana. Thereby, the search for the critical routes for an origin-destination (O-D) pair is confined to a limited geographical region around it.

A Geographical Information System (GIS) system was developed for evaluation of seismic vulnerability of the proposed Indiana Emergency Routes. The GIS based implementation of the technical information developed in this project through FEMA HAZUS-MH software was conducted in collaboration with the Polis Center in Indianapolis, nationally recognized and FEMA authorized HAZUS implementation facility. HAZUS software may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in southwestern Indiana. It uses information from the Indiana Department of Transportation (INDOT) maintenance database and selected structural and geotechnical information from the bridge drawings data base. Vulnerability assessment was performed using the general information from maintenance and final calibration was performed using a series of cases based on bridge drawings and soil exploration descriptions contained in these drawings.

Findings

Computational experiments were conducted to investigate the effects of varying the budget and the relative weights of the two objectives. A set of

critical routes was obtained. Seismic retrofit planning plays an important role in determining the effectiveness of the operational stage of earthquake

response management. At a strategic level, it is dependent on the routes critical for effective emergency response. However, past studies have not addressed the identification of critical routes under budget constraints. This is a very important practical problem for emergency response planners as it involves identifying effective transportation routes while accounting for bridge structural conditions and budget limitations. It implies the identification of bridges to retrofit under earthquake response planning. This study develops a network-level methodology to assist decision-makers in identifying an effective seismic retrofit scheme for the bridges of a network.

This study formally defines the Multicommodity Maximal Covering Network Design Problem (MCNDP) and formulates it. To our knowledge, it represents the first attempt to incorporate the coverage objective in a generic network model, leading to a new class of models with significant practical implications. Past work has used the coverage criterion in identifying a path, tree or tour on a network by considering either the routing or the fixed cost, while the proposed model simultaneously considers the routing and fixed costs subject to a budget constraint. That is, none of the existing models consider routing costs, fixed costs and coverage criterion in a single framework.

From a practical standpoint, the key contribution of this study is its ability to address planning problems faced by emergency response agencies vis-à-vis disaster management. The specific problem addressed here relates to earthquake response management. Under budget constraints, there is a need to determine an effective retrofit plan for the bridges in a seismically-prone region. Most studies in the literature have adopted a local perspective to solving this problem, and do not consider the effect of potential bridge failures on the transportation system performance. The few methodologies that use a systems approach do not consider the influence of retrofit decisions on the quality of the associated routes vis-à-vis effective earthquake response. The proposed model fills this critical gap by considering the total travel time for the key O-D pairs in the network and the total population covered by them as the criteria for determining the critical routes. The

budget limitation on the total bridge retrofit costs for the links constituting the critical routes is represented as a constraint in our model. Hence, this methodology simultaneously determines the set of bridges in a network requiring retrofit and the routes that serve as focal points for earthquake response.

The evolution of the earthquake resistant bridge design requirements contained in the AASHTO Specifications and the existence of a proposed draft seismic design specification being discussed have significant implications in the assessment of the existing vulnerability, its mitigation, and the design of new bridges along the Emergency Routes. Along with this development, the USGS assessment of the seismic risk of southwestern Indiana and a change of format of the definition of the earthquake design ground motion that is integral part of the proposed draft seismic design specification also has important implications on the assessment of the operational capabilities of the Earthquake Emergency Routes of Indiana. The effects of the 2001 Nisqually Earthquake that occurred in the State of Washington gave invaluable insight on the expected situation for the State of Indiana due to a comparable bridge inventory in age and bridge types and a parallel increase of the seismic risk awareness in both regions over the last decades.

Using the information collected in this study, the GIS implementation through FEMA-HAZUS-MH software may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in southwestern Indiana. Vulnerability was evaluated for different levels of ground acceleration in order to obtain behavior patterns for selected bridges. Using these results, seismic behavior patterns were obtained for bridges located in southwestern Indiana included in the INdot Vincennes District maintenance data base. Soil amplification and liquefaction potential were evaluated for the cases having reported SPT measurements from borings. Based on this process, information critical for the identification of upgrade needs for the transportation structures part of the Earthquake Emergency Routes is made available to the INdot.

Implementation

The findings and developments of this research project are presented next in the form of suggested implementation steps by the Indiana Department of Transportation.

1. Formal adoption of the selected emergency routes or some variation of the same by the state is a necessary step. Such routes adopted and maintained by INdot will be affected by issues and policies outside the scope of those considered in this study. For instance, policy decisions, budgetary constraints, new projects, i.e. continuation of I-69, will likely affect the formal adoption of a set of emergency routes and will continue to impose changes in the future. The methodology developed in this study and the information implemented in HAZUS-MH should be of assistance to state decision makers in the future.
2. Future implementation strategies can be studied through simulation studies using appropriately updated information for different earthquake scenarios including earthquake ground motion defined in current AASHTO Standard Specifications, proposed Draft AASHTO LRFD Seismic Provisions, or other. These simulations will provide corresponding bridge retrofit scope for the proposed routes or alternative definitions.
3. The information in the maintenance database should be periodically evaluated and should include information currently available only in the bridge drawings (see Section 7.5 of the final report). This will improve the vulnerability assessment of the bridges refining the mitigation policies to adopt in the future. This expansion of the maintenance database can be extended to other counties and districts not covered in this research project.
4. Establish a program for assessing the liquefaction, soil spread, and embankment failure potential along the routes at locations other than bridge sites by providing a consistent evaluation methodology supported with the information developed in this study. This implementation will require a soil exploration program to obtain geotechnical properties of sites where no information exists or where such information was obtained years ago with different objectives than liquefaction evaluation. In the research conducted in this study, liquefaction evaluations were made using solely boring information contained in selected bridge drawings.
5. To establish procedures for emergency response under different earthquake occurrence scenarios.
6. INdot should consider the formal adoption of HAZUS-MH software for scenario management, mitigation and vulnerability studies and to train appropriate personnel. An important feature in this implementation step is the engagement of the Polis Center in the training of INdot personnel on the use of HAZUS-MH software. The Polis Center is a nationally recognized HAZUS implementation facility.

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<p>The occurrence of a strong earthquake in Indiana or neighboring states, as has occurred in the past, has serious implications for the State Transportation System with respect to adequate response. The definition of Earthquake Emergency Routes for the State of Indiana became a priority for the Indiana Department of Transportation. These Emergency Routes take into account various aspects related to transportation including coverage of population and area and travel time along these routes, and issues related to structural and geotechnical seismic vulnerability of the roads and bridges along these routes.</p> <p>Part I of the overall study report formulates a multi-commodity maximal covering network design problem (MCNDP) for identifying critical routes, for earthquake response and to seismically retrofit bridges. In the MCNDP, routes are sought that minimize the total travel time over the selected routes and maximize the total population covered, subject to a budget constraint on bridge retrofitting costs on the selected routes. The problem is formulated as a two-objective integer programming model and solved using the branch-and-cut module in the CPLEX optimizer. The model performance is analyzed using the transportation network in southwest Indiana. A problem reduction strategy is introduced to reduce computational times by recognizing that the critical routes are not usually circuitous. Thereby, the search for the critical routes for an origin-destination (O-D) pair is confined to a limited geographical region around it. To further reduce computational costs, the formulation is extended to incorporate valid inequalities that exploit the problem structure. Simulation studies are conducted to investigate the effects of varying the budget and the relative weights of the two objectives. Noninferior frontiers that illustrate the trade-offs between the conflicting objectives for different budgets are constructed to provide practical insights to decision-makers. The final selected routes in addition include constraints set by the Study Advisory Committee regarding specific roads to be included in the final set.</p> <p>Part II of the study report covers the structural and geotechnical seismic issues related to the definition of Earthquake Emergency Routes for the State of Indiana. Previous worldwide experience on bridge damage during strong earthquakes has influenced the bridge earthquake design specifications and mitigation procedures. The evolution of the relevant requirements in the AASHTO Specifications and the existence of a proposed draft seismic design specification under development as part of the NCHRP Project 12-49 have significant implications in the assessment of the existing vulnerability, its mitigation, and the design of new bridges along the Emergency Routes. Geographical Information System (GIS) was implemented to assist the evaluation of seismic vulnerability of the proposed Indiana Emergency Routes. Using the available information, the GIS implementation may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in Indiana incorporating information from the Indiana Department of Transportation (Indot) maintenance database and selected structural and geotechnical information from the relevant bridge drawings. Findings from a vulnerability assessment as well as final calibration performed using selected representative bridge data such as drawings and soil exploration descriptions are also reported. Vulnerability was evaluated for different levels of ground acceleration in order to obtain behavior patterns for selected bridges and to illustrate upgrade needs for the transportation structures on the Emergency Routes. The information contained in this report could be used to assist the engineers in conducting simulations that will help Indot to devise mitigation policies for different earthquake occurrence scenarios and to evaluate potential alternative routes.</p>			
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ABSTRACT

Awareness of the seismic risk in the State of Indiana has increased in the last decades. The possibility of very strong earthquakes in the central Mississippi Valley has been known for 200 years in the New Madrid Seismic Zone. In the last two decades identification of an independent tectonic process occurring in the Wabash River Valley has implications for the seismic risk of the State of Indiana from what has recently been defined as the Wabash Valley Seismic Zone. The occurrence of a strong earthquake in these seismic zones, or in Indiana, or in the neighboring states due to other not so well understood tectonic processes as has occurred in the past, has serious implications for the State Transportation System. The definition of Earthquake Emergency Routes for the State of Indiana became a priority for the Indiana Department of Transportation. The definition of these Emergency Routes takes into account issues related to transportation including coverage of population and area and travel time along these routes, and issues related to structural and geotechnical seismic vulnerability of the transportation structures along these routes. The transportation related issues are presented in Part I of this Report, and the structural and geotechnical seismic related issues are presented in Part II. Previous experience in the United States and elsewhere in the world with bridge damage during strong earthquakes has influenced the bridge earthquake resistant specifications and mitigation procedures. The effects of the Nisqually Earthquake that occurred in the State of Washington in 2001 give insight on the expected situation for the State of Indiana due to a comparable bridge inventory in age and bridge types and a parallel increase of the seismic risk awareness in both regions over the last decades. The evolution of the earthquake resistant bridge design requirements contained in the AASHTO Specifications and the existence of a proposed draft seismic design specification being discussed have significant implications in the assessment of the existing vulnerability, its mitigation, and the design of new bridges along the Emergency

Routes. Along with this development, the USGS assessment of the seismic risk of southwestern Indiana and a change of format of the definition of the earthquake design ground motion that is an integral part of the proposed draft seismic design specification also has important implications on the assessment of the operativeness of the Earthquake Emergency Routes of Indiana.

Vulnerability assessment was performed using the general information from inspection and maintenance, and final calibration was performed using a series of cases based on bridge drawings and soil exploration descriptions contained in these drawings. Vulnerability was evaluated for different levels of ground acceleration in order to obtain behavior patterns for selected bridges. Using these results, seismic behavior patterns were obtained for bridges located in southwestern Indiana included in the Indot Vincennes District Inspection and Maintenance Database. Soil amplification and liquefaction potential were evaluated for the cases having reported SPT measurements from borings. Based on this process, information, upgrade needs for the transportation structures part of the Earthquake Emergency Routes were obtained. With this information, it is possible to perform simulations that will help Indot in devising mitigation policies, perform simulations for different earthquake occurrence scenarios, and establish, evaluate, and implement alternatives for response strategies using the Earthquake Emergency Routes.

The Federal Emergency Management Agency (FEMA) has developed HAZUS-MH, a GIS-based software tool that empowers communities with the ability to assess risk from earthquakes, flood, and wind related disasters. While HAZUS-MH can be used to assess risk with the significant default inventory that it includes, it is possible to integrate local data into the HAZUS-MH analysis in order to produce more realistic loss estimates. As a component of the research work conducted in this project and with the approval of the Study Advisory Committee, The Polis Center from Indianapolis was tasked with the integration of bridge and soils data into HAZUS-MH for the purpose of using that tool to conduct earthquake risk analysis. The Polis Center is a FEMA authorized HAZUS-MH earthquake and flood service provider and is thus able to support the goals of INDOT in

this instance. The GIS implementation through FEMA HAZUS-MH software conducted during this study uses information from the Indiana Department of Transportation (Indot) Inspection and Maintenance Database and structural and geotechnical information from the bridge drawings data base.

CHAPTER 1 INTRODUCTION

1.1 Purpose

The purpose of the project was to propose Earthquake Emergency Routes for the State of Indiana with emphasis in the southwestern part of the State, to review the selected routes for earthquake hazards identifying conditions related to the structures along those routes that may affect their functioning in case of occurrence of a strong earthquake in the region, and to propose procedures for mitigation, evaluation and maintenance of their status as emergency routes.

1.2 Scope

1. To recommend Earthquake Emergency Routes for the Vincennes District of the Indiana Department of Transportation (INDOT), located in the southwestern corner of the State, based on travel times and scope of retrofit work necessary to implement these routes with the final decision of which routes to select resting with INDOT (see Part I of this report).
2. To develop a tool based on geographical information systems technology (GIS) that INDOT engineering may use for identifying and maintaining the Earthquake Emergency Routes network of Indiana and for establishing policies for mitigation of earthquake hazard on these routes, to perform simulations for different earthquake scenarios, and to help in the response to an actual emergency caused by the occurrence of a strong earthquake in the region.
3. To assess the seismic vulnerability of selected bridges belonging to the Vincennes INDOT District based on detailed information, and to correlate this vulnerability assessment with one obtained using just the information currently contained in the INDOT Inspection and Maintenance Database. To recommend items relevant to

the vulnerability of the routes to be added to the Inspection and Maintenance Database.

4. To give guidance on how to prioritize the mitigation of the vulnerability of existing bridges according to the analyses performed and their importance in keeping the transportation network system functional with emphasis in the Earthquake Emergency Routes selected.

1.3 Methodology

Given the multidisciplinary nature of the project, the methodology used took advantage of the most advanced resources of each discipline. The methodology corresponding to each of the components is described in detail along this report at the pertinent locations, with Chapter 4 of this report presenting a summary of the vulnerability assessment methodology used.

1.4 Participants

Joint Transportation Research Program – JTRP

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Indiana Earthquake Preparedness Committee

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1.5 Organization of the report

This report is presented in two volumes; the first one – Part I – deals with Transportation issues related to definition of the selected Earthquake Emergency Routes; this second volume – Part II – is organized in the following manner:

Chapter 2 presents background on seismic vulnerability of transportation networks. It includes the reasons behind the need of defining earthquake emergency routes. Discusses the seismic vulnerability issues linked to transportation structures. A summary of the performance of transportation networks in past earthquakes is presented. Finally, it presents and discusses the development of the specifications that govern their design including associated geotechnical issues.

Chapter 3 discusses the tectonic setting of the southwestern corner of Indiana and how it fits within the two main regional seismogenic structures – the New Madrid Seismic Zones and the Wabash River Faulting System – commenting on the earthquakes that have affected the region. The earthquake mean return period is presented and the importance of its definition for earthquake resistant design of structures is brought into

perspective by describing the values being used in current and proposed earthquake resistant bridge design specifications. The implications of adopting a draft seismic design specification by AASHTO are discussed.

Chapter 4 presents the methodology for evaluation of seismic vulnerability of the selected Indiana Emergency Routes, the available information, the Geographical Information System (GIS) implementation and its use for mitigation of vulnerability, simulation, and response to a strong earthquake in southwestern Indiana.

Chapter 5 is devoted to geotechnical sources of vulnerability for transportation structures in general and, in particular, the approach used in defining and assessing these sources of vulnerability for the Earthquake Emergency Routes of the State of Indiana.

Chapter 6 parallels the previous chapter, with the focus on bridges. The methodology used and its application is discussed in detail.

Chapter 7 defines the upgrade needs for the transportation structures part of the Earthquake Emergency Routes.

Chapter 8 presents the conclusions and recommendations of the study.

Chapter 9 lists the bibliographic references relevant to the study.

CHAPTER 2 BACKGROUND

This chapter describes previous background information related to earthquake emergency routes and performance during earthquakes of highway systems, transportation facilities, and structures with special reference to the State of Indiana. This background information has been divided in information related to highway systems, bridges, and issues associated with geotechnical aspects.

2.1 Transportation systems

The Federal Response Plan (FRP) of the United States [FEMA, 2003] created under the auspices of the Robert T. Stafford Act, which is the enabling legislation for emergency response in the United States, provides the legal mandate, the resources, and the mechanisms for an integrated emergency response after the declaration of any disaster by the President. The Federal Response Plan establishes 12 Emergency Support Functions (ESF) to facilitate the implementation and coordination of a broad spectrum of integrated emergency response activities after natural and technological disasters.

ESF No. 1 is “Transportation” and is defined as access to the disaster area. It is based on the assumptions that the transportation infrastructure in the area will sustain damage, which will influence the means and accessibility of relief services and supplies to protect people and property. The disaster responses requiring transportation capacity will be difficult to coordinate effectively during the immediate post-disaster period. Clearing of debris and completion of repairs will be gradual, in spite of best efforts, disrupting access routes for a significant period.

Transportation systems in the Central U.S. – including highways, bridges, railways, waterways, ports, and airports – are vulnerable to the effects of a damaging earthquake in the New Madrid Seismic Zone (NMSZ) [CUSEC, 2000]. Furthermore, damages to transportation systems may extend to several states, which present transportation officials in government and the private sector with unique problems and challenges.

The Central U.S. Earthquake Consortium (CUSEC) is a nonprofit organization, funded by the Federal Emergency Management Agency, which is dedicated to reducing deaths, injuries, damage to property and economic losses resulting from earthquakes occurring in the Central United States. Its members are the seven states that are most vulnerable to earthquakes in this region: Arkansas, Illinois, Indiana, Kentucky, Mississippi, Missouri, and Tennessee.

Fig. 2-1 – Map showing hypothetical maximum intensities for a magnitude 7.6 earthquake in the Central United States region [CUSEC, 2000]

The document [CUSEC, 2000] discusses the transportation system vulnerability and the earthquake risk in the Central United States taking into account the multi-state impact of future damaging earthquakes in the region including associated problems with earthquake induced hazards such as faulting, liquefaction, slope instability, dam or levee failure, and hazardous materials spills. The effects of earthquakes on the transportation system including: highway, railroad, waterway, and air transportation and liquid fuel transport; the steps that must be taken for mitigation of vulnerability of transportation systems in the region; and response and recovery policies are discussed in this important document. It assigns the following consequences of failure in a transportation system due to an earthquake or other natural disaster:

- Direct loss of life due to collapse or structural failure of the lifeline.
- Indirect loss of life due to an inability to respond to secondary catastrophes, such as fires, and/or provide emergency medical aid.
- Delayed recovery operations.
- Release of hazardous products (e.g., losses from tank cars derailed by track failure, gas leaks from ruptured utility lines) and environmental impacts.
- Direct loss of property and utility service (e.g., the collapse of a bridge carrying utilities).
- Losses due to interruption of access (e.g., export losses due to port damage).
- Disruption of economic activity across the region and nation as well as in the community directly affected.

The [CUSEC, 2000] document proposes a New Madrid Transportation Plan and Strategy, which would be the product of an intergovernmental-private sector planning process, aimed at addressing the following:

1. Loss estimates and functionality assessments for selected earthquake scenarios.
2. Common set of planning assumptions for federal, state, and local governments.
3. Criteria for decisions on establishing field operations in a multi-state, multiple region disaster.
4. Proper allocation of resources to multiple impacted areas.

5. State versus Federal roles in determining priorities and planning for infrastructure repair.
6. Procedures and criteria for conflict resolution in meeting requests for resources.
7. Pre-disaster determination of conditions that must be present before federal transportation support is withdrawn.

Similar tasks have been studied in other regions of the U.S., but there has been none requiring such a broad coordination between different States and the Federal Government. The role played by CUSEC and the State Departments of Transportation in implementing this plan can not be underestimated. At present, CUSEC has compiled the Emergency Routes shown in Fig. 2-2.

Fig. 2-2 – Earthquake Emergency Routes and staging facilities of the CUSEC State Departments of Transportation

These routes are currently under study individually by each State. Detailed literature review on transportation issues is presented in Part I of this report.

2.2 Bridges

2.2.1 Bridge performance during earthquakes

In general, the likelihood of bridge damage increases if the ground motion is particularly intense, the soils are soft, the bridge was constructed before modern codes were implemented, or the bridge configuration is irregular [Moehle and Eberhard, 2000]. Depending on these conditions, bridge damage could be grouped in the following general categories:

1. *Unseating of the superstructure* — Short seat lengths at span hinges or simple supports permit the partial or total collapse of the superstructure if the displacements caused by the ground motion exceed the support length. Curved bridges are especially vulnerable for this condition. Restrainers have been used for several years to mitigate this problem; although, there were cases in which they were not effective during the Northridge Earthquake [EERI, 1995a].
2. *Column failure* — Lack of adequate confining transverse reinforcement leads to brittle modes of failure of reinforced concrete columns and piers. Short lap splices or inadequate embedment length of anchorage at the foundation of vertical reinforcement has led to numerous failures. For steel columns, local buckling may allow damage leading to collapse.
3. *Damage to abutments* — The abutment is affected by the underlying soil conditions, the type of foundation, and the demands imposed by the superstructure movement. Usually, abutments are elements where it is difficult to provide ductility and toughness.
4. *Damage to bearings* — Bearings used in bridges designed only for gravity effect are especially prone to failure under horizontal displacement of the superstructure transverse to the direction of thermal expansion.

5. *Structural configuration* — Unique complex structures with curved or significant skew geometry, or bridges having unique features or details, are more susceptible to damage induced by ground motion.
6. *Age, Modifications and Maintenance* — Bridge design specifications have changed in recent years as a result of experience from earthquakes and structural research. The correlation between bad behavior and old bridges is high as observed in many recent earthquakes. Changes introduced after original construction sometimes affect behavior. Numerous cases in which protection barriers or walls reduce the clear height of columns have led to failures under earthquake ground motions. Deterioration of the bridge due to corrosion and other causes has had an influence in the observed behavior.
7. *Geotechnical conditions* — Liquefaction, soil spread, slope instability, proximity to the fault, approach settlement, and other causes of distress influence the response, and damage of the bridge.

The following publications describe extensively bridge behavior and damage during earthquakes [ACI, 1999], [Moehle and Eberhard, 2000], [Penzien, 2000], and [Priestley et al., 1996]. Reports from earthquakes where damage to bridges is specially described are: San Fernando, California, 1971 Earthquake [Lew et al., 1971]; Chile 1985 Earthquake [EERI, 1986]; Whittier Narrows, California, 1987 Earthquake [EERI, 1988]; Loma Prieta, California, 1989 Earthquake [EERI, 1990] and [NRC, 1994]; Costa Rica 1991 Earthquake [EERI, 1991]; Northridge, California, 1994 Earthquake [EERI, 1995a] and [CRSI, 1994], the Kobe, Japan, 1995 Earthquake [EERI, 1995b]; the Turkish Earthquakes of 1999 [EERI, 2000]; the Taiwan Earthquake of 1999 [EERI, 2001], and the 2001 Nisqually, Washington State, Earthquake [Ranf et al., 2001], and [Nisqually EQ Clearinghouse Group, 2001]. Just a summary will be presented limiting it to cases of bridges designed using U.S. or comparable specifications. Direct reference is made to the relevant U.S. earthquakes. Even with these limitations, several things have to be kept in mind in translating this information for application in the State of Indiana. These issues will be discussed after the performance information is presented.

Although there is record of previous damage to bridges both in the U.S. and abroad, the February 9, 1971, San Fernando, California, Earthquake constitutes the first case on intense damage to bridges of modern design that belonged to the Interstate system [Lew et al., 1971]. The 1971 San Fernando Earthquake affected the greater Los Angeles area and specially the northern San Fernando Valley. Post-earthquake reports by the California State Highway Commission indicated that the earthquake damaged approximately 11 miles of multilane freeways and 6 miles of conventional state highways, in addition to numerous city and county streets. Approximately 1/5 of the total damage cost to transportation infrastructure was represented in damage to sixty bridges. Several of these bridges either collapsed or were severely damaged to the point of having to be replaced. Considerable disruption of traffic was caused by the bridge damage. The heavily traveled Interstate 5 was reopened to traffic using emergency detours within five days of the earthquake occurrence. Approximately 80% of the bridge damage cost was concentrated in 4 miles of highway at the intersection of I-5, I-210, and State Road 14. It is interesting to note that one of the collapsed curved bridges rebuilt after the San Fernando Earthquake collapsed during the Northridge 1994 Earthquake.

Fig. 2-3 – Bridge collapse during the February 9, 1971, San Fernando Earthquake

The Loma Prieta, California, 1989 Earthquake [EERI, 1990] provided insight on the influence of geotechnical conditions on bridge behavior by the preponderance of damage in the perimeter of San Francisco Bay where relatively deep and soft soil deposits amplified the ground motion. The collapsed portions of the two-story Cypress Street viaduct (see Fig. 2-4) coincide with the soft soil sites. Soil conditions may have played a role in damaging a structure of 1950's with detailing common at that time that did not provide toughness.

Fig. 2-4 – Collapse of the Cypress Street Viaduct during the Loma Prieta, California, 1989 Earthquake

Loma Prieta Earthquake also highlighted the vulnerability of adjacent short and long spans imposing differential deformation demands and producing the collapse of the short span next to a large span (see Fig. 2-5). During this earthquake there were five bridge collapses (counting the extensive Cypress Viaduct as one) and four other bridges sustained major damage.

Fig. 2-5 – Collapse of a span of the San Francisco-Oakland Bay Bridge during the Loma Prieta, California, 1989 Earthquake

The Costa Rica 1991 Earthquake [EERI, 1991] confirmed the influence of soil conditions on bridge behavior especially in abutment failures, embankment settlement, and liquefaction. Collapses caused by unseating of skewed spans were also observed.

The 1994 Northridge Earthquake, [EERI, 1995a] and [CRSI, 1994], produced high accelerations throughout the Los Angeles metropolitan area where the California Department of Transportation owns and maintains approximately 2,200 bridges. Approximately 1,200 were located in zones that experienced horizontal accelerations more than $0.25g$ and several hundred bridges were subjected to accelerations more than $0.5g$ [ACI, 1999].

Five bridges presented partial or complete collapse. All these structures were of reinforced and/or prestressed concrete, with construction completion dates ranging from 1964 through 1976. Several had been retrofitted with hinge restrainers following the 1971 San Fernando Earthquake. Collapse causes in all cases appear to involve column flexural or shear failures (see Fig. 2-6), or unseating at in-span or abutment hinges (see Fig. 2-7). Three reinforced and/or prestressed concrete bridges sustained major to moderate damage

to their columns but did not collapse. There was damage to bridge abutments throughout the epicentral region.

Fig. 2-6 – Failure of flared column in the Route 118, Mission Gothic Undercrossing during the Northridge, California, 1994 Earthquake

The observed abutment damage included shear key cracking and failure, spalling or failure of abutment backwalls and wingwalls, approach settlement, and approach slab buckling. Damage in steel bridges comprised pounding damage between adjacent elements, buckling of cross bracing, bending of cross-brace gussets, bearing damage including anchor bolt and restrainer fractures, and damage to supporting abutments and pier walls.

The epicentral region contained 132 bridges that had been retrofitted using post San Fernando, 1971, details (Caltrans Phase I), and 63 with post Loma Prieta retrofit details (Caltrans Phase II). Retrofit consisted in hinge restrainers and/or column jackets. Most of the retrofitted bridges performed adequately although, in some cases, hinge restrainers

presented problems. Column jackets, whether made of steel or fiberglass, performed well in all cases.

Fig. 2-7 – Collapse of the Gavin Canyon Undercrossing during the Northridge, California, 1994 Earthquake

The Los Angeles County Department of Public Works reported serious damage in only four out of 1,500 bridges on the county system. The City of Los Angeles reported that 62 out of a total of 800 spans were damaged, and two required closure due to column damage in one case, and fill settlement in the other. Other damage to city bridges consisted of approach-fill settlement, pavement cracking, shear key damage, superstructure rotation, rocker bearing damage, and architectural damage.

Several important observations were made regarding the performance of the transportation system [EERI, 1995a].

- In this earthquake, all bridge collapses were associated with poor performance of older columns with inadequate transverse reinforcement and short seat widths, both of which have been identified as sources of vulnerability in previous earthquakes. The progress in knowledge and design practice was evident in the good performance of bridges constructed or retrofitted to current standards.

- Architectural column flares affected the pier behavior. Earlier design practice assumed that flares spalled off, whereas actually they act as an integral part of the column.
- Current and older design procedures may not adequately represent the distribution of design forces in long, multiframe bridges. Failure and damage in end frames indicates that the stiffer end frames may attract larger forces than anticipated in design.
- As with other construction forms, structural changes can occur during construction and maintenance that invalidate the design assumptions. It is important to verify that as-built conditions are consistent with design assumptions, and not just the design drawings. Improved documentation of design assumptions may be desirable.
- Caltrans Phase I hinge restrainer retrofits had mixed performance. There were many examples of restrainer unit failure (cable fracture, fitting failure, and diaphragm punching).
- Older bridges with 6-inch and 8-inch seats should be reevaluated, even if retrofitted with restrainers.
- Caltrans Phase II retrofits appeared to perform well in all instances. (Its focus was on retrofitting columns in single-column bridges because of the perception that they were more vulnerable than multicolumn bridges.)
- Several skewed bridges collapsed. An apparent cause was torsional response associated with skewed geometries. Methods are needed for improved design of skewed bridges, possibly including elimination of the skew where feasible, elimination of in-span hinges, and lengthening of seats.
- Damage to abutments and approaches was widespread. Considering the extent of this damage for a moderate earthquake, it seems that the current design strategy to accept abutment damage should be reevaluated.
- Major damage and collapse in multicolumn bridges indicates that the increased reliability associated with redundancy of multicolumn bents is not necessarily sufficient to avoid collapse. Retrofit priorities given to multicolumn bridges and

high-traffic-volume bridges may need to be reconsidered. Furthermore, damage and collapse required closure of bridges that usually carry heavy traffic volume.

- Modern traffic management techniques were effective in managing traffic following the earthquake. Furthermore, emergency procedures, including strong incentive and disincentive clauses, were effective in achieving rapid reconstruction of the freeways.

The February 28, 2001, Nisqually, Washington State, Earthquake affected the Seattle, Tacoma, Olympia region [Ranf et al., 2001], and [Nisqually EQ Clearinghouse Group, 2001]. No bridge collapses were observed during this earthquake. In total, 78 bridges were damaged. A total of 10 bridges sustained moderate damage, 16 had mild damage, and 52 bridges had minor damage. The event had a moment magnitude of 6.8 and a depth of focus of 52 km on the Juan de Fuca Plate. The distance from the hypocenter to Seattle was 78 km, 57 km to Tacoma, and 54 km to Olympia.

Fig. 2-8 – Damage to the I-5 Holgate Street overpass, Seattle, during the 2001 Nisqually, Washington State, Earthquake

The Nisqually Earthquake caused moderate ground motion throughout the Puget Sound region. Of the 31 stations for which preliminary information was available, only 13 showed peak ground accelerations more than 10%g and only 2 stations recorded values more than 25%g. Several correlations of damage with bridge characteristics and ground motion intensity were made [Ranf et al., 2001]. For 72 bridges that were damaged (6 movable bridges were excluded because of their unusual characteristics) types of damage were: 48 with damage to concrete, 6 with damage to steel, 11 with damage to beams, restrainers or joints, and 7 with damage due to settlement.

In order to help interpret the correlation of damage with age of the bridge, Figure 2-9 shows the age of construction of the total bridge inventory in the region. The number of bridges built increased from the beginning of the 1950's and then decreased at the beginning of the 1980's coinciding with the construction of the interstate highway system. Figure 2-10 shows the correlation of age of construction with damage observed. From this figure it is evident that bridges built between 1910 and 1940 suffered more damage.

Fig. 2-9 – Age of the total bridge inventory affected by the 2001 Nisqually, Washington State, Earthquake

Figure 2-11 shows the spectral accelerations in percentage of g ($T = 0.3$ s) for each bridge site for the total bridge inventory in the area and Fig. 2-12 correlates the damage observed with the spectral acceleration. Correlation of spectral acceleration and bridge damage is evident from these figures.

Fig. 2-10 – Percentage of damaged bridges by decade of construction for the 2001 Nisqually, Washington State, Earthquake

Fig. 2-11 – Spectral acceleration (%g at $T = 0.3$ s) at the bridge site for the total bridge inventory affected by the 2001 Nisqually, Washington State, Earthquake

Fig. 2-12 – Percentage of bridge damaged for different values of spectral acceleration (%g at $T = 0.3$ s) at the bridge site during the 2001 Nisqually, Washington State, Earthquake

The bridge behavior observed during the Nisqually Earthquake is relevant because it may be similar to what would be expected in southwestern Indiana for a large earthquake in the New Madrid zone. The accelerations expected are of the same order of magnitude of those recorded during Nisqually Earthquake. The bridge inventory has similar characteristic with respect to age and type of bridges. Awareness of the seismic risk in the Pacific Northwest evolved about the same time as that for awareness of the seismic risk in the Central U.S. Earthquake resistant design of new bridges started about the same time in Central U.S. and Pacific NW.

2.2.2 Bridge seismic design criteria evolution

The early days

In 1931 the American Association of State Highway Officials (AASHO) published the first edition of the *Standard Specifications for Highway Bridges* [AASHO, 1931]. These specifications were – and still are – for ordinary bridges having span lengths less than 500 feet. No reference to considering earthquake effects on bridges was made in the *Standard Specifications* until the fifth edition (1949) in which a statement requiring that earthquake effects should be considered was included without guidance on how to account for them. The same statement was kept in the sixth (1953) and seventh (1957) editions.

The eighth edition (1961) of the *Standard Specifications* [AASHO, 1961] was the first to specify an earthquake loading for design (EQ) to be applied statically in any horizontal direction as part of Group VII load combination which included along with earthquake effects: dead loads, earth pressure, buoyancy, and stream flow effects. This load combination was to be used in the working stress design (WSD) procedure with a 1/3 increase in allowable stress permitted for occasional loads. The earthquake load was defined using a seismic coefficient (C) that multiplied the dead load (D):

$$EQ = C \cdot D \quad (2-1)$$

Values of C were 0.02 for structures supported on spread footings where the soil bearing capacity was rated to be greater than 4 t/ft², 0.04 for structures supported on

spread footings where the soil bearing capacity was rated to be less than 4 t/ft², and 0.06 for structures founded on piles. No seismic zone factors or seismic zoning map was provided, leaving to the State Bridge Commissioner or the State Highway Department the decision of considering the State, or regions within the State, as seismic. These seismic provisions were an extension of the lateral force requirements for buildings developed prior to 1961 by the Structural Engineers Association of California (SEAOC). The seismic loading provisions of the eighth (1961) edition of the *Standard Specifications* were kept, without modification, in the ninth (1965), tenth (1969) and eleventh (1973) editions.

The 1975 AASHTO Interim

As a result of the 1971 San Fernando, California, Earthquake, during which many highway bridges were severely damaged and some even collapsed – as described previously – the California Department of Transportation (Caltrans) issued, in 1973, a new seismic design procedure for bridges which formed the basis of the *1975 AASHTO Interim Specifications for Highway Bridges* [AASHTO, 1975]. This document specified an equivalent static lateral force:

$$EQ = C \cdot F \cdot W \quad (2-2)$$

The procedure was restricted to bridges having supporting members of approximately equal lateral stiffness. The equivalent static lateral force was to be applied in any horizontal direction as part of the same Group VII load combination used in the eighth (1961) *Standard Specifications* edition in a working stress design procedure permitting a 1/3 increase in allowable stress. In Eq. (2-2), W represents dead load, F is a framing factor assigned the values of 1.0 for single columns and 0.8 for moment resisting frames with the horizontal force acting along the frame, and C was a combined response coefficient expressed by:

$$C = \frac{A \cdot R \cdot S}{Z} \quad (2-3)$$

in which A denotes the maximum expected peak ground acceleration (PGA) associated with each seismic risk zone of the map of the United States shown in Fig. 2-13, R is a normalized ($\text{PGA} = 1g$) acceleration response spectral value for a rock site, S is a soil amplification factor, and Z is a force reduction factor depending upon structural-component type which accounts for the allowance of inelastic deformations.

The numerical values specified for A were $0.09g$, $0.22g$, and $0.50g$ in seismic zones numbered 1, 2, and 3 respectively in the map of Fig. 2-13.

Fig. 2-13 – Seismic Risk Map contained in the
1975 AASHTO Interim Specifications for Highway Bridges

Numerical values for R , S , and Z were not provided in the *1975 AASHTO Interim Specifications for Highway Bridges*; instead four plots of C as a function of period T were given for different values of A . Each of these plots represents a different depth range of alluvium to rock-like material. The ranges were 0-10 ft, 11-80 ft, 81-150 ft, and greater than 150 ft. Figure 2-14 shows the response coefficient C values as a function of period T for 81 to 150 ft depth of alluvium. Minimum values for C were set at 0.10 for values of A greater than or equal to $0.3g$, and 0.06 for values of A less than $0.3g$.

Fig. 2-14 – Response coefficient C for 81-150 ft depth of alluvium contained in the 1975 AASHTO Interim Specifications for Highway Bridges

Period T was to be evaluated using the relationship

$$T = 0.32 \sqrt{\frac{W}{P}} \quad (2-4)$$

in which P was equal to the total uniform static load required to cause a 1-inch lateral deflection of the whole structure. For complex or irregular structures the 1975 Interim Specifications required use of the modal response spectrum analysis method to obtain design loads. For structures adjacent to active faults, sites with unusual geologic conditions, unusual structures, and structures having a fundamental period greater than 3 seconds it required that the design should be made using current seismicity, soil response, and dynamic analysis techniques. The 1975 AASHTO Interim Specifications for Highway Bridges also included a procedure for design of restraining features to limit the displacement of the superstructure including hinges, ties, and shear blocks.

The seismic design criteria contained in the *1975 AASHTO Interim Specifications* were kept in the twelfth (1977), thirteenth (1983), and fourteenth (1989) editions of *AASHTO's Standard Specifications*. The only modification was to allow the use of load-factor design (LFD) or ultimate strength design (USD). For WSD the same Group VII load combination was specified with a 1/3 allowable stress increase – see Eq. (2-5) –. When using LFD, the Group VII combination was as shown in Eq. (2-5).

$$\begin{aligned}\text{Group VII (WSD)} &= D + E + B + SF + EQ \\ \text{Group VII (LFD)} &= \gamma [\beta_D \cdot D + \beta_E \cdot E + B + SF + EQ]\end{aligned}\tag{2-5}$$

where D denotes dead load; E earth pressure; B buoyancy; and SF stream flow; $\gamma = 1.3$; $\beta_D = 0.75$ when checking columns for minimum axial load and maximum moment or eccentricity, $\beta_D = 1.0$ when checking columns for maximum axial load and minimum moment, and $\beta_D = 1.0$ for flexure and tension members; and $\beta_E = 1.3$ for lateral earth pressure and $\beta_E = 0.5$ for positive moment in rigid frames.

Although the introduction of these requirements for computing seismic forces and their use in design was an improvement with respect to previous practice, no corresponding changes were introduced in the detailing requirements for the structural materials within the *Standard Specifications*. Seen from a more recent perspective, the lack of requirements to introduce toughness to the structural elements as was being required at the same time for the earthquake resistant design of buildings [ACI, 1977] would turn later to be one of the major sources of seismic vulnerability for transportation structures designed and built during this era.

The ATC 6 and Division I-A of AASHTO Standard Specifications

In the aftermath of the February 9, 1971, San Fernando, California, Earthquake the Structural Engineers Association of California (SEAOC) established the Applied Technology Council (ATC), an independent, nonprofit, tax-exempt corporation for the purpose of implementing current technological developments into active structural engineering practice. Their initial efforts were aimed at earthquake resistant design of buildings. In the late 1970's with the sponsorship of the National Science Foundation and

the National Bureau of Standards the Applied Technology Council published document ATC-3-06 [ATC, 1978] devoted to building earthquake resistant design. This document was the draft of what was to become current Model Code seismic requirements in the United States and many countries. In 1981 ATC, with the sponsorship of the Federal Highway Administration, Department of Transportation, published document *ATC-6 Seismic Design Guidelines for Bridges* [ATC, 1981]. These guidelines with minor revision made by the National Center for Earthquake Engineering Research (NCEER) in Buffalo, NY, under the sponsorship of the National Cooperative Highway Research Program (NCHRP), were adopted as the earthquake resistant design requirements for bridges in the fifteenth (1992) edition of *AASHTO Standard Specifications*. These requirements were kept without modification in the sixteenth (1996) and in the current seventeenth (2002) edition [AASHTO, 2002] as *Division I-A Seismic Design*.

Under the requirements of *Division I-A Seismic Design* of the current 17th Edition of the *Standard Specifications* the acceleration coefficient A for the bridge site must be obtained from the acceleration contour maps provided. The northern Midwest portion of the map of horizontal acceleration A (expressed as a percent of gravity) in rock with 90 percent probability of not being exceeded in 50 years – mean return period 475 years – is shown in Fig. 2-15. This map was prepared by the U.S. Geological Survey (USGS) for the 1988 edition of the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* [NEHRP, 1988]. Although the values of A in this map do not differ in the order of magnitude from those contained in the original *ATC-6* 1981 report for the same geographical regions, the 1988 map uses a finer resolution by assigning values of A in a county by county basis and presenting them in smaller acceleration increments.

Once the acceleration coefficient is obtained, the bridge structure must be classified as either “Essential” or “Other” in accordance with given guidelines. An importance classification ($IC=I$) shall be given to Essential bridges and ($IC=II$) shall be given to ordinary bridges. The classification of “Essential” must be given to all bridges located in zones where the value of A is greater than $0.29g$ and on the basis of social, survival, security, or defense requirements. Guidance is given in the commentary of *ATC-6* for

including bridges in either importance group. The bridge also must be assigned to a Seismic Performance Category (SPC) A, B, C or D as a function of the acceleration coefficient A as defined in Table 2-1.

Fig. 2-15 – Northern Midwest portion of the map of horizontal acceleration A (g) in rock with 90 percent probability of not being exceeded in 50 years (*Division I-A Seismic Design of the current 17th Edition of the Standard Specifications*)

Table 2-1 – Seismic Performance Categories (SPC) *Division I-A Seismic Design of the current 17th Edition of the Standard Specifications*

Acceleration Coefficient	Bridge Classification	
	Essential	Other
$A \leq 0.09g$	A	A
$0.09g < A \leq 0.19g$	B	B
$0.19g < A \leq 0.29g$	C	C
$0.29g < A$	D	C

The effects of site conditions on bridge response are determined from a Site Coefficient (S) based on soil profile types summarized in Fig. 2-16.

Fig. 2-16 – Soil Profile Types
(Division I-A Seismic Design of the current 17th Edition of the Standard Specifications)

No dynamic analysis is required in *Division I-A Seismic Design* of the current 17th Edition of the Standard Specifications for bridges having single spans, regardless of the value of the site acceleration coefficient A , and for all bridges in SPC A. All other bridges, regular or irregular, having two or more spans must be analyzed by at least one of two dynamic analysis procedures, namely, the single-mode spectral method (SMSM) or the multi-mode spectral method (MMSM). The SMSM is specified as minimum for

regular bridges in SPC B, C, and D; while the MMSM is specified as minimum for irregular bridges in these same categories. An "irregular" bridge is defined as one having abrupt or unusual changes in mass, stiffness, and/or geometry from abutment to abutment; a "regular" bridge is one not meeting the definition of an "irregular" bridge.

The seismic input in any horizontal direction to be used in each of these minimum dynamic analysis procedures is specified in terms of an elastic seismic response coefficient, C_s , as expressed for the single-mode spectral SMSM method for a bridge period T by:

$$C_s = \frac{1.2 \cdot A \cdot S}{T^{2/3}} \quad (2-6)$$

The value of C_s need not exceed $2.5 \cdot A$. For sites with soil profile types III and IV in areas where A is greater or equal to $0.3g$, C_s need not exceed $2.0 \cdot A$.

Fig. 2-17 – Seismic response coefficient C_s for different soil profiles normalized with respect to acceleration coefficient A (*Division I-A Seismic Design of the current 17th Edition of the Standard Specifications*)

For the multi-mode spectral method (MMSM) the same relationship for C_s is used and for each individual vibration mode “ m ” the mode period T_m must be used in Eq. (2-6)

as T . In this case a reduction of C_s is allowed for modes other than the fundamental and periods shorter than 0.3 sec, with the value of C_s tending to the value of A at a period equal to zero. For long periods greater than 4 sec a minimum value of C_s is required.

Deformations in the nonlinear range of response of the bridge elements are expected to occur when subjected to the earthquake design ground motion prescribed in the *Division I-A Seismic Design* of the current 17th Edition of the *Standard Specifications*. Response modification factors (R) are prescribed for the cases listed in Table 2-2.

Table 2-2 – Response Modification Factors (R) *Division I-A Seismic Design* of the current 17th Edition of the *Standard Specifications*

Substructure	R	Connections	R
Wall-Type Pier	2	Superstructure to abutment	0.8
Reinforced concrete pile bents		Expansion joints within a span of the superstructure	0.8
a. Vertical piles only	3		
b. One or more battered piles	2		
Single columns	3	Columns, piers, or pile bents to cap beam or superstructure	1.0
Steel or composite & steel concrete pile bents		Columns or piers to foundation	1.0
a. Vertical piles only	5		
b. One or more battered piles	3		
Multiple-column bent	5		

In a great departure from the earthquake resistant design of buildings in which the response modification factor is assigned for the structure as a whole by dividing the design forces by R before performing the analysis; in the *ATC-6* document the analysis, static or dynamic, is performed for the bridge without reducing the design forces by the response modification factor. Here it is used at the element level by dividing the design forces by R just when computing modified values (EQM) that replace values of (EQ) in Eq. (2-5). This form of use for the response modification factor in bridges has been kept up to *Division I-A Seismic Design* of the current 17th Edition of the *Standard Specifications*.

Other major departure from previous bridge design practice introduced by the *ACT-6* document was to assign differential detailing requirements depending on the Seismic Performance Categories (SPC). This leads to increased structural toughness from

category A to D, with category A corresponding to the *Standard Specifications* requirements without any seismic provisions beyond using seismic forces in analysis and design and no special detailing. The reinforced concrete requirements for SPC B to D were based on corresponding requirements of the ACI 318 Code [ACI, 1977] for buildings. This may be considered a landmark in improved expected performance of transportation structures subjected to strong earthquake ground motion.

Other significant improvement with respect to previous practice was the introduction of dimension of minimum support-width length to be used at expansion ends of girders in all bridges; regardless of seismic performance category and number of spans (see Fig. 2-18). Prescribed minimum support lengths depend on angle of skew of the support, distance between expansion joints, pier height, and seismic performance category.

FIGURE 3.10 Dimensions for Minimum Support Length Requirements

Fig. 2-18 – Minimum support length definition (*Division I-A Seismic Design* of the current 17th Edition of the *Standard Specifications*)

AASHTO LRFD

Reinforced concrete design departed from the working-stress design (WSD) philosophy in the mid 1950's and by the early 1970's had moved completely into ultimate strength design (USD) philosophy. Other materials lagged but by the mid 1980's

a push for probabilistic design methods coined a new term for USD as load and resistance factor design (LRFD) which was adopted especially by the steel industry and later by AASHTO. Between 1988 and 1993 with the sponsorship of the Federal Highway Administration under the National Cooperative Highway Research Program (NCHRP) developed and published in 1994 the first edition of *AASHTO LRFD Bridge Design Specifications*, reviewed later for a second edition in 1998 [AASHTO, 1998], and recently in a third edition in 2004 [AASHTO, 2004].

The AASHTO LRFD Bridge Design Specifications, First (1994), Second (1998), and Third (2004) Editions, requires that each bridge component and connection satisfy all limit states in accordance with the relation

$$\eta \sum (\gamma_i \cdot Q_i) \leq \phi \cdot R_n \quad (2-7)$$

in which η is a factor related to a ductility factor η_D , a redundancy factor η_R , and an operational importance factor η_i in accordance with $\eta = \eta_D \cdot \eta_R \cdot \eta_i$; γ_i is a statistically-based load factor applied to force effect Q_i ; and ϕ is a statistically-based resistance factor applied to the nominal resistance R_n . The numerical values to be used for these factors can be found in the LRFD Specifications (AASHTO LRFD, 1994, 1998, and 2004).

The value of Q_i for the value of i representing an extreme seismic event, designated EQ, is found using the same procedure described above for *Standard Specifications*, Fifteenth (1992), Sixteenth (1996), and seventeenth (2002) Editions.

An additional bridge classification, "Critical," has been added to the LRFD Specifications; and the number of substructure response modification factors R , have been increased and made different in all three classifications, "Critical," "Essential," and "Other" as indicated in Table 2-3. The response modification factors for connections remained the same in the *LRFD Specifications* than in the *Standard Specifications*.

Although, at first glance it may look that the impact of adoption of *AASHTO LRFD Specification* to replace the *AASHTO Standard Specifications* with respect to seismic design is minor because they are based on the same requirements, this is not true for bridges in the "Critical" and "Essential" categories where seismic design forces described

by EQM (EQ/R) increase by a factors ranging from 1.33 to 3.33 for “Critical” bridges and from 1.33 to 1.5 for “Essential” bridges.

Table 2-3 – Response Modification Factors (R) in *AASHTO LRFD Specifications*

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers-larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
• Vertical piles only	1.5	2.0	3.0
• One or more battered piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite & steel concrete pile bents			
• Vertical piles only	1.5	3.5	5.0
• One or more battered piles	1.5	2.0	3.0
Multiple-column bent	1.5	3.5	5.0

The presented material describes current *AASHTO Specifications* both *Standard* and *LRFD*. Notwithstanding, several important changes on seismic bridge specifications are being discussed in the corresponding drafting committees and it is important to be aware of their implications, if approved. These changes come basically from three sources:

- the first one corresponds to changes in the California Department of Transportation (Caltrans) seismic design criteria,
- the second one from changes in the way to define the design ground motion both in the description of the movement at rock by the U.S. Geological Survey (USGS) that is discussed in Chapter 3, and
- the third in the way to assess the amplification caused by the soil profile by the Building Seismic Safety Council (BSSC) as part of the National Earthquake Hazard Reduction Program (NEHRP) for incorporation in the NEHRP Recommended Provision for Seismic Regulations for New Buildings that is discussed in 2.3.1.

Some of these new trends have already been adopted as regulations in the corresponding earthquake resistant design requirements for new buildings. A brief description of these trends follows.

Caltrans Seismic Design Criteria

In the aftermath of the Loma Prieta Earthquake of 1989, the California Department of Transportation (Caltrans) decided to sponsor a project to conduct a critical review and to recommend changes where needed in the seismic design part of the Caltrans Bridge Design Specifications (BDS) also known as the California Seismic Design Criteria (SDC). The result of this project was presented in document ATC-32 [ATC, 1996] which provided a number of recommended improvements to seismic bridge design practice in California. The project focused on issues related to seismic loading, structural response analysis, and component design. Special attention was given to design issues related to reinforced concrete components, steel components, foundations, and conventional bearings. In addition, the specifications were revised to give a bigger role to the evaluation of displacement and to use it as a tool to define performance criteria. Although a new definition of the design ground motion was contained, the production of new seismic risk maps was considered then outside the scope of the project.

The ATC-32 was, as the title indicated, a recommendation. It took some time for Caltrans to update the *Caltrans SDC*. In July 1999 Version 1.1 of the *Caltrans SDC* was published [Caltrans, 1999], with Version 1.2 in December of 2001 [Caltrans, 2001], and Version 1.3 in February 2004 [Caltrans, 2004].

The *Caltrans SDC* adopts a performance-based approach specifying minimum levels of structural system performance, component performance, analysis, and design practices for ordinary standard bridges. For bridges with non-standard features or operational requirements above and beyond the ordinary standard bridge a greater degree of attention than specified by the SDC is required. Many of the methodologies contained in the SDC evolved from the seismic retrofit program and some of the procedures are major departures from previous practice while others are slight modifications to current practice. The most significant change in design philosophy for new bridges is a shift from

a force-based assessment of seismic demand to a displacement-based assessment of demand and capacity. The displacement approach is based on comparing the elastic displacement demand to the inelastic displacement capacity of the primary structural components while insuring a minimum level of inelastic capacity at all potential plastic hinge locations.

The SDC document [Caltrans, 2004] warns about its applicability in places different from California: *“This document is intended for use on bridges designed by and for the California Department of Transportation. It reflects the current state of practice at Caltrans. This document contains references specific and unique to Caltrans and may not be applicable to other parties either institutional or private.”* One important aspect that makes its direct applicability elsewhere difficult is the way the earthquake design ground motion is defined. California has unique tectonic characteristics with a relatively thin earth’s crust producing predominantly shallow earthquakes. The fault mapping of California is comprehensive; leading to definition of design ground motion supported on relatively well-known faults that lend themselves to establish earthquake mean recurrence periods with somewhat less uncertainty than in other places in the U.S. (this will be discussed later in Chapter 3 with respect to the implication for the State of Indiana). This means that the design ground motion proposed by *Caltrans SDC* is particular for the tectonic setting and the types of soil profiles common in California. Other important aspect is that the *Caltrans SDC* is focused on concrete bridges. The *Caltrans SDC* specifies target ductility demands, defined as the ratio of the estimated global frame displacement demand to the yield displacement of the subsystem from its initial position to the formation of plastic hinge, for different structural components. For instance, for multi-column bents a value of 5 is prescribed while for wall piers in the strong direction a value of unity ($= 1$) is given. The structure must be designed to resist the internal forces generated when the structure reaches its Collapse Limit State. The Collapse Limit State is defined as the condition when a sufficient number of plastic hinges have formed within the structure to create a local or global collapse mechanism.

The local displacement capacity of a member is based on its rotation capacity (see Fig. 2-19 for cantilever column and Fig. 2-20 framed columns), which in turn is based on

its curvature capacity. The curvature capacity must be determined from a moment-curvature ($M-\phi$) analysis. Each ductile member shall have a minimum local displacement ductility capacity of 3 to ensure dependable rotational capacity in the plastic hinge regions regardless of the displacement demand imparted to that member.

Fig. 2-19 – Local displacement capacity – cantilever column with fixed base
(*Caltrans Seismic Design Criteria* [Caltrans, 2004])

Fig. 2-20 – Local displacement capacity – framed column assumed as fixed-fixed
(*Caltrans Seismic Design Criteria* [Caltrans, 2004])

Caltrans SDC defines how the expected material properties can be obtained, how to compute the plastic moment capacity for ductile concrete members, how to perform the moment curvature ($M-\phi$) analysis, and how to obtain the shear capacity of the

components. The designer is required to check that the nominal capacity of the superstructure longitudinally and of the bent cap transversely are sufficient to ensure that the columns respond in the nonlinear range prior to the superstructure or bent cap reaching its expected nominal strength. *Caltrans SDC* requires assessing the force and deformation demands and capacities on the structural system, and its individual components, using analysis. The document prescribes for ordinary standard bridges the use of equivalent static analysis and linear elastic dynamic analysis for estimating the displacement demands, and inelastic static analysis (push-over) for establishing the displacement capacities. The *Caltrans SDC* document gives guidance with respect to the use of appropriate cracked sections for each type of analysis.

The geotechnical engineer is required to provide the following information relative to the bridge site: seismicity, fault distance, earthquake magnitude, peak rock acceleration, soil profile type, liquefaction potential, and foundation stiffness or the soil parameters necessary for determining the force deformation characteristics of the foundation. The document contains standard acceleration response spectra for preliminary design, but in general requires a site specific spectrum for design. The standard acceleration and displacement response spectra provided are function of soil profile type and design earthquake magnitude. Magnitude ranges from 6.5 to 8, and envisioned peak ground accelerations in rock as high as $0.7g$ are presented. Acceleration amplification caused by the soil profile follows latest BSSC and NEHRP recommendations (see 2.3.1). The foundation subsystem response must be evaluated based on the quality of the surrounding soil. Soils are classified as competent, poor, or marginal.

In the *Caltrans SDC* the bridge designer is required to maintain the ratio of effective stiffness between any two bents within a frame or between any two columns within a bent within prescribed limits. Hinge seat width are prescribed to accommodate the anticipated thermal movement, prestress shortening, creep, shrinkage, and the relative longitudinal earthquake displacement demand between adjacent frames. As a rule adequate seat width must be provided to prevent unseating as a primary requirement. Hinge restrainers are considered secondary members to prevent unseating. Moment resisting connections between the superstructure and the column must be designed to

transmit the maximum forces produced when the column has reached its flexural and shear overstrength capacity. *Caltrans SDC* considers bearings as sacrificial elements for ordinary standard bridges. Typically bearings must be designed and detailed for service loads. However, bearings must be checked to insure their capacity and mode of failure are consistent with the assumptions made in the seismic analysis. The designer must detail bearings so they can be easily inspected for damage and replaced or repaired after an earthquake. Columns flares require special treatment to insure proper behavior of the column. At the foundation, the size and number of piles and the pile group layout must be designed to resist service level moments, shears, and axial loads and the moment demand induced by the column plastic hinging mechanism. The linear elastic demand analysis model must include an effective abutment stiffness that accounts for expansion gaps, and incorporates realistic values of the embankment fill response. Seat type abutments must be designed to resist elastically all transverse service load and moderate earthquake demands.

Draft AASHTO LRFD Update

The *Caltrans SDC* imposed a trend that was picked up by the National Cooperative Highway Research Program (NCHRP) for incorporation in an update of the *AASHTO LRFD Specifications*. A research project sponsored by AASHTO in cooperation with the Federal Highway Administration (FHWA) produced a draft update of the AASHTO LRFD Specifications that is summarized and described in NCHRP Report 472 [NCHRP, 2002] and presented in final form in four reports published by the Applied Technology Council: “Recommended LRFD guidelines for the seismic design of highway bridges – Part I: Specifications” [ATC/MCEER, 2003a], “Recommended LRFD guidelines for the seismic design of highway bridges – Part II: Commentary and Appendices” [ATC/MCEER, 2003b], “Liquefaction Study Report for Recommended LRFD guidelines for the seismic design of highway bridges” [ATC/MCEER, 2003c], and “Design Examples for Recommended LRFD guidelines for the seismic design of highway bridges” [ATC/MCEER, 2003d]. The research was performed by a joint venture of the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake

Engineering Research (MCEER), Buffalo, NY. The project purpose was to develop new specifications for the seismic design of bridges, considering all aspects of the design process. These aspects include the following: (1) design philosophy and performance criteria, (2) seismic loads and site effects, (3) analysis and modeling, and (4) design requirements. The new specification should be nationally applicable with provisions for all seismic zones. The emphasis was on design of new bridges rather than on retrofit of existing ones. The *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* contains a number of new concepts and additions, as well as some major modifications to existing provisions. These are summarized in the following paragraphs.

The definition of the design earthquake has been moving away from the 10% probability of exceedance in a 50-year life span of the structure towards a definition of a maximum considered earthquake with a lower (3%) probability of exceedance and sometimes a longer life span (75 years). On the other hand, the use of effective peak ground acceleration as the mapped parameter to define the design ground motion has been substituted in the specifications for design of new buildings for a direct definition of the design response spectrum including the site effects. The USGS has developed maps for these new definitions as will be explained in Chapter 3 of this report. The *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* uses these new definitions and maps.

The proposed provisions provide performance objectives and damage states for two design earthquakes with explicit design checks to ensure that the performance objectives are met (see Table 2-4).

In Table 2-4 the upper-level event, termed the “rare” or Maximum Considered Earthquake (MCE), describes ground motion that, for most locations, is defined probabilistically and has a probability of exceedance of 3% in 75 years (approximately 2500-year mean return period). The frequent or expected earthquake has a probability of exceedance of 50% in 75 years (approximately 100-year mean return period). For the service level: “Immediate” means full access to normal traffic and the bridge must be available for traffic after an inspection, and “Significant Disruption” means limited access (reduced lanes, light emergency traffic) may be possible after shoring but the

bridge may need to be replaced. Detailed geometric constraints on service level are presented in the draft. For the damage levels: “None” means evidence of movement may be present but no notable damage; “Minimal” means some visible signs of damage produced by minor nonlinear response (narrow flexural cracking), no permanent deformations, and repair can be performed under non-emergency conditions, “Significant” means no collapse, but permanent offsets, cracking, reinforcement yielding, and major spalling of concrete may require closure for repairs. Partial or complete replacement of columns may be required in some cases.

Table 2-4 – Design Earthquakes and Seismic Performance Objectives in the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)*

Probability of exceedance for design earthquake ground motion		Performance level	
		Life Safety	Operational
Rare earthquake (MCE) 3% in 75 years	Service	Significant Disruption	Immediate
	Damage	Significant Disruption	Minimal
Frequent or expected earthquake 50% in 75 years	Service	Immediate	Immediate
	Damage	Minimal	Minimal to none

The design ground motion in *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* is described through a design response spectrum that can be obtained from the general procedure or from a site-specific study. The site-specific study is required if the bridge is considered a major or very important structure, when the site is located within 10 km of an active fault, or when the site has a Type F soil profile (see 2.3.1) with peats or highly organic soils, high plasticity clays, or very thick deposits of soft to medium stiff clays.

The general procedure defines response spectra for the rare MCE earthquake and for the frequent earthquake based on maps produced (or being produced in the case of the frequent earthquake) by the USGS. The acceleration spectrum for 5% of critical damping is obtained using a two point procedure. The spectral design acceleration at 0.2-second

period, S_{DS} , and at 1-second period, S_{D1} , are obtained from Equations (2-8) and (2-9) respectively.

$$S_{DS} = F_a \cdot S_S \quad (2-8)$$

$$S_{D1} = F_v \cdot S_1 \quad (2-9)$$

where S_S and S_1 are the 0.2-second and 1-second period spectral accelerations on rock (Class B site, see 2.3.1) from ground motion maps and F_a and F_v are site coefficients associated with the site class and mapped acceleration value. The maps for the Maximum Considered Earthquake contained in the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* are the same maps produced by the USGS for BSSC to be included in the 1997 NEHRP recommendations for new buildings [NEHRP, 1997]. These maps are presented and discussed in Chapter 3. For the frequent earthquake there were no maps available when the draft was produced and the draft suggests obtaining them by interpolation from the hazard curves produced by the USGS compatible with the maps for the MCE. The design spectrum is obtained from the values given by Equations (2-8) and (2-9). Two control periods are given: T_s and T_0 defined in Equations (2-10) and (2-11) respectively. Figure 2-21 shows the design spectrum construction using the two-point method.

$$T_s = \frac{S_{D1}}{S_{DS}} \quad (2-10)$$

$$T_0 = 0.2 \cdot T_s \quad (2-11)$$

For periods less than or equal to T_0 , the design response spectral acceleration is defined by Eq. (2-12). Note that as T tends to $T = 0$ seconds, the resulting value of S_a is equal to the peak ground acceleration (PGA).

$$S_a = 0.60 \cdot \frac{S_{DS}}{T_0} \cdot T + 0.4 \cdot S_{DS} \quad (2-12)$$

For periods greater than or equal to T_0 and less than or equal to T_s , the design response spectral acceleration, S_a , is defined by Eq. (2-13).

$$S_a = S_{DS} \quad (2-13)$$

For periods greater than T_s the design response spectral acceleration, S_a , is defined by Eq. (2-14).

$$S_a = \frac{S_{D1}}{T} \quad (2-14)$$

Fig. 2-21 – Design Response Spectrum – Probabilistic general procedure
(Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49))

When a site-specific response spectrum is used, a probabilistically based spectrum must be obtained using a characterization of the seismic sources and ground motion attenuation that includes current scientific interpretations, including uncertainties in seismic source and ground motion models and parameter values. If the spectrum obtained this way exceeds the limits shown in Fig. 2-22, a deterministic spectrum may be used.

The deterministic design response spectrum may be used in regions having known active faults if the deterministic spectrum ordinates are less than those of the probabilistic spectrum. The deterministic spectrum must be the median-plus-standard-deviation spectrum calculated for characteristic maximum magnitude earthquakes on known active faults (the corresponding building requirements require 150% instead of the 184% for bridges), but must not be less than the spectrum shown in Fig. 2-22. If there is more than one active fault in the site region, the deterministic spectrum must be computed as the

envelope of spectra for the different faults. Alternatively a deterministic spectrum may be computed for each fault, and each spectrum, or the spectrum that govern bridge response, may be used for the analysis of the bridge. When the design response spectrum is determined from a site-specific study, the spectrum ordinates must not be less than 2/3 of the ordinates of the probabilistic spectrum obtained following the general procedure. The draft permits also the use of acceleration time histories instead of a design response spectrum and provides guidance on how to define them. Design for vertical acceleration effects are prescribed in detail for bridges located less than 50 km from an active fault.

Fig. 2-22 – Minimum Deterministic Design Response Spectrum
(Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49))

The *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* defines a Seismic Hazard Level (SHL) classification into four groups (I to IV). The bridge must be assigned in the larger SHL obtained from Table 2-5 or 2-6.

Table 2-5 – Seismic Hazard Level based on $F_v S_1$
Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)

Seismic Hazard Level	Value of $F_v S_1$
I	$F_v S_1 \leq 0.15g$
II	$0.15g < F_v S_1 \leq 0.25g$
III	$0.25g < F_v S_1 \leq 0.40g$
IV	$0.40g < F_v S_1$

Table 2-6 – Seismic Hazard Level based on $F_a S_S$
Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)

Seismic Hazard Level	Value of $F_a S_S$
I	$F_a S_S \leq 0.15g$
II	$0.15g < F_a S_S \leq 0.35g$
III	$0.35g < F_a S_S \leq 0.60g$
IV	$0.60g < F_a S_S$

The Seismic Design Level is used, in turn, to define the Seismic Design and Analysis Procedure (SDAP) and the Seismic Detailing Requirements (SDR). Table 2-7 defines the Seismic Design and Analysis Procedure.

Table 2-7 – Seismic Design and Analysis Procedure (SDAP)
Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)

Seismic Hazard Level	Life Safety	Operational
I	A1	A2
II	A2	C/D/E
III	B/C/D/E	C/D/E
IV	C/D/E	C/D/E

Brief description of the different Seismic Design and Analysis Procedures (SDAP):

- SDAP A — Observance of minimum seat widths, checking of connection design forces for bearings, and use of minimum shear reinforcement in concrete piles are the only requirements. The horizontal connection design force in the restrained direction must be 10% of the vertical reaction for SDAP A1 and 25% for SDAP A2. No rigorous analysis is required instead these default values are used as design forces.
- SDAP B — Does not require a seismic demand analysis. Capacity design principles and minimum design detailing are required. There are certain restrictions to use SDAP B associated with span length (maximum 80 feet), skewness of the bridge (maximum 30°), bent stiffness, and element cross-section dimension and reinforcement, and other.
- SDAP C — Requires the use of the capacity design spectrum approach. This procedure applies only to bridges that behave and can be modeled as a single-

degree-of-freedom system. The procedure is also restricted to bridges with a very regular configuration, bridges not having more than six spans, spans not greater than 60 feet, maximum skewness of 30°, and other.

- SDAP D — Requires the use of the elastic response spectrum method. Cracked sections are required. The procedure is divided into a single-mode equivalent lateral load procedure, and a multimode dynamic analysis procedure.
- SDAP E — Consist in an elastic response spectrum analysis used for design plus a displacement capacity verification performed afterwards.

Response modification factor, R_B , to be used in SDAP D and E are presented in Table 2-8 for the substructure. Response modification factor for connections are all $R = 0.8$ but are not intended for cases where the design of the connection is made using capacity design principles.

Table 2-8 – Base Response Modification Factors, R_B , for the substructure
Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)

Substructure Element	Performance Objective			
	Life Safety		Operational	
	SDAP D	SDAP E	SDAP D	SDAP E
Wall pier – larger dimension	2	3	1	1.5
Columns – Single and multiple Ductile detailing	4	6	1.5	2.5
Pile bents and drilled shafts – vertical piles – above ground	4	6	1.5	2.5
Pile bents and drilled shafts – vertical piles – 2 diameters below ground level – no owner’s approval required	1	1.5	1	1
Pile bents and drilled shafts – vertical piles – in ground – owner’s approval required	N/A	2.5	N/A	1.5
Pile bents with batter piles	N/A	2	N/A	1.5
Seismically isolated structures	1.5	1.5	1	1.5
Steel braced frame – ductile	3	4.5	1	1.5
Steel braced frame – Nominally ductile	1.5	2	1	1
All elements for frequent or expected earthquake	1.3	1.3	0.9	0.9

The value of R must be obtained from Eq. (2-15)

$$R = 1 + (R_B - 1) \cdot \frac{T}{T^*} \leq R_B \quad (2-15)$$

where $T^* = 1.25 T_s$ and T_s is defined in Eq. (2-10).

Table 2-9 defines the Seismic Detailing Requirements (SDR) for each Seismic Hazard Level (SHL) and Table 2-10 defines the component detailing requirements for each SDR.

Table 2-9 – Seismic Detailing Requirements (SDR)
Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)

Seismic Hazard Level	Life Safety	Operational
I	1	2
II	2	3
III	3	4
IV	5	6

Minimum seat width must not be less than 1.5 times the displacement of the superstructure at the seat including P-Δ effects, Δ_m , obtained using Eq. (2-16), but can not be less than the value of N obtained from Eq. (2-17):

$$\Delta_m = R_d \cdot \Delta$$

$$R_d = \begin{cases} \left(1 - \frac{1}{R}\right) \cdot \frac{T^*}{T} + \frac{1}{R} & \text{for } T < T^* \\ 1 & \text{for } T \geq T^* \end{cases} \quad (2-16)$$

$$N = \left[0.1 + 0.0017L + 0.007H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(2\frac{B}{L}\right)^2} \right] \cdot \frac{1 + 1.25F_v S_1}{\cos \alpha} \geq 1.25 \cdot \Delta_m \quad (2-17)$$

where,

Δ = displacement demand from the seismic analysis

Δ_m = nonlinear displacement demand

R_d = displacement amplification factor

L = distance between joints in m

H = height of the tallest pier between the joints in m
 B = width of the superstructure in m
 α = skew angle

In Eq. (2-17) the ratio B/L need not be taken greater than 3/8.

The proposed LRFD design provisions contain an incentive from a design and construction perspective for performing a more sophisticated “pushover analysis” [NCHRP, 2002]. The response modification factor (R) increases approximately 50 percent when a pushover analysis is performed, primarily because the analysis results will provide a greater understanding of the demands on the seismic resisting elements. The analysis results are assessed using additional plastic rotation limits on the deformation of the substructure elements to ensure adequate performance.

The proposed LRFD provisions provide a mechanism to permit the use of some seismic resisting systems and elements — termed earthquake resisting systems (ERS) and earthquake resisting elements (ERE) — that are not permitted in current AASHTO provisions. Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. The classification of ERS and ERE into three categories: permissible, permissible with owner’s approval, and not recommended, is done to trigger due consideration of seismic performance that leads to the most desirable outcome.

The state of the art in earthquake resistant bridge design has been presented. The more recent changes in mandatory and proposed specifications derive from behavior experiences during recent earthquakes; from the intensive seismic retrofit programs in some states, specially in the West Coast; and from recent research in seismic behavior of bridges.

Table 2-10 – Component Detailing Requirements for SDR's — *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)*

Component		SDR 1	SDR 2	SDR 3	SDR 4	SDR 5	SDR 6
Seat width		Use Eq. (2-17)					
Bearing	Conventional	0.10 · DL	0.25 · DL	Capacity design procedure or elastic forces with $R = 0.8$			
	Isolation	Detailed and tested for 1.1 times 5% in 75-years forces and displacements					
Column	Flexure	Non-seismic requirements		SDAP B and C – non-seismic or minimum steel or P-Δ or 50% in 75-years forces for C. SDAP D and E – Moment demand divided by R or minimum steel or P-Δ			
	Shear	Non-seismic requirements	Minimum shear reinforcement	From capacity design procedures or elastic forces with $R = 0.67$			
Connection of column to superstructure, bent beam, footing/pile cap		N/A except bearing above	N/A except bearing above	Design forces from capacity design using over-strength ratios of 1.5 and 1.4 for concrete and steel respectively			
Soil and pile aspects of foundation design	Concrete	N/A	Piles top 3D minimum reinforcement	Design forces from capacity design using over-strength factor of 1.0	Design forces from capacity design and overstrength factor of 1.5 and 1.4 for concrete and steel respectively. Minimum shear reinforcement apply		
	Steel	N/A	N/A				
Pile bents	Concrete	N/A	Min. shear reinf. down to 10D below surface	Design forces from capacity design using over strength ratio of 1.5 and 1.4 for concrete and steel respectively. Minimum shear reinforcement as per SDR 2.			
	Steel	N/A	N/A				
Abutments		N/A	N/A	SDAP B and C – non-seismic req. SDAP D and E – Detailed req.	Detailed requirements		
Liquefaction		Only if requested by owner		If $M_w < 6$ not required. If > 6 full procedure	Full procedure		
ESR/ERE		N/A	N/A	Use procedure	System requiring owner’s approval not permitted		
Approach/settlement slab		N/A	N/A	N/A	Encouraged but not mandated	N/A	Required

Bridge retrofit

The leader in seismic retrofit programs has been the State of California. There are more than 12,000 bridges in the California State Highway system, plus an additional 12,000 city and county bridges. Since the 1971 San Fernando earthquake struck the Los Angeles area Caltrans has been engaged in an ongoing bridge retrofit program. Initially, Caltrans Seismic Retrofit program consisted of restraining sections of 1,262 bridges with steel cables. The Seismic Retrofit program now involves strengthening the columns of existing bridges by encircling certain columns with a steel casing or, in a few instances, an advanced woven fiber casing. In addition to the column casing, some of the bridge footings are made bigger and given more support by placing additional pilings in the ground or by using steel tie-down rods to better anchor the footings to the ground. In a few projects bridge abutments are made larger and the existing restrainer units are made stronger because encasing the columns make them stiffer and can change the way forces are transmitted within the bridge. Many Seismic Retrofits involve "hinge seat extensions" which enlarge the size of the hinges that connect sections of bridge decks and helps prevent them from separating during severe ground movement.

The Seismic Retrofit program was divided into Phase 1 and Phase 2. Phase 1 included 1,039 bridges identified for strengthening after the Loma Prieta quake. Phase 2 identified an additional 1,364 bridges for strengthening following the January 1994 Northridge earthquake. Currently the state's bridge earthquake strengthening program will involve approximately 2,200 additional structures, including the state's toll bridges. In addition a total of 1,114 city and county-owned bridges have been identified as candidates for seismic strengthening.

Washington State has carried similar programs. Between 1999 and 2001 more than 350 bridges were retrofitted statewide. After the 2001 Nisqually, Washington State, Earthquake the scope of the retrofit program was expanded.

Since 1983 the Federal Highway Administration has published three documents on seismic retrofit of highway bridges [FHWA, 1983, 1987, and 1995]. The latest, 1995 document, is compatible with current *Division I-A of the AASHTO Standard*

Specifications [AASHTO, 2002]. This document covers procedures for preliminary screening of the bridges, detailed evaluation, and design of retrofit measures.

Fig. 2-23 – Caltrans Seismic Retrofit Program Poster

The preliminary screening – Chapter 2 – emphasizes the need of developing an inventory of bridge seismic information accompanied by a seismic rating system, and gives guidance for establishing them. Using the vulnerability rating a priority index of retrofit is computed.

The detailed evaluation guidelines – Chapter 3 – define the different evaluation methods starting with the analysis procedures: analysis procedures for capacity/demand method, and analysis procedure for lateral strength method. Then, gives detailed requirements for performing a capacity/demand evaluation of the bridge, and establishes

the principles for applying the lateral strength method. Using the results of the two procedures an overall assessment of the bridge components is described including assessment of: bearings and expansion joints; columns, walls, and footings; abutments; and the possibility of liquefaction. Potential retrofit measures are discussed for each of the assessed components.

The rest of the document [FHWA, 1995] is devoted to detailed retrofit measures for the assessed components including: Chapter 4 – discusses seismic retrofit strategies including conventional and innovative earthquake protective systems; Chapter 5 – deals with the retrofit of bearings, seats, and expansion joints; Chapter 6 – covers retrofit of columns, cap beams, and joints; Chapter 7 – discusses the retrofit measures for foundations including footings and abutments; Chapter 8 – covers retrofit measures for bridges on hazardous sites including bridges across or near active faults and bridges on liquefiable soils discussing potential remedial solutions for these cases ; and Chapter 9 – discusses protective measures using seismic isolation. The appendices cover: (A) detailed indication for obtaining capacity/demand ratios for bridge components, (B) the assessment of members strength and deformation capacity, and (C, D, and E) worked examples for two bridges and the use of cable restrainers.

Although this document was developed a few years ago, it covers all the basic procedures. This document [FHWA, 1995] combined with some of the detailed concepts covered by the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] makes an excellent seismic bridge retrofit guide that is totally up-to-date.

2.3 Geotechnical issues

In the previous section when discussing the evolution of the bridge seismic design criteria the parameters associated with geotechnical issues were presented without explaining and discussing their background. The following sections present the corresponding background and relevant information that that will be needed for the assessment for geotechnical issues of the earthquake emergency routes of Indiana as presented in Chapter 4.

2.3.1 Ground motion amplification due to soil profile characteristics

Association of earthquake damage with poor soil conditions have been known for many years. Early approach to the problem was manifest in seismic codes through a difference in seismic design forces associated with bearing capacity used in foundation design (see 2.2.2). With the increase deployment of strong ground motion accelerometers from the 1950's on, instrumental evidence of the influence of the underlying soil profile permitted interpretations of earthquake damage in several events such as the 1957 San Francisco, California, Earthquake; the 1957 Mexico City Earthquake; the 1964 Anchorage, Alaska, Earthquake; and the 1967 Caracas, Venezuela, Earthquake.

From the mid 1970's on, a clearer picture emerged and different approaches were adopted to account for the effect of amplification by the soil profile. For the first time in the 1974 SEAOC Blue Book Lateral Force Requirements for Buildings [SEAOC, 1974] included a coefficient S , called then Coefficient for Site-Structure Resonance having values between 1.0 and 1.5 depending on the ratio between the fundamental period of the structure, T , and a characteristic site period T_s . The 1974 SEAOC Requirements included an Appendix for obtaining a range of values for the soil deposit characteristic period. This procedure was adopted by the 1976 Uniform Building Code and was kept for several issues of this Code.

Studies conducted for moderate magnitude events in the western U.S. [Seed et al., 1976a] permitted grouping different soil profiles into broad categories [Seed et al., 1976b]. This type of soil profiles and the corresponding amplifications were combined with studies of spectral shapes and were included in the ATC-3-06 project [ATC, 1978] for incorporation in model building earthquake resistant regulations. This classification comprised three types of soil profiles namely: Type 1 – Stiff soils and rock, Type 2 – Deep cohesionless or stiff clay soils, and Type 3 – Soft to medium clays and sands. As part of the requirements a soil coefficient, S , was introduced. This coefficient amplified the medium to long period part of the acceleration spectrum in rock to model the effects of the soil according to the soil profile type by factors of 1.0, 1.2, and 1.5 respectively. This is the same soil profile classification that was used in the seismic bridge design recommendation ATC-6 project [ATC, 1981].

During the 1985 Mexico City Earthquake very large soil amplifications were recorded. This brought the introduction of an additional soil profile type for deep soft clay deposits that was termed Type 4 and was assigned a soil coefficient $S = 2$. With this addition this corresponds to the soil profile classification used when the ATC-6 requirements were modified to be adopted as *Division I-A Seismic Design* of the fifteenth (1992) edition of *AASHTO Standard Specifications*. These requirements have been kept without modification in the sixteenth (1996) and in the current seventeenth (2002) edition [AASHTO, 2002] of the *AASHTO Standard Specifications*. The soil profile definition and soil factor, S , were shown previously in Fig. 2-16.

Information mainly from the 1989 Loma Prieta, California, Earthquake, and the 1985 Mexico City event, in addition to data from other cases in other parts of the world permitted in the early 1990's to restudy the ground motion amplification due to soil profile characteristics and to propose a new classification of soil profile types and soil amplification factors [Whitman, 1992]. These new procedures take into account the level of shaking, rock stiffness, and soil type, and the stiffness and depth effect on the amplification of ground motions at short and long periods. These new soil profile types and soil amplification factors were incorporated into the *1994 NEHRP Provisions* [NEHRP, 1994]. Among the more important changes are larger amplification factors for lower levels of acceleration caused by the soil responding in the linear range with less damping as compared with larger amplitude movement responding in the nonlinear range and inducing greater damping. Some adjustments were introduced in the *1997 NEHRP Provisions* [NEHRP, 1997] with the change of the definition of the design ground motion introduced then, and were kept in the *2000 NEHRP Provisions* [NEHRP, 2000].

The soil site must be assigned into one of six soil profile classes labeled A to F. The definition of the soil profile is based on averaged soil properties for the upper 100 feet (30 m) of soil profile. The properties used for this definition are: average shear wave velocity ($\bar{v}_s$), average standard penetration resistance ($\bar{N}$), average standard penetration resistance for the cohesionless soils only ($\bar{N}_{ch}$), and average undrained shear strength in case of cohesive soils ($\bar{s}_u$). These averages are weighted with respect to each layer thickness. Table 2-11 indicates how these parameters are used to define the soil profile.

Table 2-11 – Site Classification [NEHRP, 2000]

Site class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A Hard rock	>5000 ft/s (> 1500 m/s)	N/A	N/A
B Rock	2500 to 5000 ft/s (760 to 1500 m/s)	N/A	N/A
C Very dense soil or soft rock	1200 to 2500 ft/s (360 to 760 m/s)	> 50	> 2000 psf (> 100 kPa)
D Stiff soil	600 to 1200 ft/s (180 to 360 m/s)	15 to 50	1000 to 2000 psf (50 to 100 kPa)
E Soft soil	< 600 ft/s (< 180 m/s)	< 15	< 1000 psf (< 50 kPa)
	Any profile with more than 10 ft (3 m) of soil having <ul style="list-style-type: none"> • Plasticity Index $PI > 20$, • Moisture content $\omega \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf (25 kPa) 		
F Soils requiring site- specific evaluation	1. Soils vulnerable to potential failure or collapse (liquefiable, quick- or highly-sensitive clays, collapsible weakly cemented soils). 2. More than 10 ft (3 m) of peat and/or highly organic clays. 3. More than 25 ft (7.5 m) of very high plasticity clays ($PI > 75$). 4. More than 120 ft (37 m) of soft to medium clays.		

N/A =Not applicable.

Figure 2-24 shows the values of the coefficient F_a as a function of the site class and the short period maximum considered earthquake spectral acceleration S_s . Figure 2-25 shows

the values of the coefficient F_v as a function of the site class and the 1-second period maximum considered earthquake spectral acceleration S_1 . For sites class F a site-specific procedure must be used for determining the maximum considered earthquake spectral acceleration.

Fig. 2-24 – Values of F_a [NEHRP, 2000]

It is important to note that the reference accelerations for obtaining F_a and F_v , as shown in Figure 2-24 and 2-25, were changed in the *1997 NEHRP Provisions* [NEHRP, 1997] from what was presented originally in the *1994 NEHRP Provisions* [NEHRP, 1994]. In the 1994 Edition the abscissa of Figures 2-24 and 2-25 were mapped values of effective peak ground acceleration A_a (EPA) and acceleration A_v derived from effective peak ground velocity (EPV), respectively, based on a 10% probability of being exceeded in a 50-year life span of the structure — 475 years mean return period —. In the 1997 Edition the definition of the design ground motion was changed to the spectral values of the maximum considered earthquake — 2500 years mean return period — described through S_5 and S_1 respectively, as used in Figures 2-24 and 2-25.

Fig. 2-25 – Values of F_v [NEHRP, 2000]

Although this methodology for evaluation of site effects was originally introduced in documents for earthquake resistant building design, it is as well applicable to bridge design, as recognized in the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002]. This last document requires a site-specific evaluation for Site Class F soils, when the bridge is considered a major or very important structure requiring a higher degree of confidence of meeting the seismic performance objectives, or when the site is within 10 km of an active fault. The *Draft* document includes Appendix 3A containing guidelines for conducting site-specific geotechnical investigations and dynamic response analyses. Appendix 3A gives guidance on the site-specific geotechnical investigation to be performed, the modeling of the soil profile, the selection of the ground motion at bedrock to use in the analysis, the procedure of the analysis, and how to interpret the results.

2.3.2 Liquefaction

Liquefaction is a process caused by the earthquake ground motion vibration by which sediments below the water table temporarily lose strength and behave as a viscous liquid

rather than a solid. The type of sediments most susceptible are clay-free deposits of sand and silts; occasionally, gravel liquefies [Seed and Idriss, 1982]. Seismic waves, primarily shear waves, passing through saturated granular layers, distort the granular structure, and cause loosely packed groups of particles to readjust. This readjustment increases the pore-water pressure between particles if drainage cannot occur. If the pore-water pressure raises to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid producing what is called liquefaction of the soil.

In the liquefied condition, soil may deform with little shear resistance. Deformations large enough to cause damage to structures are called ground failures. The ease with which a soil can be liquefied depends primarily on the looseness of the soil, the amount of cementing or clay between particles, and the amount of drainage restriction. The amount of soil deformation following liquefaction depends on the looseness of the material, the depth, thickness, and extension measured in area of the liquefied layer, the ground slope, and the distribution of loads applied by the structure.

Fig. 2-26 – Damage due to liquefaction on Bridge 002/6s-w
2001 Nisqually, Washington, Earthquake

Liquefaction is restricted to certain geologic and hydrologic environments primarily recent deposited sand and silt in areas with high ground water levels. Generally, the younger and looser the sediment and the higher the water table, the more susceptible the soil is to liquefaction. Sediments most susceptible to liquefaction include Holocene (less than 10,000-year-old) delta, river channel, flood plain, and aeolian deposit, and poorly

compacted fills. Liquefaction has been most abundant in areas where ground water lies close — generally within 30 ft — to the ground surface; few instances of liquefaction have occurred in areas with ground water deeper than 60 ft. Dense soils, including well-compacted fills, have low susceptibility to liquefaction.

Only when liquefaction is accompanied by some form of ground displacement or ground failure it is destructive to the built environment. For engineering purposes, it is not the occurrence of liquefaction that is of prime importance, but its severity or its capability to cause damage. Adverse effects of liquefaction can take many forms. These include: flow failures; lateral spreads; ground oscillations; loss of bearing strength; settlement; and increased pressure on retaining walls.

Flow failures — Is evident by lateral displacement of large masses of soil. Flows may consist on completely liquefied soil or blocks of intact material riding on a layer of liquefied soil. It occurs on sites having loose saturated sands or silts lying on steep slopes.

Fig. 2-27 – Turnagain Heights, Anchorage, a sector approximately 2,600 m by 270 m slid 21 m toward Cook Inlet. Sand lenses liquefied and moved down slope. Slope failure began 2 minutes after the start of ground motion – 1964, Alaska, Earthquake

Lateral spread — Lateral spreads involve lateral displacement of large, surficial blocks of soil as a result of liquefaction of a subsurface layer. Displacement occurs in response to the combination of gravitational forces and inertial forces generated by an earthquake. Lateral spreads generally develop on gentle slopes (most commonly less than 3 degrees) and move toward a free face such as an incised river channel. Horizontal displacements commonly range up to several meters. The displaced ground usually breaks up internally, causing fissures, scarps, horsts, and grabens to form on the failure surface. Damage caused by lateral spreads is severely disruptive. For example, during the 1964 Alaska Earthquake, more than 200 bridges were damaged or destroyed by spreading of floodplain deposits toward river channels. The spreading compressed the superstructures, buckled decks, thrust stringers over abutments, and shifted and tilted abutments and piers [NAS, 1973].

Fig. 2-28 – Damage caused by lateral spreading at Sunset Lake trailer park in Tumwater – 2001 Nisqually, Washington, Earthquake

Loss of Bearing Strength — When the soil supporting the structure liquefies and loses strength, large deformations can occur within the soil which may allow the structure to settle and tip. Conversely, buried tanks and piles may rise buoyantly through the liquefied soil. For example, many buildings settled and tipped during the 1964 Niigata, Japan,

Earthquake. The most spectacular bearing failures during that event were in an apartment complex where several four-story buildings tipped as much as 60 degrees. Apparently, liquefaction first developed in a sand layer several meters below ground surface and then propagated upward through overlying sand layers. The rising wave of liquefaction weakened the soil supporting the buildings and allowed the structures to slowly settle and tip. In many cases, the weight of a structure will not be great enough to cause the large settlements associated with soil bearing capacity failures. However, smaller settlements may occur as soil pore-water pressures dissipate and the soil consolidates after the earthquake. The eruption of sand boils (fountains of water and sediment emanating from the pressurized, liquefied sand) is a common manifestation of liquefaction that can also lead to localized differential settlements.

Fig. 2-29 – Settlement of pile cap in liquefied soil beneath industrial building south of downtown Seattle – 2001 Nisqually, Washington, Earthquake

Increased Lateral Pressure on Retaining Walls — If the soil behind a retaining wall liquefies, the lateral pressures on the wall may greatly increase. As a result, retaining walls maybe laterally displaced, tilt, or structurally fail, as has been observed for waterfront walls retaining loose saturated sand in a number of earthquakes.

Now we turn our attention to the evolution of the engineering procedures to evaluate the liquefaction potential. The number of variables that should be taken into account to properly evaluate the liquefaction potential can be grouped into three broad groups [Seed and Idriss, 1982]: soil properties (dynamic shear modulus, damping characteristics, unit

weight, grain characteristics, relative density, and soil structure), environmental factors (procedure of soil formation, seismic history, geologic history – aging, cementation –, lateral earth pressure coefficient, depth of water table, and effective confining pressure), and earthquake characteristics (intensity of ground shaking, and duration of ground shaking). A combination of in-situ test and laboratory tests of undisturbed samples may be used to obtain properties that can be used to define the liquefaction potential using engineering judgment to properly account for the large intrinsic variability of the measured properties — because of the difficulty of retrieving and testing really undisturbed samples — and unknown or difficult to estimate parameters, specially those associated with the earthquake ground motion.

Because of the noted reasons, since the mid 1970's a methodology termed the “simplified procedure” has evolved to be the accepted standard of practice for evaluating liquefaction resistance of soils. This procedure is based on regressions of data from field observations and laboratory measurements. Recently an update of the procedures involved was made by a panel of experts under the sponsorship of the National Center for Earthquake Engineering and the National Science Foundation [Youd and Idriss, 2001]. The proposed updated procedure constitutes the method of choice in which an all-encompassing procedure is not employed.

The procedure requires the computation of two variables that describe the seismic demand on a soil layer — cyclic stress ratio (*CSR*) — and the capacity of the soil layer to resist liquefaction — cyclic resistance ratio (*CRR*) — respectively. The factor of safety to liquefaction of the soil layer is the obtained as:

$$FS = \frac{CRR}{CSR} \quad (2-18)$$

The cyclic stress ratio, *CSR*, is obtained from:

$$CSR = \left(\frac{\tau_{av}}{\sigma'_{vo}} \right) = 0.65 \cdot \left(\frac{a_{max}}{g} \right) \cdot \left(\frac{\sigma_{vo}}{\sigma'_{vo}} \right) \cdot r_d \quad (2-19)$$

where a_{max} = peak ground acceleration at the ground surface generated by the earthquake; g = acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively; τ_{av} = induced shear stress; and r_d = stress reduction coefficient accounting for flexibility of the soil profile.

For a depth below ground surface z in m, r_d can be obtained from:

$$r_d = \begin{cases} 1.0 - 0.00765 \cdot z & \text{for } z \leq 9.15 \text{ m} \\ 1.174 - 0.0267 \cdot z & \text{for } 9.15 \text{ m} < z \leq 23 \text{ m} \end{cases} \quad (2-20)$$

Four field tests are recommended for routine evaluation of liquefaction resistance CRR : the cone penetration test (CPT), the standard penetration test (SPT), shear-wave velocity (V_s) measurements, and for gravelly sites the Becker penetration test (BPT). Each test has its advantages and limitations with the CPT providing the most detailed soil stratigraphy and robust field-data based liquefaction resistance curves now available. CPT testing should always be accompanied by soil sampling for validation of soil type identification. The SPT has a longer record of application and provides disturbed soil samples from which fines content and other grain characteristics can be determined. Measured shear-wave velocities provide fundamental information on small-strain soil behavior that is useful beyond analyses of liquefaction resistance. V_s is also applicable at sites, such as landfills and gravelly sediments, where CPT and SPT soundings may not be possible or reliable. The BPT test is recommended only for gravelly sites and requires use of rough correlations between BPT and SPT, making the results less certain than other tests. Where possible, two or more test procedures should be applied to assure adequate definition of soil stratigraphy and a consistent evaluation of liquefaction resistance.

When using the standard penetration test (SPT) the blow count must be normalized to take into account the overburden pressure and the hammer energy. This is done by adjusting the blow count values, N , to a normalized value termed $(N_1)_{60}$ that corresponds to an overburden pressure, P_o , of approximately 100 kPa (2 kip/sq ft) and a hammer efficiency of 60%. The cyclic resistance ratio (CRR) is first defined for an ideal clean-sand base curve for magnitude 7.5 earthquakes:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200} \quad (2-21)$$

Equation (2-21) is valid for $(N_1)_{60}$ values less than 30. For greater values clean granular soils are too dense to liquefy and are classified as non-liquefiable. A series of adjustments must be made to $CRR_{7.5}$, as follows:

Fines content adjustment — CRR increase with soil fines content (FC) by adjusting $(N_1)_{60}$ to an equivalent clean sand value $(N_1)_{60cs}$. This is performed using the following empirical equation:

$$(N_1)_{60cs} = \alpha + \beta \cdot (N_1)_{60} \quad (2-22)$$

and α and β are obtained from:

$$\alpha = \begin{cases} 0 & \text{for } FC \leq 5\% \\ e^{\left(1.76 - \frac{190}{FC^2}\right)} & \text{for } 5\% < FC \leq 35\% \\ 5.0 & \text{for } 35\% < FC \end{cases} \quad (2-23)$$

$$\beta = \begin{cases} 1.0 & \text{for } FC \leq 5\% \\ 0.99 + \frac{FC^{1.5}}{1,000} & \text{for } 5\% < FC \leq 35\% \\ 1.2 & \text{for } 35\% < FC \end{cases}$$

Other corrections to SPT — Several corrections in addition to the fines content influence SPT results. They are incorporated by:

$$(N_1)_{60cs} = N_m \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S \quad (2-24)$$

where N_m = measured SPT blows; C_N = factor to take into account the overburden stress; C_E = factor to take into account hammer energy ratio; C_B = factor to take into account the borehole diameter; C_R = factor to take into account the rod length; and C_S = correction for samplers with or without liners. Table 2-12 lists the values for these corrections factors.

Table 2-12 – Corrections to SPT [Youd and Idriss, 2001]

Factor	Equipment variable	Term	Correction
Overburden pressure	—	C_N	$(P_o / \sigma'_{vo})^{0.5}$
Overburden pressure	—	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5 – 1.0
Energy ratio	Safety hammer	C_E	0.7 – 1.2
Energy ratio	Automatic trip Donut	C_E	0.8 – 1.3
Borehole diameter	65-115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	< 3 m	C_R	0.75
Rod length	3 – 4 m	C_R	0.8
Rod length	4 – 6 m	C_R	0.85
Rod length	6 – 10 m	C_R	0.95
Rod length	10 – 30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler w/o liners	C_S	1.1 – 1.3

The recommended procedure [Youd and Idriss, 2001] presents procedures to obtain CRR based on cone penetration test (CPT) and on shear wave velocity (V_s). They were not used in present research due to lack of CPT and V_s information for bridge sites in Indiana with few exceptions [Bobet et al., 2001]. The interested reader is referred to [Youd and Idriss, 2001].

Magnitude correction — An earthquake magnitude scaling factor (MSF) must also be applied using Eq. (2-25).

$$FS = \left(\frac{CRR_{7.5}}{CSR} \right) \cdot MSF \quad (2-25)$$

The magnitude scaling factor, MSF , for moment magnitude, M_w , is presented in Eq. (2-26).

$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \quad (2-26)$$

The proposed *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] requires a liquefaction potential evaluation, as indicated in Table 2-10, for the different component detailing requirements (SDR). For SDR 1 and 2 no evaluation is required unless requested by the owner. For SDR 3 to 6 its must be performed unless requested by the owner or one of the conditions in Table 2-13 are met.

Table 2-13 – Cases for SDR 3 to 6 for which liquefaction potential evaluation need not be performed in *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)*

Magnitude for the 3% probability of exceedance 75-year event	Normalized SPT blow count, $(N_1)_{60}$	$F_a \cdot S_s$
$M_w < 6.0$	N/A	N/A
$6.0 \leq M_w < 6.4$	> 15	N/A
	> 10	$0.375g > F_a \cdot S_s \geq 0.25g$
	> 5	$0.250g > F_a \cdot S_s$

Maps of mean earthquake magnitude are provided. These maps were prepared by the USGS using the maximum magnitude that can affect the site for all relevant seismic sources to produce the design ground motion with 3% of being exceeded in 75 years. If liquefaction potential must be evaluated, the procedures of Appendix 3B must be used.

Appendix 3B is devoted to liquefaction and other geological hazards. It uses the same principles presented [Youd and Idriss, 2001], and gives guidance for making an in depth evaluation of liquefaction potential. The results of the liquefaction assessment are used to evaluate the potential severity of three liquefaction related hazards to the bridge: (1) flow failures involving large slope failures, (2) limited lateral spread, and (3) ground settlement. It permits to evaluate these hazards on the basis of safety factor for liquefaction. If the safety factor is less than 1.0 to 1.3 the potential related hazards must be evaluated following detailed guidelines for each one of them.

In general the procedures included require improving the soil or locating the support foundation elements below the liquefiable layer and to design the bridge to meet the

performance objectives while being subjected to the settlements and/or lateral displacements imposed by the lateral spread.

2.3.3 Fault rupture

The morphology of rivers is associated with geological fault setting in many cases. The possibility of having a bridge located along the path of a geological fault is real. Two recent cases of bridge damage caused by the bridge crossing the fault path are shown in Figures 2-30 and 2-31.

Fig. 2-30 – Collapse of Bei-Feng Bridge located on the earthquake causing fault
Chi-Chi, Taiwan Earthquake, September 21, 1999

Fig. 2-31 – Bolu Viaduct, crossed by the North Anatolian Fault
November 12, 1999 Duzce, Turkey, Earthquake

In many cases the fault existence and location is difficult to detect. The degree of activity of a fault is subjective and estimation the amount of displacements and recurrence period is difficult to forecast. There are very few acceleration records obtained very close to a fault and the expected ground motion is difficult to estimate. Design for these circumstances is challenging and has a high degree of uncertainty in meeting any performance objective. The proposed *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] recommends avoiding bridge construction across faults.

2.3.4 Embankment failure

High embankments in bridge approaches may fail due to different type of problems. Liquefaction may cause lateral spread or settlement, leading to failure of the embankment. The ground motion, without liquefaction occurring, may cause the failure of the slopes. Techniques developed for embankment dams and slope stability may be used to assess the possibility of failure of the road embankment or the bridge approach.

2.3.5 Landslides

Landslides present a significant hazard to roadways in seismically active areas and can pose a hazard for bridges. Damage can be in the form of ground movement either at the abutment or extending to the central piers of the bridge. Bridges located near steep slopes, or places with a history of rock falls, or avalanches.

Fig. 2-32 – Landslide induced by the earthquake. A nearby bridge had major structural damage – Morgan Hill, California, Earthquake of April 24, 1984

Design must include dynamic slope stability analysis or other procedures of stability assessment of nearby slopes, such as Newmark sliding block method. It is important to estimate the order of magnitude of displacement during the landslide.

CHAPTER 3 SEISMIC RISK OF SOUTHWESTERN INDIANA

3.1 Tectonic setting

The great majority of the World's seismic activity occurs at the tectonic plate boundaries. Continental U.S. is contained within the North American Plate with Indiana practically in the middle of it and far away from the plate boundaries. Notwithstanding, earthquakes have been felt in Indiana since colonization began by the French in Louisiana and from the east after the Louisiana Purchase. Intraplate earthquakes can be caused by a number of different kind of stresses — large scale glacial rebound (the slow flexure of the crust back up after a large sheet of ice is removed), for example, or the broad compressional stress caused within eastern North America by the compression forces from the mid-Atlantic spreading center.

In recent decades, earth scientists have collected evidence that strong earthquakes in the central Mississippi Valley have occurred repeatedly in the geologic past. Small earthquakes happen in the region frequently. The area in which most of these quakes occur is referred to as the New Madrid seismic zone (NMSZ) which stretches from just west of Memphis, Tennessee, into southern Illinois. Several times in the past century, moderate earthquakes have been widely felt in southern Illinois and southwestern Indiana. In the last decade geologic evidence that the Wabash Valley faults, initially considered the northern portion of the New Madrid Seismic Zone, is a different system with its own tectonic setting has led to define it as an independent seismic zone that has been named the Wabash Valley Seismic Zone (WVSZ).

The New Madrid Seismic Zone (NMSZ) — Geologic structures related to earthquakes in the central Mississippi Valley region have been deeply buried over hundreds of millions of years by thick layers of sediment. Geophysical studies have revealed a major buried northeast-trending feature known as the Reelfoot Rift [Hildenbrand and Hendricks, 1995], which formed more than 500 million years ago by a process of

extension in the middle of the North American Plate. A rift structure is created when geologic forces begin to pull the Earth's crust apart. If this process continues long enough, the crust separates to form an ocean basin, as happened to form the Atlantic Ocean basin. The Reelfoot Rift is called a failed rift because the Earth's crust did not separate enough to create a new ocean basin. However, the crust was disturbed enough to form major faults that mark the axis and margins of the rift and now contribute to the occurrence of earthquakes in the NMSZ.

Fig. 3-1 – The Reelfoot Rift in the New Madrid Region

A sequence of powerful earthquakes struck the mid-Mississippi River Valley, central United States, in the winter of 1811-1812. The first one occurred December 16, 1811, Intermittent strong shaking continued through March 1812 and aftershocks strong enough to be felt occurred through the year 1817. The initial earthquake of December 16 was followed by two other principal shocks, one on January 23, 1812, and the other on February 7, 1812. Judging from newspaper accounts of damage to buildings, the February 7 earthquake was the biggest of the three. On the basis of the large area of damage (600,000 km²), the widespread area of perceptibility (5,000,000 km²), and the complex physiographic changes that occurred, the Mississippi River valley earthquakes

of 1811-1812 rank as some of the largest in the United States since its settlement by Europeans [Stover and Coffman, 1993]. The estimated magnitudes for these events vary considerably from different sources (7.2 to 8.8 depending on the parameters used to define the magnitude and the scale in which it is reported).

Fig. 3-2 – Isoseismal map for the Arkansas earthquake of December 16, 1811, first of the 1811-1812 New Madrid series [Stover and Coffman, 1993]

The association of the 1811-1812 earthquakes to a causing fault has been hindered by lack of information from a scarcely inhabited region at the time of occurrence. Since the early 1970's a great amount of multidisciplinary research have been devoted to find and define characteristics of the events and the tectonic setting in order to understand the tectonic process taking place and validate seismic risk assessments for the region.

From the information gathered from different sources it is accepted (with dissent) that the causing faults were those shown in Fig. 3-3. The USGS along with the Geological

Surveys of the CUSEC member States and several universities in the region have to a certain degree reached consensus on what is known with certainty and what needs more research to frame adequately the seismic risk assessment for the region. The primary types of evidence are geological — paleoseismological, stratigraphic, geomorphic, structural, and sedimentological information — and seismological, instrumental and historical.

Fig. 3-3 – The New Madrid seismic zone showing the three faults that ruptured in 1811-1812 and earthquakes recorded in recent years [Hough, 2002]

Paleoliquefaction investigations have provided constraints on recurrence in the NMSZ and have been conducted at 44 sites. From these studies it is known that two pre-1811 episodes of liquefaction comparable to that of 1811-1812 can be interpreted [Participants, 2000]. Best estimates of the dates of these events are 1450 and 900 (Fig. 3-4), which leads to a median recurrence interval ranging between 267 and 644 years. The volume of sand mobilized at many of the liquefaction sites in the NMSZ implies they could not have been formed by local events of moderate magnitude. Further indication of

very strong shaking comes from geotechnical measurements at liquefaction sites in the NMSZ, which show the shallow sub-surface materials to be only moderately liquefiable.

Fig. 3-4 – Estimated ages of paleoliquefaction features (vertical axis) arranged by site location, from north (left) to south (right), excluding the Current River sites. Vertical gray shaded segments indicate most probable ages, with brackets showing approximate two standard deviation uncertainties. Shaded horizontal bars indicate the inferred timing of paleoearthquakes. Dashed horizontal line is at 1811-1812. [Participants, 2000]

The paleoliquefaction interpretations also suggest that recurrence rates for events with magnitude on the range of 5 and the largest earthquakes cannot be simply related. Magnitude regressions of the large events for the approximate periods of recurrence implied from the paleoliquefaction record indicate that the mean return period for the smaller magnitude events should be smaller (more frequent). Quoting from [Participants, 2000]: “In short, the largest earthquakes occur much more frequently than the rate of smaller earthquakes would imply. Finally, the paleoliquefaction record suggests that the clustering of earthquakes that occurred in 1811-1812 also occurred in prior events. Multiple units of vented material are evident in many of the sandblows and have been interpreted as resulting from major events that occurred within weeks to months of each other. The lack of soil development, but evidence for bioturbation in materials between the units, constrains the timing between clustered events.”

The length of the active fault system and the lack of relative motion measured across the system make the use of models that work well for earthquakes occurring in plate-boundary settings not appropriate. Models for intraplate tectonics have recently been proposed but their use is just being implemented in the NMSZ. Magnitude estimates of large New Madrid earthquakes rely on uncertain conversions of observations of their effects measured as Modified Mercalli intensities derived mainly from newspapers accounts. It is evident that the spread of intensity is much larger than for comparable events in the West Coast, as shown in Fig. 3-5. The magnitude conversion may be biased by different causes, but one that has been pointed out several times is that soil profile amplification may have increased the reported magnitudes because most towns at the time were settlements along rivers.

Fig. 3-5 – Comparison of areas of damage from the New Madrid and San Francisco Earthquakes

Slip rates for the known faults in the region estimated mainly using characteristics of secondary features show that three episodes of deformation are apparent in folded fluvial deposits overlying the Reelfoot fault with their ages close to the three major events evident in the paleoliquefaction record. The inferred rates of slip are compatible with the short recurrence interval for major earthquakes evident in the paleoseismic record. The

potential for currently classified as non-active faults within and beyond the NMSZ producing moderate-to-large earthquakes remains unknown, although Quaternary surface faulting has been documented on the periphery of the NMSZ [Participants, 2000].

Wabash Valley Seismic Zone — The Wabash Valley Fault System is about 90 km long and 50 km wide in southeastern Illinois, southwestern Indiana, and northwestern Kentucky near the deepest part of the Illinois Basin. The fault system consists of subparallel, high-angle normal faults that have vertical displacements as great as 150 m. The faults bound horsts and grabens, and commonly overlap one another. Major fault plains dip at angles ranging from 50 to 85 degrees. Individual fault blocks are only slightly tilted, and drag is generally absent or weakly expressed.

Fig. 3-6 – Wabash River Valley Faults [René and Stanonis, 1995]

In the past decade, there has been increasing awareness that the seismic hazard in this zone may be greater than the historical earthquake record would suggest. Numerous prehistoric large magnitude earthquakes have struck southern Indiana and Illinois; some may have been larger. Geologic evidence for these earthquakes in the form of sand-and-gravel intrusions in river sediments has been discovered at more than 200 sites in the Wabash River Valley and along its tributaries in Indiana and Illinois. This information is the result of USGS studies done in cooperation with the Indiana Geological Survey, Indiana University, Illinois Department of Natural Resources-Illinois State Museum and the Illinois State Geological Survey. Clastic dikes filled with sand and gravel, interpreted to be the result of earthquake induced liquefaction, occur throughout much of southern Indiana and adjacent parts of Illinois. At least seven and probably eight prehistoric earthquakes have been documented during the Holocene, as well as, at least one during the latest Pleistocene (see Fig. 3-7) [Munson et al., 1997]. Nearly all of these liquefaction features originated from earthquakes centered in southern Indiana and Illinois, and not further south in the nearby source region of the great 1811-12 New Madrid earthquakes [Crone and Wheeler, 2000].

Fig. 3-7 – Map of southern two-thirds of Indiana showing sites where ancient sandblows have been found, and showing areas of liquefaction for six major prehistoric earthquakes. [Kirby, 2001]

In the absence of well-determined data on the timing of paleoevents and the amount of tectonic slip associated with those events, it is impossible to estimate reliable or even meaningful Holocene or late Quaternary slip rates [Crone and Wheeler, 2000]. In summary the tectonics of the Wabash Valley Seismic Zone it not well understood at

present. No historical earthquakes in the Wabash Valley region have been strong enough to cause liquefaction. It is likely that numerous other magnitude 6 to 7 Holocene earthquakes have struck the region, but did not leave a record because of the lack of liquefiable deposits in large parts of the region.

3.2 Earthquake history of Indiana

Earthquakes originated within Indiana and from neighboring seismic zones have been felt in Indiana since the start of the colonization. Figure 3-8 shows the seismicity for the New Madrid and Wabash Valley Seismic Zones. The following description of the seismic history of Indiana was abridged from [Stover and Coffman, 1993].

Fig. 3-8 – Earthquakes in the New Madrid and Wabash Valley, red circles indicate earthquakes that occurred from 1974 to 2002 with magnitudes larger than 2.5, green circles denote earthquakes that occurred prior to 1974 [USGS, 2002]

The most damaging Indiana earthquake originating within the State occurred on September 27, 1909, near the Illinois border between Vincennes and Terre Haute. Some chimneys fell, several building walls were cracked, light connections were severed, and pictures were shaken off the walls. It was strong in Indianapolis and Oakland City. It was felt over an area of 80,000 km² (see Fig. 3-9) including the southwestern half of Indiana, all of Illinois and parts of Iowa, Kentucky, Missouri, Arkansas, and probably in parts of Kansas.

Fig. 3-9 – Isoseismal map for the Wabash River Valley, Indiana, Earthquake of September 27, 1909.

The latest important earthquake originated within the State occurred June 18, 2002, with magnitude 4.6 (USGS, but $m_b = 5.0$ according to other sources) near Evansville. There was very little damage. The location of the earthquake and its hypocentral depth (14-18 km) indicated ongoing deformation along reactivated Precambrian and Paleozoic basement structures, in a zone of recurring seismic activity, and in an area of possibly heightened neotectonic strain [Hamburger et al., 2002]. Figure 3-10 shows the intensity

map for this earthquake obtained from the Community Internet Intensity Map USGS program.

Fig. 3-10 – Isoseismal map for the Evansville, Indiana, Earthquake of June 18, 2002

Other damaging earthquakes originated in Indiana include the April 29, 1899, Earthquake with rated intensity VI to VII on the Modified Mercalli Scale. It was strongest at Jeffersonville and Shelbyville; at Vincennes, chimneys were thrown down and walls cracked. It was felt over an area of 110,000 km².

In 1876, twin shocks fifteen minutes apart were felt over an area of 160,000 km². A shock in 1887 centered near Vincennes was felt over 200,000 km² and an 1891 shock damaged property and frightened people in church at Evansville.

Indiana has also suffered from damage caused by earthquakes originating in neighboring States. The worst occurred on November 9, 1968, and centered near Dale in southern Illinois. The shock, a magnitude 5.3, was felt over 1,500,000 km² and 23 States including all of Indiana. Intensity VII was reported from Cynthiana where chimneys were cracked, twisted, and toppled; at Fort Branch where groceries fell from shelves and a loud roaring noise was heard, and at Mount Vernon, New Harmony, Petersburg, Princeton, and Stewartsville, all of which had similar effects.

Almost exactly ten years earlier on November 7, 1958, an earthquake originating near Mt. Carmel, Illinois, caused plaster to fall at Fort Branch. Roaring and whistling noises were heard at Central City and the residents of Evansville thought there had been an explosion or plane crash. It was felt over 90,000 km² of Illinois, Indiana, Missouri, and Kentucky.

On March 2, 1937, a shock centering near Anna, Ohio, threw objects from shelves at Fort Wayne and some plaster fell. Plaster was also cracked at Indianapolis. Six days later, another shock originating at Anna brought pictures crashing down and cracked plaster in Fort Wayne and was strongly felt at Lafayette.

The great New Madrid earthquakes of 1811 and 1812 must have strongly affected the State, particularly the southwestern part, but there is little information available.

3.3 Design earthquake return period

For the foregoing discussion on the seismic risk of Indiana it is important to establish a common base for comparing the probability of exceedance of the design ground motion, the earthquake mean return period, and the lapse of exposure or life of the structure.

The probability, q , of having ground motion that exceeds in one year a pre-fixed value of a descriptive parameter, a_o , – for example peak ground acceleration – is:

$$q = P(a \geq a_o, \text{ in one year}) \quad (3-1)$$

The probability of not exceeding the value of a_o in t years, under the assumption of statistical independence between events, is:

$$q_t = (1-q) \underset{1}{(1-q)} \underset{2}{(1-q)} \underset{3}{(1-q)} \cdots \underset{t}{(1-q)} \quad (3-2)$$

Then:

$$q_t = (1-q)^t \quad (3-3)$$

The probability, q_o , of having at least one event with a value greater than a_o in a time span of t years is:

$$q_o = 1 - q_t = 1 - (1-q)^t \quad (3-4)$$

The return period is defined as the mean time, in years, between events producing a value of the descriptive parameter greater or equal to a_o :

$$\bar{T} = \frac{1}{q} \quad (3-5)$$

Then:

$$q_o = 1 - q_t = 1 - \left(1 - \frac{1}{\bar{T}}\right)^t \quad (3-6)$$

For example, if the design earthquake has been defined as an event that produces a value of the descriptive parameter having a probability of 3% of being exceeded in a 75 year interval; using the formulation presented: $q_o = 0.03$ and $t = 75$ years. Applying Eq. (3-6):

$$0.03 = 1 - \left(1 - \frac{1}{\bar{T}}\right)^{75} \quad \text{then} \quad 0.97 = \left(1 - \frac{1}{\bar{T}}\right)^{75} \quad \text{and} \quad \left(1 - \frac{1}{\bar{T}}\right) = 0.97^{\left(\frac{1}{75}\right)} = 0.999594$$

It follows that $\frac{1}{\bar{T}} = 1 - 0.999594 = 0.00040604$, and the mean period of recurrence for this case is, then:

$$\bar{T} = \frac{1}{0.00040604} = 2463 \text{ years}$$

Table 3-1 – Design Ground Motion Mean Return period used in different documents

Document and reference	Structure life-span (years)	Probability of exceedance	Mean Return Period (years)
AASHTO Standard Specifications for Highway Bridges 5 th Edition (1949)	N/A	N/A	N/A
AASHTO Standard Specifications for Highway Bridges 8 th Edition (1961)	N/A	N/A	N/A
AASHTO 1975 Interim Specifications for Highway Bridges [AASHTO, 1975]	N/A	N/A	N/A
ATC-3-06 Recommendations for Buildings [ATC, 1978]	50	10%	475
ATC-6 Design Guidelines for Bridges [ATC, 1981]	50	10%	475
Division I-A of AASHTO Standard Specifications (15 th to current 17 th Edition) [AASHTO, 2002]	50	10%	475
AASHTO LRFD Specifications (1 st , 2 nd , and 3 rd Edition) [AASHTO, 1998]	50	10%	475
Caltrans Seismic Design Criteria [Caltrans, 2001]	N/A	N/A	N/A
NEHRP Recommendations for New Buildings [NEHRP, 1997] – Maximum Considered Earthquake (see Note)	50	2%	2475
Draft AASHTO LRFD Seismic Provisions [NCHRP, 2002] – Maximum Considered Earthquake	75	3%	2463
Draft AASHTO LRFD Seismic Provisions [NCHRP, 2002] – Frequent Earthquake	75	50%	109

Note – In NEHRP 1997 the Design Ground Motion for buildings corresponds to 2/3 of the Maximum Considered Earthquake ground motion. This reduction is not used in the Draft AASHTO LRFD seismic provisions for bridges although the definition of the Maximum Considered Earthquake is practically the same.

Figure 3-11 shows the sensitivity to the mean return period when changing the life span of the structure or the probability of exceedance.

Fig. 3-11 – Mean return period as a function of the probability of exceedance and the life span of the structure in years

3.4 Seismic risk assessment for bridge design in Indiana

The awareness of the seismic risk associated with the New Madrid Seismic Zone and more recently the Wabash Valley Seismic Zone, as presented before, has been increasing in the last decades with the corresponding increase in the seismic forces and related requirements used in design of building structures and bridges. The following sections present the seismic risk parameters presented by the bridge specifications in the last decades in order to set a common frame of reference to be used in the vulnerability assessment of the transportation structures within the selected Earthquake Emergency Routes of Indiana.

Figure 3-12 shows the 1949 revision of the seismic risk map developed by the U.S. Coast and Geodetic Survey [Housner, 1970]. Table 3-2 presents the peak ground accelerations suggested by Housner to be used with the map of Fig. 3-12.

Fig. 3-12 – U.S. Coast and Geodetic Survey Seismic Probability Map of the United States (Revised 1949)

Table 3-2 – Maximum zonal acceleration to be used with map of Fig. 3-12 [Housner, 1970]

	Maximum Acceleration	M
Zone 3 (near a great fault)	0.50g	8.5
Zone 3 (not near a great fault)	0.33g	7.0
Zone 2	0.16g	5.75
Zone 1	0.08g	4.75
Zone 0	0.04g	4.25

This map was developed before seismic zoning was introduced in the building or bridge design requirements. Although the map was referred as a seismic probability map it actually was based on Mercalli intensity and no guidance was given on the corresponding return period or probability of exceedance. Seen from a modern point of view this map contains features such as a neighboring Zone 1 and 3 areas without a transition Zone 2 in between, and other that currently would be frowned at. Notwithstanding, the general location of the zones and envisioned peak ground acceleration do not differ much from more recent maps. In this map southwestern Indiana

is located in a Zone 2 corresponding to a maximum acceleration of 0.16g and a corresponding magnitude of 5.75 for the design event. The rest of Indiana, with the exception of a small Zone 2 bordering Ohio, was located in a Zone 1 (maximum expected acceleration of 0.08g and magnitude of 4.75 for the design event. No design response spectrum was defined. This map was never mandatory in Indiana for bridges or buildings.

3.4.1 1975 AASHTO Interim Specifications for Highway Bridges

In Section 2.2.2 the *1975 AASHTO Interim Specifications* [AASHTO, 1975] were presented. Figure 3-13 shows the northern Midwest portion of the map including Indiana (the whole U.S. map is shown in Fig. 2-13). The seismic design criteria contained in the 1975 AASHTO Interim Specifications was included in the twelfth (1977), thirteenth (1983), and fourteenth (1989) editions of AASHTO's Standard Specifications.

Fig. 3-13 – Northern Midwest portion of the map included with the *1975 AASHTO Interim Specifications* [AASHTO, 1975]

No more detailed map was included with the 1975 Interim. The southwestern corner of Indiana containing Gibson, Posey and Vanderburgh counties was a Zone 3 with a peak ground acceleration $A = 0.5g$. A region south of an approximate line linking Terre Haute, Indianapolis and Fort Wayne was included in a Zone 2 ($A = 0.22g$ PGA). The rest of the State was located in a Zone 1 ($A = 0.09g$ PGA).

Table 3-3 – Maximum values for the Response Coefficient (C) in Indiana
1975 AASHTO Interim Specifications [AASHTO, 1975]

Depth of Alluvium	Zone 1 ($A = 0.09g$)		Zone 2 ($A = 0.22g$)		Zone 3 ($A = 0.50g$)	
	C max	T at peak	C max	T at peak	C max	T at peak
0 – 10'	0.060	-	0.072	0.30 s	0.163	0.30 s
11' – 80'	0.066	0.40 s	0.120	0.35 s	0.203	0.30 s
80' – 150'	0.080	0.60 s	0.110	0.55 s	0.155	0.50 s
> 150'	0.065	0.65 s	0.090	0.55 s	0.120	0.50 s

3.4.2 ATC 6 and AASHTO Division I-A

The *ATC-6* [ATC, 1981] recommendations were modified to be incorporated in the fifteenth (1992) edition of *AASHTO Standard Specifications* as *Division I-A Seismic Design*. These requirements were kept without modification in the sixteenth (1996) and in the current seventeenth (2002) edition [AASHTO, 2002]. The *ATC-6* document used for peak ground acceleration in rock, A , the map of A_v originally proposed for the ATC-3-06 building requirements project.

The ATC-3-06 requirements for buildings were taken over by the BSSC and became the NEHRP recommended provisions. By the time the *ATC-6* proposed bridge requirements were incorporated into *AASHTO Standard Specifications* the maps had been updated in the 1988 NEHRP provisions [NEHRP, 1988] by the USGS [Algermissen et al., 1990]. The map included in the 15th Edition AASHTO Standard Specifications and still contained in the current 17th Edition correspond to the A_v map as modified in 1988. Figure 3-14 shows the State of Indiana portion of this map. The values given correspond to peak ground acceleration in rock expressed as percentage of the acceleration of gravity for a probability of exceedance of 10 percent in 50 years, which is approximately equal to a 15 percent probability of exceedance in the 75-year design life now advocated for a typical highway bridge (see Fig. 3-11).

The southwestern corner of Indiana is located within contours labeled for peak ground acceleration of $A = 0.075g$ and $A = 0.10g$; including totally (t) or partially (p) the following 20 counties: Brown (p), Clay (p), Daviess (t), Dubois (p), Gibson (t), Greene (t), Knox (t), Lawrence (p), Martin (t), Monroe (p), Morgan (p), Owen (t), Pike (t), Posey (t), Putnam (p), Spencer (p), Sullivan (t), Vanderburgh (t), Vigo (p), and Warrick (t)..

Section 3.2 of *Division I-A Seismic Design* indicates that “*Local maxima (and minima) are given inside the highest (and lowest) contour for a particular region. Linear interpolation shall be used for sites located between contour lines and between a contour line and local maximum (or minimum).*”

The rest of the south half of the state is located within a contour line for peak ground acceleration $A = 0.05g$ and local maxima labeled as $A = 0.06g$. This zone includes Terre Haute and Indianapolis. The northern half of the State, including the cities of Lafayette and Fort Wayne, located within the contour line for peak ground acceleration $A = 0.05g$ and local maxima labeled as $A = 0.04g$.

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Fig. 3-14 – Values for peak ground acceleration for the State on Indiana in *Division I-A Seismic Design of 17th Edition of AASHTO Standard Specifications* [AASHTO, 2002]

All editions of the AASHTO LRFD Specifications (1994, 1998, 2004) [AASHTO, 2004] include the same map but contain variations in the design requirements that may lead to a different design although based on the same seismic risk assessment (see 2.2.2).

3.4.3 USGS 1996 Maps for the Draft AASHTO LRFD Seismic Provisions

In 1993, the USGS embarked on a major project to prepare updated national earthquake ground motion maps. The result of that project was a set of probabilistic maps published in 1996 that cover several rock ground motion parameters (peak ground acceleration — PGA — and elastic response spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 sec) and three different probability levels or return periods (10% probability of exceedance in 50 years, 5% probability of exceedance in 50 years, and 2% probability of exceedance in 50 years — which is approximately equal to 3% probability of exceedance in 75 years). In addition to the maps, the ground motion values at any specified latitude and longitude can be obtained via the Internet.

Fig. 3-15 – 1996 USGS Map of peak ground acceleration in rock for a probability of exceedance of 10% in 50 years

These maps form the rock ground motion basis for seismic design using the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002]. Figure 3-15 shows the 1996 USGS Map of peak ground acceleration in rock for a probability of exceedance of 10% in 50 years that although not used in the *Draft AASHTO LRFD Seismic Provisions*, is given here for reference with previous requirements.

Fig. 3-16 – 1996 USGS Map of peak ground acceleration in rock for a probability of exceedance of 3% in 75 years (= to 2% in 50 years)

The upper-level event, termed the “rare” or Maximum Considered Earthquake (MCE), describes ground motions that, for most locations, are defined probabilistically and have a probability of exceedance of 3% in 75 years (mean return period approximately 2500 years). Figure 3-16 shows the 1996 USGS Map of peak ground

acceleration in rock for a probability of exceedance of 2% in 50 years (equivalent to 3% in 75 years as shown in Fig. 3-11) that although not used also in the *Draft AASHTO LRFD Seismic Provisions*, is given here for reference with previous requirements using a different return period.

For locations close to highly active faults, the mapped Maximum Considered Earthquake ground motion was deterministically bounded so that the levels of ground motion did not become unreasonably high. Deterministic bounds on the ground motion were calculated by assuming the occurrence of maximum magnitude earthquakes on the highly active faults. It is equal to 150% of the median ground motion for the maximum magnitude earthquake, but not less than 1.5g for the short-period spectral acceleration plateau and 0.6g for 1.0-sec spectra acceleration (see Fig. 2-22). Deterministic bounds were applied in high-seismicity portions of the West Coast, Alaska, and Hawaii leading to a design ground motion lower than the ground motion for 3% probability of exceedance in 75 years. The Maximum Considered Earthquake governs the limits on the inelastic deformation in the substructures and the design displacements for the support of the superstructure.

The lower level design event, termed the “expected” or “frequent” earthquake, defines a ground motion corresponding to 50% probability of exceedance in 75 years. This event ensures that essentially elastic response is achieved in the bridge substructure for the more frequent or expected earthquake. This design level is similar to the 100-year flood and has similar performance objectives. An explicit check on the strength capacity of the bridge substructure is required. Parameter studies made as part of the development of the draft provisions show that the lower level event will only impact the strength of columns in parts of the western United States.

With respect to Indiana the USGS 1996 Maps used the then recently identified paleoearthquakes in southern Indiana and Illinois based on widespread paleoliquefaction features as described in 3.1. An areal zone with a higher $M_{\max}$ of 7.5 to account for such large events was used. The Wabash Valley $M_{\max}$ zone that was used in the maps was based on the Wabash Valley fault zone.

Fig. 3-17 – 1996 USGS Map of spectral acceleration S_0 in rock for a probability of exceedance of 3% in 75 years (= to 2% in 50 years)

Fig. 3-18 – 1996 USGS Map of spectral acceleration S_1 in rock for a probability of exceedance of 3% in 75 years (= to 2% in 50 years)

The *Draft AASHTO LRFD Seismic Provisions* uses maps that define spectral ordinates at periods of 0.2 sec. (S_S) and 1.0 sec. (S_1) to be used as a two point procedure to obtain the design response spectrum (see Fig. 2-21). Figures 3-17 and 3-18 show the maps for S_S and S_1 in rock respectively for the Maximum Considered Earthquake having a 3% probability of exceedance in 75 years.

No maps were developed by USGS for the expected frequent earthquake, defining a ground motion corresponding to 50% probability of exceedance in 75 years. The *Draft AASHTO LRFD Seismic Provisions* suggests the values of S_S and S_1 for this earthquake may be obtained from interpolation from information provided by USGS either in the CD-ROM published with the maps or in the Internet by using latitude and longitude for the site or the ZIP code of the location. The web site to perform these operations is: <http://eqint.cr.usgs.gov/eq/html/deaggint.html>

Fig. 3-19 – Results obtained for Vincennes, IN, for spectral acceleration S_S for the frequent earthquake, in rock for 50% probability of exceedance in 75 years

Figure 3-19 show the results obtained for Vincennes for spectral acceleration S_S in rock for 50% probability of exceedance in 75 years as required for the frequent earthquake. The values obtained were $S_S = 0.06g$ and $S_1 = 0.01g$.

3.4.4 2002 Update of the 1996 USGS Maps

In 2002 changes were introduced by the USGS [Frankel et al., 2002] to the maps developed in 1996. Numerous changes were introduced with varying effects on the mapped values. Some of these changes affected the parameters used in the New Madrid Seismic Zone and the Wabash Valley Seismic Zone. Among them the more important are changes in the mean recurrence time, the characteristic magnitude, and the spatial concentration of New Madrid sources of large earthquakes, and the incorporation of additional attenuation relations. Two versions of the maps exist one from January and the other from October. Only the later version maps are be presented here.

The 2002 update uses a shorter mean recurrence time for characteristic earthquakes in New Madrid than was used in the 1996 maps, as well as a smaller median magnitude than that applied in 1996. The three equally weighted “fictitious sources” used in the 1996 maps were substituted for three sources consisting in a fault trace matching recent micro-earthquake activity and two adjacent sources that are situated near the borders of the Reelfoot Rift. It is important to note that the probabilistic ground motions for the 10% probability of exceedance level have increased markedly around the New Madrid area, compared to the 1996 maps. This is caused by the shorter mean return time of 500 years for characteristic earthquakes used in the 2002 maps. The $M_{\max}$ 7.5 zone assigned to the Wabash Valley area was enlarged to include the most likely rupture zones from paleoearthquakes with magnitudes above about 7.0.

Significant differences between the 1996 and 2002 maps are caused by the inclusion of additional attenuation relations in the 2002 maps. These changes affected mainly the Atlantic seaboard. There is little change for the probabilistic ground motions at 2% probability of exceedance in 50 years for most of the central U.S. During 2003, changes were made twice to the 2002 maps but these changes only affect some areas of the West Coast.

Fig. 3-20 – 2002 USGS Map for Peak Ground Acceleration in rock with 10% probability of exceedance in 50 years

Fig. 3-21 – 2002 USGS Map for Peak Ground Acceleration in rock with 3% probability of exceedance in 75 years (= to 2% in 50 years)

Fig. 3-22 – 2002 USGS Map for S_5 Spectral Acceleration in rock with 3% probability of exceedance in 75 years (= to 2% in 50 years)

Fig. 3-23 – 2002 USGS Map for S_1 Spectral Acceleration in rock with 3% probability of exceedance in 75 years (= to 2% in 50 years)

In April 2004 the USGS included in their website the possibility of producing maps for latitude and magnitude pairs. The Indiana PGA map for 475-year return period is presented in Fig. 3-20 and for the 2500-year return period in Fig. 3-21, and the maps for S_S and S_1 for 2500-year return period in Figures 3-22 and 3-23 respectively.

3.4.5 JTRP Project SPR 2812 – Analysis of Seismic Hazard Assessment for Indiana

A Joint Transportation Research Program Project (SPR 2812) lead by Purdue Professors J. Haase and R. Nowack is currently working on an assessment of the seismic hazard in Indiana with specific application to transportation structures. The results of this research were not available to be used in the Emergency Routes Project but undoubtedly will have a large impact in the application of future bridge design specifications for the Indiana Department of Transportation and seismic rehabilitation policies to be used with existing bridges. Once the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* is adopted by AASHTO, and later by INDOT, the need for having an in-house State assessment of the seismic risk to be coordinated with the USGS will become very important.

The *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* includes in Commentary to Section 3.10.2.1 a clause that reads: “*In lieu of using the national ground motion maps referenced in the Specification, ground motion response spectra may be constructed based on approved State ground motion maps. To be accepted the development of State maps should conform to the following:*

1. *The definition of design ground motion should be the same as described in Article 3.10.1.2 and Table 3.10.1-1.*
2. *Ground motion maps should be based on a detailed analysis demonstrated to lead a quantification of ground motion at a regional scale that is as or more accurate than achieved at the scale of the national maps. The analysis should include: characterization of seismic sources and ground motion that incorporates current scientific knowledge; incorporation of uncertainty in seismic source and ground motion models and parameter values used in the analysis; detailed documentation of map development; detailed peer review. The peer review process should*

preferably include one or more individuals from the U.S. Geological Survey who participated in the development of the national maps.”

3.4.6 Comments on the results from the different maps and bridge design requirements

Mapped values for acceleration in rock in Bloomington, Evansville, Indianapolis, Jasper, New Albany, Terre Haute, and Vincennes (see Fig. 3-24 for location) are shown as read from the different seismic risk maps in Table 3-4.

Fig. 3-24 – Location where acceleration mapped values are compared for the different seismic risk map versions (see Table 3-4). Base map shows INDOT districts.

In comparing the values shown in Table 3-4 is must be noticed that the values from the 1949 map developed by the U.S. Coast and Geodetic Survey [Housner, 1970] and in the 1975 AASHTO Interim [AASHTO, 1975] are higher, as a rule, than the values for peak ground acceleration contained in *Division I-A Seismic Design* of the current 17th Edition of the *AASHTO Standard Specifications* [AASHTO, 2002]. It may be argued than the values contained in Division I-A correspond to EPA (Effective Peak Ground Acceleration) which takes into account several additional parameters such as duration of

the strong ground motion and are not directly comparable to the instrumental peak ground acceleration recorded in an accelerometer, in addition to the acceleration in the 1949 and 1975 documents not being defined clearly with respect to mean return period.

Table 3-4 – Seismic Zone definition and mapped values of acceleration for selected cities in Indiana for the different maps

Map	City (County) → Parameter Presented ↓	Bloomington (Monroe)	Evansville (Vanderburgh)	Indianapolis (Marion)	Jasper (Du Bois)	New Albany (Floyd)	Terre Haute (Vigo)	Vincennes (Knox)
1949 USCGS	Zone	2	2	1	2	1	2	2
	PGA	0.16g	0.16g	0.08g	0.16g	0.08g	0.16g	0.16g
AASHTO 1975 Interim	Zone	2	3	2	2	2	2	2
	A (PGA)	0.22g	0.50g	0.22g	0.22g	0.22g	0.22g	0.22g
AASHTO Div. I-A	A (PGA) 10% PE 50y	0.078g	0.085g	0.063g	0.075g	0.068g	0.070g	0.090g
1996 USGS Maps	PGA 10% PE 50y	0.045g	0.130g	0.032g	0.070g	0.039g	0.056g	0.110g
	PGA 2% PE 50y	0.14g	0.49g	0.08g	0.20g	0.12g	0.08g	0.40g
	S_s 2% PE 50y	0.28g	0.80g	0.20g	0.60g	0.22g	0.36g	0.80g
	S_1 2% PE 50y	0.12g	0.30g	0.10g	0.16g	0.14g	0.14g	0.20g
2002 USGS Maps	PGA 10% PE 50y	0.046g	0.120g	0.034g	0.070g	0.043g	0.060g	0.100g
	PGA 2% PE 50y	0.12g	0.40g	0.10g	0.20g	0.12g	0.16g	0.30g
	S_s 2% PE 50y	0.28g	0.80g	0.20g	0.60g	0.28g	0.36g	0.60g
	S_1 2% PE 50y	0.12g	0.20g	0.10g	0.16g	0.12g	0.12g	0.16g

The maps in current bridge design requirements contained in *Division I-A* of the *AASHTO Standard Specifications* were developed by the USGS in the 1970's and

updated in 1988 and they are comparable in respect of having the same mean return period with the 1996 and 2002 PGA USGS maps for 10% probability of exceedance in 50 years (Figures 3-15 and 3-20). The impact of the knowledge that has been accumulated about the New Madrid and Wabash Valley Seismic Zones is evident in the more detailed acceleration values reported. The values of PGA for the same return period in the new 1996 and 2002 maps are lower for Bloomington, Indianapolis, Japer, New Albany and Terre Haute, and higher for Evansville and Vincennes. This is consistent with the reported new understanding of the tectonic process of the Wabash Valley Seismic Zone and if the new maps were made part of *Division I-A* of the *AASHTO Standard Specification* the impact on the design of new bridges in Indiana would be marginal if not less demanding in many cases.

In order to assess the impact of adopting the proposed *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] as seen solely from the design acceleration point of view the following aspects come into play: (a) a change from a 475-year to a 2500-year return period design earthquake, (b) a change from peak ground acceleration (effective or accelerographic reported) description of the design ground motion to be routinely converted into a design spectrum to a new direct definition of the design spectrum, (c) site effects that amplify design acceleration in a different way including now short period accelerations along with long period ones both of them specially affecting the low acceleration range where precisely Indiana sites are located, and (d) use of a “frequent” earthquake for serviceability assessment.

The pertinent comments will follow in the same order listed. Although, the research presented in this report deals mainly with existing bridges — the reference for any bridge upgrade has been customarily related to current or in the process of being adopted requirements for new bridges — the approach adopted minimizes the dependence on the seismic risk assessment and focus on the expected behavior for increasingly severe ground motions without putting a tap on them.

(a) *Impact of change of design event return period* — It is debatable that with a relatively short-timed documented record of events proper extrapolations can be performed. Extrapolation is common in other civil engineering disciplines with

acceptable results from the point of view of proper design parameters and public awareness of the problems involved in distinguishing between acceptable risk and forecast. Flood control is a good example of the differences involved and the acceptability of the design parameters involved. On the other hand, both extreme wind design and earthquake resistant design have come into public scrutiny because of the gap between code design parameters and reported measured values. This has brought a trend to define design parameters in the same order of magnitude of the reported measured values. The basis for this change is well documented in the related literature and the real impact on the design procedures has been moderate because the reference to actual behavior during catastrophic events is the judging parameter in most cases. The new seismic risk maps presented in the USGS 1996 and 2002 maps describe a Maximum Considered Earthquake (MCE) that is supposed to produce measurable parameters such as peak ground acceleration of the same order of magnitude of those that may be recorded in the occurrence of a catastrophic event. How these values are made compatible with acceptable design parameters — from the economic and code requirements perspectives — have been solved in different ways.

The building design requirements were the first to adopt this approach [NEHRP, 1997]. The MCE was accepted but the design ground motion was defined as $2/3$ that produced by the MCE. The end result was design parameters that did not differ much from what was being used. The proposed *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] does not introduce the same reduction, although changes the accepted Response Modification Factors (R) to higher values that may bridge the difference (please compare values listed in Tables 2-3 and 2-8). Other subtle difference between the approach for buildings and for bridges is that in the case of site specific studies the building requirements require that the deterministic spectrum be obtained from 150% the mean of the spectra computed for characteristic maximum magnitude earthquakes while for bridges it requires the median-plus-one-standard-deviation which leads to 184% of the mean spectrum. One important conclusion is that in the future the basic seismic risk design factors will be different for buildings and for bridges although based on the same seismic risk maps.

(b) *Impact of change from PGA to spectral description* — The description of the design ground motion through a response spectrum was formalized in the ATC 3-06 project [ATC, 1978] based on the work of N. Newmark. The spectrum description was based on two parameters defined as Effective Peak Acceleration (EPA) and Effective Peak Velocity (EPV) that were related to the spectral ordinates as shown in Fig. 3-25 with EPA made equal to parameter A_a and EPV proportional to A_v . This procedure was the one adopted in ATC 6 for bridges and later in *Division I-A* of the *Standard AASHTO Specification* and is the basis of current 17th edition with the acceleration parameter A corresponding to the mapped value of A_v of the 1988 maps[NEHRP, 1988] by the USGS [Algermissen et al., 1990].

Fig. 3-25 – $EPA = S_a/2.5$ and $EPV = S_v/2.5$ obtained from a response spectrum with 5% damping as prescribed by ATC 3-06 [ATC, 1978]

The use of two different parameters to define the design response spectrum one based on acceleration and the other based on velocity was a wise decision because attenuation with distance for the two parameters is not the same. The 2.5 proportional constant was developed by Newmark based on the study of many accelerograms and their corresponding response spectrum and in general works well for most records but the commentary of the ATC-3-06 made it clear that effective peak acceleration and effective peak velocity were different from the peak acceleration in the accelerogram, and the peak velocity obtained by integration of the accelerogram.

The proposed *Draft AASHTO LRFD Seismic Provisions* moves away from this definition of the design ground motion and directly states the design spectrum severing the link with the effective peak ground acceleration and velocity, although it states a way to obtain the peak ground acceleration as $0.4S_{DS}$ as was shown in Fig. 2-21 thus apparently keeping the same 2.5 ratio. As shown in Table 3-4 for the seven Indiana sites, in the 2002 USGS maps the ratio between S_S and PGA for the same return period reports ratios between 2 and 3 with a mean of 2.27. Although these ratios are not fixed and will probably change in the future as new versions of the maps appear, the impact of moving from a PGA definition to a directly defined spectrum as the description of the design ground motion is minor.

Figures 3-26 and 3-27 show the combined impact of the change in return period and the spectral definition from the current *Division I-A* seismic design requirements response spectrum to the one required in the proposed *Draft AASHTO LRFD* requirements using the 2002 USGS maps.

Fig. 3-26 – Bridge Design Spectra in rock for Bloomington, Evansville, Jasper, and Vincennes in current *Division I-A* and proposed *Draft AASHTO LRFD* requirements

Fig. 3-27 – Bridge Design Spectra in rock for Indianapolis, New Albany, and Terre Haute in current *Division I-A* and proposed *Draft AASHTO LRFD* requirements

Table 3-5 – Analysis and Design requirements for selected cities in Indiana for the current *Division I-A* and proposed *Draft LRFD* requirements (spectral values in rock)

Specs	City (County) → Requirements ↓	Bloomington (Monroe)	Evansville (Vanderburgh)	Indianapolis (Marion)	Jasper (Du Bois)	New Albany (Floyd)	Terre Haute (Vigo)	Vincennes (Knox)
AASHTO Div. I-A	SPC (Essent./Other)	A/A	A/A	A/A	A/A	A/A	A/A	A/A
Proposed Draft LRFD	SHL	II	IV	II	III	II	III	III
	SDAP (Life S. /Oper.)	A2/ CDE	CDE/ CDE	A2/ CDE	BCDE/ CDE	A2/ CDE	BCDE/ CDE	BCDE/ CDE
	SDR (Life S. /Oper.)	2/3	5/6	2/3	3/4	2/3	3/4	3/4

Table 3-5 shows the implication from the point of view of the corresponding associated design analysis and detailing requirements presenting the Seismic Performance Category (SPC) (see Table 2-1) of the current requirements and for the

proposed Draft LRFD requirements the Seismic Hazard Level (SHL) (see Tables 2-5 and 2-6), the Seismic Design and Analysis Procedure (SDAP) (see Table 2-7), and Seismic Detailing Requirements (SDR) (see Tables 2-9 and 2-10).

(c) *Impact of change of site effects evaluation procedure* — The USGS 1996 and 2002 maps are defined at the B-C soil profile type interface (see Table 2-11) which is the dividing line between rock and very dense soil or soft rock. The soil amplification factors are both $F_a = 1.0$ and $F_v = 1.0$ for soil profiles type B. This is similar to the Soil Profile Type I of *Division I-A* of the current 17th Edition of the *Standard AASHTO Specifications* for which has a Site Coefficient $S = 1.0$ (see Fig. 2-16). Up to this point the procedures are similar. When other soil profile types come into play, the difference is large because the proposed *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] has soil amplification factors F_a that affects the short period part of the design spectrum. AASHTO current Standard Specifications does not have a comparable soil dependent factor. Both F_a and F_v are acceleration dependent with comparatively larger values for lower accelerations (see Figures 2-24 and 2-25). Just for comparison purposes a site, common in Indiana, with more than 30 ft of soft to medium stiff soil under current *Division I-A* requirements would be a Type III soil profile and would have a Site Coefficient $S = 1.5$ that amplify the medium to long period part of the design spectrum. The same soil would probably be classified as a Type D Stiff Soil when using the proposed *Draft AASHTO LRFD Seismic Provisions*. Table 3-6 shows the values obtained for the cases being use for comparison.

For the short period portion of the design spectrum under the proposed *Draft Seismic Provisions* the amplification caused by the soil profile would be 18% to 60% percent higher than the current requirements with no amplification required. For the medium to long period portion of the design spectrum the values under the proposed Draft Seismic Provisions the amplification caused by the soil profile would be 33% to 60% percent higher than the current requirements. The reason for this large impact is linked to Indiana having mapped accelerations in the low range where the soil amplification effect is relatively larger.

Table 3-6 – Values of the site coefficient for a Type III soil profile under current *Division I-A* requirements and a Type D soil under proposed *Draft LRFD Provisions*

Specs	City (County) → Parameter Presented ↓	Bloomington (Monroe)	Evansville (Vanderburgh)	Indianapolis (Marion)	Jasper (Du Bois)	New Albany (Floyd)	Terre Haute (Vigo)	Vincennes (Knox)
AASHTO Div. I-A	A (PGA) 10% PE 50y	0.09g	0.09g	0.06g	0.09g	0.06g	0.06g	0.09g
	Site Coefficient S	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Draft AASHTO LRFD plus 2002 USGS Maps	S_s 2% PE 50y	0.28g	0.80g	0.20g	0.60g	0.28g	0.36g	0.60g
	Soil Type D Factor F_a	1.58	1.18	1.60	1.32	1.58	1.51	1.32
	S_1 2% PE 50y	0.12g	0.20g	0.10g	0.16g	0.12g	0.12g	0.16g
	Soil Type D Factor F_v	2.32	2.00	2.40	2.16	2.32	2.32	2.16

Figures 3-28 and 3-29 show the combined impact of the change in return period, the spectral definition and the soil amplification requirements for a Type III soil profile from the current Division I-A seismic design requirements design spectrum to the one required in the proposed Draft AASHTO LRFD requirements for a Type D profile using the 2002 USGS maps.

Table 3-7 shows the implication from the point of view of the corresponding associated design analysis and detailing requirements presenting the Seismic Performance Category (SPC) (see Table 2-1) of the current requirements and for the proposed Draft LRFD requirements the Seismic Hazard Level (SHL) (see Tables 2-5 and 2-6), the Seismic Design and Analysis Procedure (SDAP) (see Table 2-7), and Seismic Detailing Requirements (SDR) (see Tables 2-9 and 2-10).

Fig. 3-28 – Bridge Design Spectra in soil for Bloomington, Evansville, Jasper, and Vincennes in current *Division I-A* and proposed *Draft AASHTO LRFD* requirements

Fig. 3-29 – Bridge Design Spectra in soil for Indianapolis, New Albany, and Terre Haute in current *Division I-A* and proposed *Draft AASHTO LRFD* requirements

It is evident from the values presented for rock and for soil that the impact of the proposed Draft LRFD requirements is significant both for the design forces and for the analysis and detailing requirements. Under the current Division I-A requirements the analysis and detailing are the simplest (SPC-A) while for the proposed Draft LRFD the most strict covered (SHL-IV) are required for Evansville and Vincennes in both cases (rock and soil) studied and for Jasper in soil. In no case the simpler (SHL-I) is required.

The implications from the point of view of complexity of design and cost of construction are significant.

Table 3-7 – Analysis and Design requirements for selected cities in Indiana for the current *Division I-A* and proposed *Draft LRFD* requirements (spectral values in soil)

Specs	City (County) → Requirements ↓	Bloomington (Monroe)	Evansville (Vanderburgh)	Indianapolis (Marion)	Jasper (Du Bois)	New Albany (Floyd)	Terre Haute (Vigo)	Vincennes (Knox)
AASHTO Div. I-A	SPC (Essent./Other)	A/A	A/A	A/A	A/A	A/A	A/A	A/A
Proposed Draft LRFD	SHL	III	IV	II	IV	III	III	IV
	SDAP (Life S. /Oper.)	BCDE/ CDE	CDE/C DE	A2/ CDE	CDE/ CDE	BCDE/ CDE	BCDE/ CDE	CDE/ CDE
	SDR (Life S. /Oper.)	3/4	5/6	2/3	5/6	3/4	3/4	5/6

(d) *Impact of use of “frequent” earthquake* — The proposed *Draft AASHTO LRFD Seismic Provisions* introduces a frequent earthquake to be used in verification of serviceability of the bridge. Figure 3-19 shows the value 0.06g obtained for S_S at Vincennes for this earthquake based on results of the 1996 USGS maps. The impact of using such a low spectral acceleration for verification of serviceability will be probably negligible.

CHAPTER 4 METHODOLOGY FOR EVALUATION OF SEISMIC VULNERABILITY OF THE INDIANA EMERGENCY ROUTES

4.1 Route Alternatives

U.S. Congress and the Federal Administration through the FHWA have defined priority and critical routes in different instances. One such example is the National Truck Network established in 1982 by the Surface Transportation Assistance Act in order to designate a national network of highways that would allow the passage of trucks of specified minimum dimensions and weight. Figure 4-1 shows the National Truck Network in southwestern Indiana.

Fig. 4-1 – National Truck Network in southwestern Indiana

One more recent highway system definition is the National Highway System. The National Highway System Designation Act of 1995 selected 264,000 km of roadways as the National Highway System (NHS) including 4,750 km in Indiana. Figure 4-2 shows the whole National Highway System.

Fig. 4-2 – National Highway System

NHS consists of five parts. The first component is the Interstate Highway System, which accounts for almost 30 percent of NHS. The second component includes 21 congressionally designated high-priority corridors (see Fig. 4-3), three of which include highways in Indiana as part of them. The third component is the non-interstate portion of the Strategic Highway Corridor Network identified by the Department of Defense in cooperation with the Department of Transportation. These corridors and the interstate highways are critical strategic links. The fourth component is major Strategic Highway

Corridor Network connectors linking major military installations and other defense-related facilities to the Strategic corridors. The fifth component are important arterial highways that serve interstate and interregional travel and that provide connections to major ports, airports, public transportation facilities, and other intermodal facilities.

Fig. 4-3 – Congressional High Priority Corridors

The National Highway System and its role in determining geometric design of highways is discussed in Section 40-1.03 (01) of Chapter 40 "Basic Design Controls" of the Indiana Department of Transportation's Design Manual.

Figure 4-4 shows the highways designated in Indiana as part of the National Highway System. It includes in the north-south direction: SR-69 from I-64 to Mount Vernon in the Kentucky border (Ohio River), US 41 from Terre Haute to Evansville and across the Ohio River to Kentucky, I-164 from I-64 to Evansville, SR-57 from the intersection with SR-67/US-231 to I-64, US-231 from the intersection with SR-67 to Kentucky border

(Ohio River) south of Rockport, and SR-67 from Indianapolis to the intersection with SR-57. In the east-west direction includes US-50 from the Ohio border to Vincennes in the Illinois border (Wabash River), I-64 for the Ohio border to Illinois border (Wabash River), SR-66 from Rockport on the Kentucky border (Ohio River) to Evansville, and SR-62 from Evansville to Mount Vernon on the Kentucky border (Ohio River).

Fig. 4-4 – Highways in Indiana designated as part of the National Highway System

From the methodological point of view it is evident that the routes in southwestern Indiana contained in both the National Truck Network and the National Highway Network had to be studied as candidates for the Earthquake Emergency Routes of Indiana.

Part I of this report presents different alternatives for Earthquake Emergency Routes for the State of Indiana based on trade off of three parameters, namely total travel time, population covered, and number of square feet of bridge deck (as a parameter associated with eventual retrofit cost).

The Indiana Department of Transportation provides maintenance within the State to Interstate Highways, U.S. Routes, and Indiana State Highways. County roads and city roads are excluded unless special agreements are in effect. The grid of routes presented in Fig. 4-5 corresponds to routes that belong to the National Truck Network, the National Highway System, and are within the domain of the Indiana Department of Transportation, and is presented as the preferred choice of Earthquake Emergency Routes of Indiana.

Fig. 4-5 – Selected grid of Earthquake Emergency Routes
in the Indot Vincennes District

The grid of routes presented in Fig. 4-5 comprises the following:

- US 41 from Terre Haute South to Evansville and the Kentucky border (Ohio River).
- SR 67 from Freedom to SR 57 junction and SR 57 from SR 67 to I-64.
- I-164 from I-64 to US 41.
- US 231 from SR 54 at Bloomfield south to Rockport and to Kentucky border (Ohio River).
- SR 37 from Bloomington south to SR 237 and SR 237 to Cannelton and the Kentucky border (Ohio River).
- SR 154 from Illinois border (Wabash River) to US 41, and SR 54 from US 41 east to SR 45.
- US 50 from Illinois border (Wabash River) at Vincennes east to SR 446 east of Bedford.
- SR 64 from Illinois border (Wabash River) to Princeton to US 41.
- I-64 from Illinois border (Wabash River) to SR 66 at Carefree.
- SR 57 from US 41 past Evansville Airport to I-164.
- SR 62 from Illinois border (Wabash River) thru Evansville to US 231
- SR 66 from US 41 east to Rockport.

4.2 Available Information

The Indiana Department of Transportation has a very well organized data base where information for the purposes of present project was obtained. The information used was the following:

- *Indot Inspection and Maintenance Database* – This database was modernized just before the initiation of the project. It consists in a MS Access database that contains records of all bridges being maintained by Indot. It follows Federal Highway Administration guidelines [FHWA, 1996] and contains additional information beyond the FHWA requirements. Only the portion related to bridges

in the Vincennes Indot District was used containing 827 bridges, distributed by Counties as shown in Table 4-1. See Fig. 4-6 for their location.

Table 4-1 – Distribution by County of Vincennes District bridges contained in Indot Inspection and Maintenance Database

County	No. Vincennes District Bridges in Database
Crawford	35
Daviess	22
Dubois	50
Gibson	66
Greene	41
Knox	42
Lawrence	36
Martin	51
Monroe	1
Orange	27
Owen	60
Perry	62
Pike	63
Posey	65
Spencer	60
Sullivan	40
Vanderburgh	121
Vigo	84
Warrick	72
Washington	1
Total	827

- *Bridge Drawings* – The bridge drawings at the Indot Vincennes District offices were consulted for bridges located on the selected Earthquake Emergency Routes. The drawings contain the original design plans, those corresponding to changes and modifications, and a description of the soil boring performed at the time of design of the bridge reporting SPT values in many cases.
- *GIS Information* – Geographical Information System (GIS) related to the State of Indiana, and other information was supplied by Indot. This information can also be downloaded from the following Indiana University webpage: <http://igs.indiana.edu/arcims/southwest/download.html>

Fig. 4-6 – Vincennes District bridges contained in Indot Inspection and Maintenance Database (those whose drawings were used in this research are marked as blue filled dots)

4.3 GIS Implementation

A two tier approach was developed for managing the information described in the previous section. The first stage consisted in developing tools for assessing the vulnerability condition of the studied Earthquake Emergency Routes. The second stage consisted in implementing the results in a dynamic database with Geographical Information Systems (GIS) capabilities.

For the first stage a database was developed as a search-engine to allow the static and dynamic evaluation of the condition of the selected critical routes prior to and following an earthquake disaster. Several specialized software programs were developed to act as

independent units — all of them interacting with the database as needed — for evaluating the different sources of vulnerability along the Earthquake Emergency Routes.

Three modules were developed, as shown in Fig. 4-7. The first module corresponds to a *Data Management Module*. This module handles the information related to the routes, and bridges. The information included is the one described in the previous section obtained from Indot Inspection and Maintenance Database.

Fig. 4-7 – Organization of the GIS implementation

The *Pattern Recognition Module* uses the information contained in the Data Management Module to process and obtain vulnerability assessment of bridge structures and geotechnical aspects of the bridges and routes and was used to feed information to the Scenario Management Module.

The *Scenario Management Module* is implemented on the extraordinary capabilities of HAZUS software developed by the Federal Management Agency [FEMA, 2005] that is capable of accepting the feed-back of the results obtained from the Pattern Recognition Module. HAZUS is capable of obtaining vulnerability assessments for any case in particular for mitigation purposes, allows simulations for different earthquake occurrence scenarios, and will allow in the future to be used when online capabilities are implemented within Indot for monitoring response to an actual earthquake occurrence.

4.4 Vulnerability Assessment Approach

The assessment of seismic vulnerability for a significant number of structures requires the use of identification of patterns of behavior of these structures to the envisioned earthquake ground motion. The approach used in comparable previous cases has been to define groups of structures having similar properties or configuration for which common behavior patterns can be defined. These behavior patterns have been referred in the literature as fragility curves relating probability of having a particular type of damage to a ground motion descriptor such as peak ground acceleration.

For implementation in this project several shortcomings were identified in using this approach:

- The inventory of bridges under study covers from bridges recently built to bridges built in the 1910's and 1920's. Figure 4-8 shows the year of construction of the Indot Vincennes District bridges. Establishing general fragility curves giving probability that a certain type of damage occurs for such a diverse inventory lead to many types of fragility curves and their use in establishing actual vulnerability may be questionable.
- Many of the bridges have been rebuilt (except bridges built after 1990) at different moment. Figure 4-9 shows the percentage of the Vincennes District bridges that have been rebuilt, and the average year when the rebuilding took place. The influence of such a significant percentage of bridges having been rebuilt makes the fragility curve type of approach even more difficult to apply.
- Associating the bridge vulnerability of a set value of seismic risk assessment as presented in a specific map, for example the one contained in Current 17th Edition of the Standard AASHTO Specifications, will require a new assessment if the map contained in the requirements changes. The adoption of new more modern specifications will surely take place in the near future, but the nature of the changes that will be introduced before adoption are unknown presently. The adoption of the proposed Draft LRFD Seismic Requirements has been voted

down by several states who have questioned the complexity and eventual unneeded extra cost that their adoption will bring.

Fig. 4-8 – Number of bridges built per decade (Vincennes District)

Fig. 4-9 – Percentage of bridges rebuilt according to decade of construction and average year when rebuilding was made (Vincennes District bridges)

Based on these limitations a different approach was implemented in this study. The approach used is based in selecting a number of bridges to study their seismic behavior under varying earthquake ground motion severity thus making the study independent of current requirements and making the results valid under mapped values having different return period or acceleration (peak ground or spectral) values. Using the results from the study of these bridges a procedure to extrapolate the results to the rest of the bridges in the Vincennes District was devised, requiring as information only that contained in the Indot Inspection and Maintenance Database. This establishes two levels of confidence on the vulnerability assessment. The first one is general and covers all bridges in the Inspection and Maintenance Database. The second one is more detailed because is based on the information contained in the bridge drawings. The advantage of this approach is that it permits to gage the vulnerability of the whole bridge inventory. It permits devising policies of general nature with respect to the selection, or variation in the future, of the Earthquake Emergency Routes. It also permits a more detailed assessment by supplying additional information to the GIS system data base, in case the detailed data is not already in the system. A detailed explanation of the procedure is given in Chapter 6.

4.5 Mitigation and Simulation

Vulnerability mitigation policies that cover the bridge inventory in southwestern Indiana are possible using the HAZUS GIS system. The assessment of the impact of a variation of the seismic risk maps or the simulation of occurrence of an earthquake with specific location and magnitude can be easily achieved by matching the new values to the already computed curves relating the bridge expected damage to the level of acceleration. If a bridge is retrofitted, the changes introduced to the bridge substructure or superstructure can be introduced to the database for future assessments or simulations.

CHAPTER 5 GEOTECHNICAL ASSOCIATED SOURCES OF VULNERABILITY

5.1 Introduction

This chapter describes the methodology implemented for processing the geotechnical information available for the selected bridges and to be used, in the future, as the database is expanded by adding more information from drawings or from soil explorations. The Inspection and Maintenance Database does not contain geotechnical information; therefore, the approximate vulnerability assessment does not include evaluation of the geotechnical sources of vulnerability.

Not all the studied bridge drawings contained soil exploration boring information. In the majority of cases studied the boring logs included in the drawings contained the soil description for each layer and reported the Standard Penetration Test (SPT) values at appropriate intervals. In rock sites many of the boring logs report the recovery percentage appropriately. In some instances the boring logs only report the soil classification for each layer without any SPT values. Textural description of the soil is the accepted practice for defining the soil classification in the field by the boring team to be reported in the boring log. In general, especially for older borings, the description is based on the U.S. Department of Agriculture textural classification chart [USDA, 1951] presented in Fig. 5-1. Percentages of sand, silt, and clay are established visually based only in the particle size. The soil classification description used corresponds to the one in the zone of the figure where the three percentage values meet. No laboratory procedures are involved in assigning the soil layer description. The AASHTO Soil Classification System or the Unified Soil Classification (ASTM D2487 — “Standard Classification of Soils for Engineering Purposes — Unified Soil Classification System”) require laboratory tests and are routinely performed on the soil samples and included later in the soils report.

Fig. 5-1 – USDA Textural Soil Classification Chart [USDA, 1951] routinely used for soil classification by soil exploration boring teams

The software implementation uses the soil exploration boring reports contained in the selected bridge drawings to study the geotechnical sources of vulnerability for each of the bridge sites. Other sites can be studied by adding information to the database in the future.

Two evaluations are performed using the boring information: a soil profile amplification assessment and a liquefaction potential assessment. These evaluations are made for varying peak ground accelerations in rock to be consistent with the adopted approach in this research project of obtaining the vulnerability for different levels of ground motion at the site and not to depend on a unique description of ground motion as contained in a particular set of seismic risk maps. Soil profile amplification and liquefaction potential are acceleration dependent; therefore, the values obtained cover different seismic risk levels.

The possibilities of evaluation are presented in the main screen of the software developed for this purpose, as shown in Fig. 5-2.

Fig. 5-2 – Main screen for the liquefaction evaluation module developed

This module permits to enter, save, and retrieve boring information for any bridge site borings, and to process it to obtain the soil profile amplification potential, evaluate the liquefaction susceptibility, and to save to results for further analysis.

5.2 Soil profile amplification

The amplification potential of the soil profile is needed for the liquefaction evaluation and for the bridge vulnerability assessment; in the former case for estimating the ground motion acceleration at the surface and in the later case for defining the ground motion description for evaluation of the bridge. The methodology used is based on the procedure contained in the 2000 NEHRP Provisions [NEHRP, 2000] as described in Section 2.3.1 of this report.

The first step is to define the site class (A to F) as presented in Table 2-11. The classification is made using average shear wave velocity ($\bar{v}_s$), average standard penetration resistance ($\bar{N}$), average standard penetration resistance for the cohesionless

soils only ($\bar{N}_{ch}$), and average undrained shear strength in case of cohesive soils ($\bar{s}_u$). The software implementation is capable of using the more appropriate parameter from the soil exploration and laboratory reports. For the selected bridges because only Standard Penetration Test are reported the amplification potential is evaluated from the average standard penetration resistance ($\bar{N}$) computed for the upper 100 ft (30 m) of the site profile using Eq. (5-1). Profiles containing distinctly different layers are subdivided into those layers by a number that ranges from 1 to n .

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (5-1)$$

where d_i is the thickness of any layer between 0 and 100 ft (30 m) and N is the Standard Penetration Resistance (SPT) not to exceed 100 blows/ft as directly measured in the field without corrections.

Once the site is classified as being Class A to E (Class F requires a site specific evaluation) the curves presented in Figures 2-24 and 2-25 are used to obtain the amplification parameters F_a and F_v for the acceleration value under study. Spectral acceleration (S_s or S_l) values are used directly to obtain F_a and F_v . If peak acceleration in rock is used (A) the value is multiplied by 2.5 to be used in Fig. 2-24 to obtain F_a and is used directly in Fig. 2-35 to obtain F_v .

The values obtained were compared with results using the program SHAKE as described for several sites in Indiana in [Bobet et al., 2001] obtaining good correlations of the amplification levels obtained.

Figure 5-3 shows the results obtained for a specific case. The figure reports that in this case (Bridge No. 41-42-04638) the bridge drawings included four borings. It also indicates that the procedure classified the soil profile in all borings as being Type E. The statistics shown in the screen indicate the number of borings that report a certain soil profile type. As opposed to buildings, where usually a single soil profile type is present, bridges by being extended structures may have different soil profile types under the abutments or at the bridge piers. This is specially true in alluvial plain sites, where many

of Indiana bridges are sited. Figure 5-3 also shows in graphic form the relationship between the horizontal acceleration in rock and the horizontal acceleration in surface for peak ground acceleration, or spectral ordinates (S_s and S_1).

Fig. 5-3 – Typical results for soil profile classification and amplification potential

In total 182 bridge sites located on the defined Earthquake Emergency Routes were evaluated. This includes all the bridges located on the Emergency Routes whose drawings contained soil boring records shown as part of them. The information was used directly to define the soil profile type for each boring using the procedure described. When reporting the soil profile type that causes the maximum amplification the results shown in Fig. 5-4 are obtained indicating that for 159 bridges out of 182 (87%) the soil profile is classified as Type E (see Table 2-11 for description of the soil profiles). Soil profile Type D is obtained in 16 bridges (9%), soil profile types (B and C) just carry 7 bridges (4%), and none are classified in soil profile Type A. These results are hardly surprising for the State of Indiana where very few rock type sites are found.

Notwithstanding, this situations has important implications from the point of view that mild earthquakes may produce moderate to high peak ground accelerations on surface due to the soil amplification caused by the intervening soil responding essentially in a elastic manner without the possibility of a decrease caused by the damping inherent in the nonlinear response of the soil ever occurring.

Fig. 5-4 – Worse soil profile type at bridge site for 182 bridges on the Earthquake Emergency Routes

5.3 Liquefaction

The liquefaction potential is evaluated using the procedure described in Section 2.3.2 of this report. The software implementation accepts information from field test in the form of cone penetration test (CPT), standard penetration test (SPT), or shear-wave velocity (V_s) measurements. The application of the procedure using the boring information available was restricted to evaluations using Standard Penetration Test values, but as more information is included in the database it is desirable to use more reliable field information such as CPT or shear wave velocity measurements for evaluations in the future.

For the selected bridge evaluation corrections, as described in Section 2.3.2, for fines content and earthquake magnitude were implemented. The corrections to SPT listed in Table 2-12 were implemented but not used (except overburden pressure and rod length corrections that were used in all cases) due to lack of information on the boring equipment and procedures as reported in the bridge drawings. As the database is expanded in the future the software implementation permits the use of these corrections if reported in the boring log.

Fines content in a soil can have a significant influence in the liquefaction potential of a soil layer. The procedure to account for the presence of fines in the evaluation of the liquefaction potential is to adjust the value of the corrected blow count $(N_1)_{60}$ using Eq. (2-22) to an equivalent value for clean sand, $(N_1)_{60cs}$. The effect of the adjustment is an increase of the value of $(N_1)_{60}$ effectively used. It is considered [Youd and Idriss, 2001] that clean granular soils having a value of $(N_1)_{60}$ greater than 30 are too dense to liquefy and are classed as non-liquefiable. For most soil classification procedures, as shown in Fig. 5-5, fines are considered particles that can pass through a No. 200 sieve corresponding to a 0.08 mm size particle.

Fig. 5-5 – Soil particle size according to several soil classification procedures [Winkerkorn and Fang, 1975]

Boring reports as presented in bridge drawings do not include the results from the sieve analysis. In order to adjust the fine content of the soils as reported in the boring logs the percentage of silt and clay was determined from the soil classification reported in the borings using the chart presented in Fig. 5-1 or alternatively for some soil descriptions not contained there using the values suggested by the Federal Aviation Administration (FAA) classification as shown in Fig. 5-6. The implemented software permits editing the value of the fines content for any soil layer when needed.

The influence of the fines content in the evaluation of the liquefaction potential cannot be underestimated and is recommendable that in the future, as more sites are included in the database, that fine content be reported from sieve analysis of the soil.

Fig. 5-6 – FAA textural classification of soils [Winkerkorn and Fang, 1975]

When evaluating liquefaction potential the expected number of cycles needed for the pore pressure increase development has been traditionally associated with the causing earthquake magnitude. The developers of the liquefaction evaluation procedure used the moment magnitude scale — M_w . Magnitude for the 1811-1812 New Madrid events has been estimated using different techniques, reporting values that range from a lower value

of 6 to an upper value of 9. The magnitude scale in which these values have been reported is uncertain and in some cases is not reported.

The preference for moment magnitude is based on being the most consistent scale along a wide magnitude range, but it is the scale that requires more instrumental information to define its assigned value. Recent studies made by the USGS [Frankel et al., 2002] for the 2002 seismic risk maps update have set a maximum characteristic magnitude for the central and eastern U.S. for purposes of seismic risk assessment. The scale in which the characteristic magnitude is expressed is moment magnitude and has direct use in liquefaction potential evaluation.

Figure 5-7 shows the maximum characteristic moment magnitude assessed for the central and eastern U.S. A value of $M_w = 6.5$ is given for all the state of Indiana, except the Wabash River Valley that is assigned a value $M_w = 7.5$. JTRP Project SPR 2812 on Analysis of Seismic Hazard Assessment for Indiana being currently conducted by Purdue Professors Haase and Nowack will surely give more insight on the values of M_w to be used and implemented in future liquefaction potential assessments. The software developed permits the variation of the magnitude to use.

Fig. 5-7 – Maximum magnitude for the Central and Eastern U.S. [Frankel et al., 2002]

It is interesting to note that if the proposed Draft AASHTO LRFD seismic requirements are adopted in Indiana, for the seven Indiana cities evaluated for comparison purposes in section 3.4.6 of this report any new bridge design would require liquefaction evaluation for the bridge site. Table 2-13 indicates the cases in which no liquefaction evaluation is needed for Seismic Detailing Requirements (SDR) 3 to 6 that require in all cases magnitudes lower than 6.4.

Typical results screens from the implemented software are shown in Figures 5-8 to 5-10. In Fig. 5-8 the description of the boring is given reporting the soil layers and the Standard Penetration Test (SPT) values and the water table level.

Fig. 5-8 – Example description of boring log as presented in the implemented software

In Fig 5-9 the safety factor for liquefaction is given. The software first searches for the lowest surface ground acceleration that will produce liquefaction — in this case it is 15% of the acceleration of gravity g — and reports the safety factor along the soil profile. In this case liquefaction occurs in a sand layer that reports a SPT value of 17 blows per foot at elevation 410. The peak ground acceleration in rock that produces the acceleration in surface leading to liquefaction corresponds in this case to 6% g . The soil profile reported by this boring is classified as being Type E and for a peak rock acceleration of 6% g the amplification factor, F_a , has a value of 2.5 thus converting the 6% g acceleration in rock into 15% g acceleration in surface.

Fig. 5-9 – Example description of liquefaction safety factor for the lowest ground surface acceleration inducing liquefaction. In this case 15% g leads to a safety factor of 0.9 meaning that liquefaction would occur

Different peak ground accelerations in rock can be evaluated. Figure 5-10 shows the results in the liquefaction potential as described by the liquefaction safety factor for peak ground acceleration in rock of 4%g leading to a ground surface acceleration of 10%g the safety factor against liquefaction increases to a value of 1.4 thus reporting no liquefaction. In this manner the safety factor is obtained for different acceleration scenarios.

Fig. 5-10 – Example description of liquefaction safety factor for a different ground surface acceleration from the one shown in Fig. 5-9 not inducing liquefaction. In this case the lowest safety factor for 10% g surface acceleration is 1.4

The factor of safety for liquefaction for a range of ground surface acceleration is stored in the database. This permits to handle different seismic risk scenarios without having to reprocess the boring information in each instance.

Bridge 050-42-06963 reported in the previous figures (Fig. 5-8 to Fig. 5-10) has description for four borings included in the drawings. Borings 1 and 3 are classified as being Soil Profile Type D while borings 2 and 4 are classified as Soil Profile Type E. The peak ground acceleration in rock required to produce liquefaction obtained were 6% g for borings No. 1, 2, and 4 and 4% g for boring No. 3.

The situation just described is present in many of the 182 bridges studied along the Earthquake Emergency Routes. The evaluation was performed for two values of M_w ($M_w = 6.5$ and $M_w = 7.5$). Table 5-1 presents a summary of the minimum peak ground acceleration in rock required to produce liquefaction at bridge sites studied. The information is presented for the two moment magnitude studied.

Table 5-1 – Minimum peak ground acceleration in rock required to produce liquefaction on the 182 bridges sites studied along the Earthquake Emergency Routes

PGA rock (g)	No. of Bridges			
	$M_w=7.5$	%	$M_w=6.5$	%
0.02	2	1.1%		0.0%
0.03	27	14.8%	3	1.6%
0.04	23	12.6%	18	9.9%
0.05	10	5.5%	24	13.2%
0.06	3	1.6%	10	5.5%
0.07	2	1.1%	7	3.8%
0.08	1	0.5%	2	1.1%
0.09	4	2.2%	2	1.1%
0.10	1	0.5%	1	0.5%
0.11	2	1.1%		0.0%
0.12	1	0.5%	1	0.5%
0.13	1	0.5%	2	1.1%
0.14		0.0%	2	1.1%
0.15		0.0%	1	0.5%
0.16		0.0%		0.0%
0.17	1	0.5%		0.0%
0.18	1	0.5%	2	1.1%
0.28		0.0%	1	0.5%
0.42		0.0%	1	0.5%
0.48		0.0%	1	0.5%
No liquefaction	103	56.6%	104	57.1%
	182		182	

With the minimum peak ground acceleration in rock needed to produce liquefaction at the bridge site it is possible to have a general picture of the vulnerability caused by liquefaction for the bridges along the Earthquake Emergency Routes. This information is presented in Fig. 5-11. In this figure the percentage of bridge sites that may be affected by liquefaction when the peak ground acceleration in rock reaches a certain value may be obtained. At peak ground accelerations in rock of the order of 10% g the curve flattens, meaning that the number of bridges sites susceptible of liquefaction would remain essentially the same even for higher accelerations. At this level of acceleration in rock approximately 40% of the bridges located along the Earthquake Emergency Routes would have reported liquefaction. This acceleration is of the same order of magnitude of those contained in current *Division I-A of AASHTO Standard Specifications*.

Fig. 5-11 – Percentage of bridges along the Earthquake Emergency Routes that will not have liquefaction at the site for a given peak ground acceleration in rock computed using the minimum acceleration to produce liquefaction at the site

Liquefaction potential was computed at all borings for each of the studied bridge sites. Only in few cases the minimum acceleration needed to produce liquefaction is the

same for all borings. If instead of reporting the minimum peak ground acceleration in rock the average acceleration for those borings reporting liquefaction is computed, the vulnerability situation changes. Figure 5-12 shows the information computed for the average peak ground acceleration in rock to produce liquefaction at the site. The curves shown flatten at a higher acceleration (15% g) but show a similar trend. Using the average peak ground acceleration in rock at the bridge site, the number of vulnerable bridges due to liquefaction at the level of acceleration of the current AASHTO Standard Specifications reduces to approximately 35% of the bridges on the Earthquake Emergency Routes.

Fig. 5-12 – Percentage of bridges along the Earthquake Emergency Routes that will not have liquefaction at the site for a given peak ground acceleration in rock computed using the average acceleration to produce liquefaction at the site

The primary concern when interpreting the data just presented is related to the difference in peak ground acceleration in rock values for the same bridge site. If the difference between the minimum accelerations to produce liquefaction and the mean acceleration for the same bridge site is relatively large, it indicates significant variations

in the soil profile recorded in the soil exploration borings for the same bridge site. The ratio of mean to minimum peak ground acceleration producing liquefaction at the same bridge site has values that range from 1 to 6 with an average of the order of 1.4 for the bridge sites studied, indicating significant variations in the soil profiles.

To emphasize the need for engineering judgment in interpreting the results presented, the following example brings out features found in many of the bridges located in the Earthquake Emergency Routes. Bridge 50-42-04625 on the Wabash River carries US Route 50 across the river to Illinois. The bridge drawings contain records of 24 soil borings made in 1960 all of them containing appropriate information (soil description and standard penetration test – SPT – logs) to evaluate the soil amplification potential and the liquefaction potential at each boring. The evaluation of the Soil Profile Type indicates that the profile at one boring would be Type B, at five borings Type D, and Type E at the remaining 18 borings. The general classification of the bridge site based on the softer profile would assign the bridge site as being Type E. The liquefaction evaluation for the borings report possibility of liquefaction in 11 out of the 24 borings for magnitude $M_w = 7.5$. Table 5-2 presents the peak ground acceleration in rock required for producing liquefaction at each of the borings reporting liquefaction. In the same table the minimum and maximum peak ground acceleration obtained is reported as well as the mean and standard deviation for the acceleration.

In order to interpret the liquefaction potential computation for the studied bridges besides the information presented, two other variables were studied:

- Percentage of borings reporting liquefaction of the total number of borings.
- Ratio of the range of peak ground acceleration in rock producing liquefaction to the mean peak ground acceleration.

For Bridge 50-42-04625 the percentage of borings reporting liquefaction was 58% and the ratio of the range of peak ground acceleration in rock producing liquefaction to the mean peak ground acceleration was $(0.36-0.05)/0.10 = 3.1$. These values were computed for all bridges reporting liquefaction and based on the values obtained a recommendation was formulated dividing the bridges studied into four groups designated as: bridges having no liquefaction, bridges with a high probability of liquefaction, bridges

with an appreciable probability of liquefaction, and bridges having liquefaction but were a wide variability of the results is present that require additional soil exploration and a more in depth liquefaction study performed by geotechnical consultants.

Table 5-2 – Minimum peak ground acceleration in rock required to produce liquefaction on the 11 borings that report liquefaction for Bridge 50-42-04625

Boring No.	Peak Ground Acceleration in Rock for liquefaction
14	8% g
15	8% g
16	9% g
17	7% g
18	6% g
19	6% g
20	8% g
21	36% g
22	5% g
23	7% g
24	7% g
Min. PGA_{rock}	5% g
Max. PGA_{rock}	36% g
Mean PGA_{rock}	10% g
Standard Deviation	9% g

Figure 5-13 presents the percentage of bridges studied that fall into the recommendation categories for the two moment magnitude studied.

Other considerations that are beyond present study are related to issues related to variation of the soil profile as used in the evaluation that may have occurred during the years since the borings were performed. Other important consideration to take into account is related to the water table at the site. Liquefaction can only occur in relatively loose granular soils located under the water table. The potential for liquefaction studied and reported used the water table level reported in the borings. The possibility of variations of the water table level due to seasonal effect with respect to the moment the borings were performed can affect the results presented. This is specially important for

those cases not reporting liquefaction due to a depressed water table in cases where loose granular soils exist at the site.

Fig. 5-13 – Recommendation for bridges located along the Earthquake Emergency Routes

Using has HAZUS-MH [FEMA, 2005] — a GIS-based software tool developed by the Federal Emergency Management Agency (FEMA) — Indot bridge inventory was assessed for the liquefaction potential that was evaluated using the actual soil borings included in the bridge drawings, as described before. The Indot bridge data base was integrated within HAZUS-MH and the GIS functions were activated. The liquefaction potential was evaluated using HAZUS-MH own liquefaction potential evaluation routines. This was performed for the $M_w = 6.5$ earthquake scenario obtaining comparable result.

5.4 Soil spread

For a soil spread situation to occur during earthquake liquefaction has to occur. The soil spread scenarios are difficult to state and requires analysis and studies beyond what

can be implemented in application software of the type implemented in this project. In a previous JTRP project [Bobet et al., 2001] soil spread scenarios and consequences were studied for several sites in southwestern Indiana indicating the possibility of their occurrence. The study also indicated that the Indiana Department of Transportation practice of using steel H piles and steel encased concrete (SEC) piles reduces significantly the potential damage to piles during earthquakes. This study recommends mitigation studies and implementations that should be adopted for all bridge structures located along the Indiana Emergency Routes. A warning should be issued when liquefaction may occur and the bridge is supported on footings. The same type of warning is given when possibility of liquefaction is identified below the tip of piles, situation that seldom occurs.

5.5 Embankment stability

The absence of information on embankment slope geometry and description of mechanical properties of the fill material both in the Inspection and Maintenance Database and the selected bridge drawings precludes the inclusion of test cases for the Earthquake Emergency Routes. Notwithstanding, test cases studied for fill material commonly used in Indiana indicate that for embankments with slopes of 1 vertical to 2 horizontal there is a low probability of instability in the embankment itself for ground accelerations up to the order of 15% g and embankment heights lower than 30 ft (10 m). The possibility of liquefaction or other type of ground failure under the embankment must be studied independently along the routes. Information on soil profile under the embankment and properties of the fill material probably would require soil exploration for that purpose along the Emergency Routes.

CHAPTER 6 BRIDGE VULNERABILITY

6.1 Introduction

This chapter describes the methodology implemented for assessing the bridge seismic vulnerability by processing information available for all bridges located in the Vincennes District of the Indiana Department of Transportation and for selected bridges whose drawings were used. The information contained in the Indot Inspection and Maintenance Database was used directly for obtaining an approximate evaluation of the bridge seismic vulnerability. The drawings of selected bridges were used for obtaining a more reliable vulnerability assessment of each selected bridge. The results of the analysis of the bridges whose drawings were used to calibrate the approximate procedures based on the Inspection and Maintenance Database information.

6.2 Information from the Inspection and Maintenance Database

Indot Inspection and Maintenance Database is implemented in MS Access and contains information descriptive of the bridge that is relevant from the inspection and maintenance management point of view. Only part of the information contained in the database is related to structural information and although general in nature, lends itself to assign the bridge within broad seismic behavior categories.

The information from the database that was used directly in the approximate vulnerability assessment can be grouped in the following general categories: for database indexing purposes, relative to location, road under or over the bridge, date of construction and repair, general bridge geometry, superstructure characteristics including deck information, substructure characteristics, and information on the approaches and abutments. Table 6-1 presents the database keys that were used for processing the approximate vulnerability assessment.

Table 6-1 – Basic information from Indot Inspection and Maintenance Database
that was included in the analysis

Group	Information
Database	Number of Records
	Total No. of Bridge Records
	Record No.
File Data	National Bridge Inventory No.
	INDOT Bridge No.
	District Code
	County Code
	Sub-district No.
	City Town Code
	Location
Road Data	Facility Carried
	Feature Intersected
	Road Number Over
	Road Number Under
Age Data	Year Built
	Structure Designation
	Year Reconstructed
	Sufficiency Rating
Geometry Data	Number of Intermediate Piers
	Approach Structure Type
	Deck Width
	Skew Angle
	Structure Length
	Minimum Vertical Clearance Over
	Minimum Vertical Clearance Under
	Total Horizontal Clearance Over
Superstructure Data	Largest Vertical Distance
	Structure Type
	Length Maximum Span
	Number of Main Spans
	Deck Structure Type
	Deck Thickness
	Bearing Diaphragm
	Bridge Joint Type Interior
	Bridge Joint Type (N/E)
	Bridge Joint Type (S/W)
	Cross Bracing
	Hinge Pin Connection
	Intermediate Diaphragms
	Main Structure Type
	Number of Approach Spans
	Number of Beams
	Number of Floor Beams
	Number of Girders
	Number of Stringers
	Redundancy Code
Substructure Data	Abutment Type (W/S)
	Abutment Type (E/N)
	Approach Material
	Approach Construction

6.3 Approximate vulnerability assessment methodology

The approach to an approximate evaluation of vulnerability in many instances has been based just on the type of bridge. This methodological approach depends on general correlations for each bridge type, usually called fragility relationships, between probability of reaching a certain performance level (total collapse, partial collapse, repairable damage, or other) and expected ground acceleration at the bridge site. Results from this type of analysis are applicable, within the uncertainties inherent to this type of methodology, at a national or state level. As the size of the sample becomes smaller the results obtained are less reliable especially if diverse bridge types of varying age are present in the sample. In this research project it was considered that the information contained in the Inspection and Maintenance Database is appropriate to determine, in an approximate but more reliable manner, the relevant parameters that are needed to establish a vulnerability level. The relevant parameters considered are:

- Mass of the bridge
- Stiffness of the substructure
- Toughness available for adequate nonlinear response
- Strength of the lateral load resisting elements

The Inspection and Maintenance Database contains information from which the values of the relevant parameters can be estimated as follows:

The mass of the bridge is associated mainly with the mass of the superstructure and the deck is generally the main contributor. The Inspection and Maintenance Database lists the deck material, and deck width and thickness; thus permitting to obtain a good approximation of its mass. Allowances for wearing surface, barriers and, guard rails are also included. The number of girders, beams, and stringers; and the structural material is listed in the database. Their mass can be estimated as a function of a span dependent depth that is characteristic of each superstructure type. The contribution to mass of the diaphragms and bracing elements can also be estimated from the number and type as listed in the database. Having an estimative of the superstructure mass and geometry it is possible to establish minimum cross-sectional dimensions required to support the gravity effects for the bents based on the adjacent span lengths and structural type. Vertical

clearance indicates the height of the bents. Results from these estimates were compared with mass computed for the selected bridges whose drawings were used in the formal vulnerability assessment and with comparison of projects whose information was available in the literature.

The lateral stiffness of the bridge depends on the substructure type whose minimum dimensions were established in the computation of mass. The Inspection and Maintenance Database does not describe if the bridge is supported on frame or wall type bents, thus the estimate of the lateral stiffness is approached from the low side. Adjustments are made to this estimate for live load allowance especially in short span bridges.

Using the estimative of mass and lateral stiffness fundamental vibration periods for the transverse and longitudinal directions are computed taking into account the expansion joint and hinge pin connection descriptions contained in the database. Lateral force demand, longitudinal and transverse, is established for varying values of ground acceleration. Expected lateral deformation of the superstructure and base shear demand on the substructure is then estimated for each acceleration level.

The level of toughness required for adequate nonlinear response of the substructure elements is associated directly with the date of construction of the bridge. Lateral load strength is associated directly with the estimated substructure dimensions.

Bridge vulnerability is then established for each acceleration level based on excessive lateral deformations or lack of strength to resist the imposed base shear. For each acceleration level the bridge is red-tagged meaning vulnerable, yellow-tagged meaning marginal vulnerability, and green-tagged meaning not vulnerable.

These computations are made for all bridges in the Vincennes district inventory. For selected bridges the results were compared to the results obtained using the actual dimensions and reinforcement from the bridge drawings.

6.4 Formal vulnerability assessment methodology

The implemented software permits to include information from the bridge drawings in the vulnerability assessment database. This was performed for selected bridges as previously mentioned. The information covered is divided as shown in Table 6-2.

Table 6-2 – Basic information from bridge drawings needed for formal vulnerability assessment and included in the software implementation database

Group	Information	Detail
Database	File management	Same Information from Inspection and Maintenance Database
Bridge Information	General Bridge Information	Number of spans, bents, element types, units (US customary or metric SI)
		Number of vibration modes to use
		Use of rigid zones in bents
	Deck and Span Information	Deck width, thickness, additional mass,
		Span Information
Structural Elements	Geotechnical Information	Deck expansion joints type and location
		Same as described in section 5.2
		Cross-section dimensions
	Bent Column Information	Material information
		Modeling parameters (cracking, effective area, nonlinear characteristics)
		Cross-section dimensions
	Bent Beam Information	Material information
		Modeling parameters (cracking, effective area, nonlinear characteristics)
		Cross-section dimensions
	Bearings Information	Type
		Dimensions
	Restrainers	Type
		Dimensions
		Material properties
	Piles	Pile types
		Piles dimensional properties
		Piles material properties
	Footings	Footing Type
		Footing dimensional properties
		Footing material Properties
		Piles geometrical distribution
	Abutments	Abutment Type
		Abutment dimensional properties
		Abutment material Properties
		Piles geometrical distribution
Bents	Bent Types	Geometry
	Bent Location	Element type
Ground Motion	Spectrum	Location in bridge
	Accelerogram	From Bridge Specs
	Spectrum computed from accelerogram	From accelerogram database
	Increasing ground motion definition	Computation of spectrum from ground motion and smoothing possibilities
		Parameters to obtain performance patterns by variation of ground motion intensity

The information gathering process for each bridge is initiated including the information contained in the Inspection and Maintenance Database. The rest of the data must be provided from information contained in the bridge drawings.

Using this information several tasks were performed once a process is initiated. Some of these tasks are performed only once in the process — routine tasks — while others are performed repeatedly with each level of intensity of ground motion — ground motion intensity-dependent tasks—.

6.4.1 Routine tasks

Those processes that are not dependent on the ground motion intensity were performed first. They comprised the following tasks:

- Computation of element section properties.
- Computation of mass and stiffness properties for the superstructure and substructure elements.
- Computation of transverse and longitudinal vibration periods and modal shapes.
- Definition of strength in flexure and shear for all elements. For bent girders this was performed at sections where maximum stresses are expected under seismic effects. For bent piers and walls flexure-axial force interaction diagrams were computed. Reinforcement anchorage strength at joints was evaluated.
- Collapse mechanisms were evaluated for all bents defining the maximum tributary base shear strength. Distinction was made between flexural and shear collapse mechanisms.
- The lateral-load strength of bearings was evaluated. For bearings vulnerable to overturning or sliding under lateral load, the maximum transmitted shear before overturning or sliding was evaluated. Maximum available seat lengths were defined at non-fixed joints.
- For abutments and foundation elements, stiffness and deformation properties were evaluated. Strength for shear and overturning were evaluated.

From the above computed information a path of least strength was defined. For example bent piers were evaluated for the possibility of reaching shear strength before flexural strength thus initiating a brittle failure mode. At pier-foundation or wall-foundation joints the ability to transmit the forces developed when the pier reaches its maximum strength was evaluated, thus defining a weak link in the seismic load path. The maximum seismic ground-motion induced-forces that the structural system is capable of resisting were defined from the load path evaluation. These forces indicate the level of seismic base shear that may induce severe damage or collapse of the bridge.

6.4.2 Ground motion intensity-dependent tasks

The processes that depend on the ground motion intensity were performed for increasing levels of intensity of the ground motion. The ground motion intensity was defined in rock as peak ground acceleration, A_g . For a set value of peak ground acceleration in rock the following tasks (see Fig. 6-1) were performed in the order presented:

1. From the geotechnical information the soil profile amplification was determined using the procedure described in Section 5.2. This permitted the computation of spectral ordinates that are comparable to those defined through parameters S_S and S_I as described in the *Draft AASHTO LRFD Seismic Provisions*. This fully described the response spectrum for which the rest of the vulnerability assessment for the set ground motion was made.
2. The liquefaction potential was evaluated for the set acceleration value and reported. No further analysis on collateral potential of damage to the bridge caused by liquefaction was performed as explained in Section 5.4.
3. Structural element internal forces were obtained using a linear modal spectral response procedure based on the vibration periods and modal shapes computed in the routine tasks and the response spectrum from Step 1. Strength demands were compared with the strength capacity levels obtained in the routine tasks. A similar comparison was performed for the displacement demand against the maximum allowable displacement obtained in the routine tasks. The bridge was deemed not vulnerable for the set intensity of seismic ground motion and green-tagged and no

further analysis was performed if: a) the strength demand in all the structural elements was not exceeded, b) the displacement demand was within the established tolerable limit, and c) displacements at the supports did not exceed the limits established previously in the routine tasks,. If any of the strength or displacements limits were exceeded tasks 4 to 7 were performed.

4. If the shear strength level or the evaluation of reinforcement anchorage indicated possibility of a bond failure, the bridge was deemed vulnerable. The bridge was yellow-tagged or red-tagged depending on the level of expected damage and the spread of damage to the different elements.
5. If the possibility of bearing element failure by overturning or excessive displacement at movable supports was detected, the bridge was deemed vulnerable. The bridge was yellow-tagged or red-tagged based on the amount of expected deformation and the magnitude of potential horizontal or vertical permanent displacements of the superstructure.
6. The bridge was updated to red-tag if compound yellow-tagging was obtained both from the strength and the displacement checks. No further analysis was performed and the vulnerability level was reported for the set value of ground acceleration if steps 4, 5, or 6 yellow-tagged or red-tagged the bridge.
7. If the flexural strength for any element was exceeded in the evaluation performed in step 3, a nonlinear response evaluation was made. A substitute-structure procedure [Shibata and Sozen, 1976] was performed by introducing damage ratios, μ , greater than unity to the elements where flexural strength was reached. A series of analyses were carried out for increasing values of the damage ratios to the elements until the base shear strength of the bridge as computed in the routine tasks was reached. The expected lateral displacement for the set value of ground motion intensity was the one obtained when the bridge reached the base shear strength. This procedure gives comparable results to those obtained by performing a push-over analysis. The stability of the structure was judged for the displacements obtained using displacement-based procedures. Based on these

results the vulnerability of the bridge was assessed as yellow-tagged or red-tagged.

Fig. 6-1 – Flow diagram for the formal vulnerability assessment implemented

CHAPTER 7 UPGRADE NEEDS OF BRIDGES ON THE EMERGENCY ROUTES

7.1 Introduction

The approach of developing and implementing a tool for studying the alternatives for defining the Indiana Earthquake Emergency Routes and the upgrade needs of the transportation structures along these routes responds to the importance of two variables linked to the emergency routes selected, and the definition of the earthquake ground motion intensity for which the vulnerability of the current inventory of bridges should be gauged. The effect on the upgrade needs of these two variables is substantial. Variations of the road segments to be included within the emergency routes affect the travel times and the number of bridges that may need upgrading. Once a set of routes were defined, the envisioned earthquake ground motion intensity to be used in defining the vulnerability of the transportation structures affects directly the upgrade needs for these structures.

7.2 Bridges studied

Table 7-1 shows the distribution by bridge type of the inventory of bridges in the Indot Vincennes District. Currently there are 827 bridges that are located in the Interstate System, in US Routes, and in State Roads. The distribution by location in the different counties was presented in Table 4-1. A total of 230 of these bridges are located in the selected Earthquake Emergency Routes of Indiana, corresponding to 28% of the inventory. Table 7-1 also shows the distribution by bridge type of the 69 bridges that were studied in detail using the information contained in drawings. The sample was defined trying to maintain the bridge type percentages similar.

Table 7-1 – Distribution by bridge type in the Vincennes Indot District of the total bridge inventory, in the suggested emergency routes, and in the bridges studied in detail

Bridge Type	Bridge Type Description	Amount in Vincennes District	% by bridge type	Located on EQ Emerg. Routes	% by bridge type
BT	Bailey Truss	1	0.1%		
CCTB	Continuous Concrete T - Beam	1	0.1%	1	0.4%
CPCBB	Continuous Prestressed Concrete Box Beam	23	2.8%	4	1.7%
CPCIB	Continuous Prestressed Concrete I - Beam	69	8.3%	22	9.6%
CPCTB	Continuous Prestressed Concrete T - Beam	5	0.6%	2	0.9%
CRCB	Continuous Reinforced Concrete Box	1	0.1%		
CRCG	Continuous Reinforced Concrete Girder	27	3.3%	15	6.5%
CRCRF	Continuous Reinforced Concrete Rigid Frame	1	0.1%	1	0.4%
CRCS	Continuous Reinforced Concrete Slab	162	19.6%	53	23.0%
CRPG	Continuous Riveted Plate Girder	3	0.4%	2	0.9%
CSB	Continuous Steel Beam	89	10.8%	46	20.0%
CSG	Continuous Steel Girder	16	1.9%	10	4.3%
CSTA	Continuous Steel Tied Arch - Truss	2	0.2%		
KCSB	Composite Continuous Steel Beam	51	6.2%	31	13.5%
KCSBG	Composite Continuous Steel Box Girder	1	0.1%		
KCSG	Composite Continuous Steel Girder	56	6.8%	33	14.3%
KSB	Composite Steel Beam	10	1.2%		
KSG	Composite Steel Girder	8	1.0%		
MPA	Metal Pipe Arch	1	0.1%		
MPAUF	Multi-Plate Arch - Underfill	23	2.8%		
PCAUF	Precast Concrete Arch - Underfill	3	0.4%		
PCB	Precast Concrete Beam	10	1.2%	2	0.9%
PCBB	Prestressed Concrete Box Beam	61	7.4%	2	0.9%
PCIB	Prestressed Concrete I - Beam	31	3.7%	1	0.4%
PCS	Precast Concrete Slab	1	0.1%		
PCSUF	Precast Concrete Slab - Underfill	3	0.4%		
PTRCS	Post-Tensioned Reinforced Concrete Slab	1	0.1%		
RCA	Reinforced Concrete Arch	32	3.9%		
RCAOS	Reinforced Concrete Arch - Open Spandrel	1	0.1%		
RCAUF	Reinforced Concrete Arch - Underfill	17	2.1%		
RCBUF	Reinforced Concrete Box - Underfill	16	1.9%		
RCG	Reinforced Concrete Girder	51	6.2%	2	0.9%
RCS	Reinforced Concrete Slab	21	2.5%	1	0.4%
RCSUF	Reinforced Concrete Slab - Underfill	2	0.2%		
RPG	Riveted Plate Girder	2	0.2%		
SB	Steel Beam	6	0.7%	1	0.4%
SPT	Steel Pony Truss	6	0.7%		
STT	Steel Thru Truss	9	1.1%		
UCA	Unreinforced Concrete Arch	1	0.1%		
WSTG	Welded Steel Thru Girder	3	0.4%	1	0.4%
Sum		827	100.0%	230	100.0%

Figure 7-1 shows the distribution by bridge type on the Earthquake Emergency Routes. The most abundant type of bridge (23%) corresponds to Continuous Reinforced Concrete Slab bridge type both in the whole district and in the selected emergency routes. Continuous Steel Beam bridges (20%) and Composite Continuous Steel Girder (14.3%) correspond to the next largest groups.

Fig. 7-1 – Bridge type distribution on the Earthquake Emergency Routes

7.3 Retrofit Needs

The current inventory of bridges in the Vincennes Indot District was studied for varying degrees of earthquake ground motion intensity using an approximate vulnerability assessment methodology. The same approach of varying the intensity of the ground motion was performed for the selected bridges that were studied in detail.

As expected, bridge vulnerability and liquefaction potential in some bridge sites increased with the ground acceleration level. The importance of the definition of the

ground motion intensity for which the vulnerability level is assessed is explained by the number of bridges that would be red-tagged — 7% of the total Vincennes District bridge inventory — when the envisioned design ground motion contained in current 17th Edition of the AASHTO Standard Specifications are used as compared with a 15% when the ground motion description contained in the Draft AASHTO LRFD Update.

The vulnerability assessment indicates that the most frequent cause of concern is associated with unseating of the superstructure at supports (65% of bridges red-tagged) followed by shear failure of bent vertical elements (25% of bridges red-tagged). Other sources of vulnerability studied account for the rest of the red-tagged bridges. This figure is consistent for the whole inventory and for the selected bridges studies in detail. Bridge age correlates directly with vulnerability in the entire sample.

Overturning of the widely used movable expansion support of the type shown in Fig. 7-2 was found to be associated with many of the cases of unseating detected. In some instances peak ground accelerations in surface as low as 10%g would cause overturning.

Fig. 7-2 – Typical Expansion Rocker Bearing

Liquefaction as a source of vulnerability was present in the sample studied using the bridge drawings that reported soil borings. 182 bridges out of 230 reported a geotechnical exploration that could be used for evaluating liquefaction with 43% of them reporting sand or sandy soil and a water table level above the material susceptible of liquefaction. Lowest peak ground acceleration in rock that would produce liquefaction was below 5% g in several sites. The average peak ground acceleration in rock to produce liquefaction for the 79 sites where potential for liquefaction was detected was found to be approximately 5% g for the $M_w = 7.5$ scenario and 8% g for the $M_w = 6.5$ scenario.

The HAZUS-MH implementation produced comparable results indicating the approach of using the FEMA methodology is warranted.

7.4 Improvement of the approximate vulnerability assessment

The approximate seismic vulnerability assessment based on the information contained in Indot Inspection and Maintenance Database can be improved in the future by including additional information in the database. This additional information already exists in each set of bridge drawings. Although the drawings are available in electronic form they were scanned from hard copies and this format is not amenable for automatic retrieval of the information thus requiring participation of an experienced engineer for collecting and adding the information to the vulnerability assessment database.

There is a natural tradeoff between the effort of adding and maintaining this additional information and the degree of resolution sought in the approximate assessment procedure. The possibility of making a more detailed vulnerability analysis as implemented for selected bridges constitutes the upper limit of the additional information to be included. This may be unnecessary for a properly calibrated approximate procedure. The amount of additional information must be selected in a manner that maximizes the quality of the result and minimizes the data acquisition effort.

Based on this, the minimum suggested set of additional information is the following:

- Description of bent type and element dimension for all supports of the bridge.
- Reinforcement at selected sections of the bent elements focusing mainly on transverse reinforcement.

- Additional information on bearings more from the point of view of vulnerability to seismic effects.
- Additional information on foundation types and soil profile.

This additional information may be collected when the routine programmed inspections of the bridge are performed.

The improvement in dependability on the approximate vulnerability analysis with just this additional information may well justify the information gathering effort.

CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Different alternatives for Earthquake Emergency Routes for the Vincennes District of the Indiana Department of Transportation (INDOT), located in the southwestern corner of the State, were studied from the transportation point of view (Part I of this report) and the implications of seismic vulnerability of the transportation structures located in these roads were studied (Part II of this report).

The Indot bridge inventory data and soils information were incorporated in a software tool based on geographical information systems technology (GIS) developed by FEMA (HAZUS-MH [FEMA, 2005]) for evaluation of the Earthquake Emergency Routes network of Indiana and for establishing policies for mitigation of earthquake hazard on these routes. This tool may be used to perform simulations for different earthquake scenarios, and to help in the response to an actual emergency caused by the occurrence of a strong earthquake in the region.

The software tool is capable of performing a seismic vulnerability approximate assessment of the bridge inventory of the Vincennes Indot District using solely the information contained in Indot Inspection and Maintenance Database. Using information from the bridge drawings the software tool developed allows performing a detailed seismic vulnerability assessment of selected bridges. The vulnerability assessment performed for the selected bridges was used to calibrate the approximate analysis procedure based only on Indot Inspection and Maintenance Database and to evaluate the vulnerability of the selected bridges for different levels of earthquake ground motion intensity.

The reported research will help in the study and adoption of emergency response policies for different earthquake occurrence scenarios. These policies may be updated in the future as mitigation programs are implemented and their results incorporated in the bridge vulnerability assessment database.

The reported comments on the current state of seismic risk evaluation for the southwestern region of Indiana will help the State of Indiana in making decisions related to the adoption of proposed bridge seismic design provisions or proposed seismic risk maps by permitting an evaluation of the impact of the new provisions or seismic risk maps.

8.2 Recommendations

It is suggested that the findings and developments of this research project may be applied by the Indiana Department of Transportation in the following ways:

1. Indot should consider formally adopting the Earthquake Emergency Routes for the State of Indiana from the routes studied, or variations of them. It is clear that the final set of Earthquake Emergency Routes adopted and maintained will be affected by issues and policies whose scope is outside those studied and reported in this research project. Policy decisions, budgetary constraints, and decisions related to new road projects, such as the projected south western continuation of I-69, will affect the formal adoption of the selected routes and will surely impose variations in the future. The methodology developed in this research project will provide valuable information to assist Indot the decision-making process presently and in the future.

2. The study and adoption of mitigation strategies to implement in the future by Indot to update and maintain the Emergency Routes of Indiana can be made possible by performing simulations for different earthquake scenarios including earthquake ground motion defined in current AASHTO Standard Specifications, proposed Draft AASHTO LRFD Seismic Provisions, or other appropriate bridge specification. These simulations will provide corresponding bridge retrofit scope for the selected routes or alternative definitions.
3. It is recommended that Indot expands the information contained in the Inspection and Maintenance Database to include information currently available only in the bridge drawings (see section 7.5). This will improve the vulnerability assessment of the bridges refining the mitigation policies to adopt in the future. This expansion of the database can be extended to other counties and districts not covered in this research project.
4. It is recommendable that Indot adopt a program for assessing the liquefaction, soil spread, and embankment failure potential along the routes at locations different from just bridge sites by providing a consistent evaluation methodology based on the implementation developed in this research project. This implementation will require a soil exploration program to provide geotechnical properties of sites where no information exists or was performed many years ago with different objectives than liquefaction evaluation. In the reported research liquefaction evaluations were made using solely boring information contained in selected bridge drawings.
5. It is recommended that FEMA HAZUS-MH software be adopted as the methodology for vulnerability assessment, mitigation decisions through the study of appropriate earthquake occurrence scenarios, and emergency response programs tuned to the study of these scenarios. Along with this

recommendation a key feature is the training of Indot personnel in the use of FEMA HAZUS-MH software. Engaging of the services of the Polis Center of Indiana University, an authorized HAZUS-MH earthquake and flood service provider, for this purpose should be studied by Indot.

CHAPTER 9 REFERENCES

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Final Report

FHWA/IN/JTRP-2003/22

**EMERGENCY EARTHQUAKE ROUTES
FOR THE STATE OF INDIANA**

*PART I
CRITERIA FOR SELECTION OF PRIMARY ROUTES:
TRANSPORTATION ASPECTS*

by

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INDOT Research

TECHNICAL *Summary*

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Final Report

Emergency Earthquake Routes for the State of Indiana (A Two-Volume Report)

Introduction

The possibility of very strong earthquakes in the central Mississippi Valley has been known for 200 years in the New Madrid Seismic Zone. Awareness of the seismic risk in the State of Indiana has increased in the last decades. In the last two decades identification of an independent tectonic process occurring in the Wabash River Valley has implications for the seismic risk of the State of Indiana from what has recently been defined as the Wabash Valley Seismic Zone. The occurrence of a strong earthquake in these seismic zones or in Indiana or the neighboring states due to other not so well understood tectonic processes, as has occurred in the past, has serious implications for the State Transportation System. The definition of Earthquake Emergency Routes for the state is a priority for the Indiana Department of Transportation. The identification of Emergency Routes takes into account issues related to transportation including coverage of population and area and travel time along these routes.

This study formulates a multicommodity maximal covering network design problem (**MCNDP**) for identifying critical routes, for earthquake response and to seismically retrofit bridges. The **MCNDP** seeks routes that minimize the total travel time over the selected routes and maximizes the total population covered, subject to a budget constraint on bridge retrofitting costs on the selected routes. The problem is formulated as a two-objective

integer programming model and solved using the branch-and-cut module in the CPLEX optimizer. The model performance was analyzed using the transportation network of a seismically-prone region in southwest Indiana. Thereby, the search for the critical routes for an origin-destination (O-D) pair is confined to a limited geographical region around it.

A Geographical Information System (GIS) system was developed for evaluation of seismic vulnerability of the proposed Indiana Emergency Routes. The GIS based implementation of the technical information developed in this project through FEMA HAZUS-MH software was conducted in collaboration with the Polis Center in Indianapolis, nationally recognized and FEMA authorized HAZUS implementation facility. HAZUS software may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in southwestern Indiana. It uses information from the Indiana Department of Transportation (INDOT) maintenance database and selected structural and geotechnical information from the bridge drawings data base. Vulnerability assessment was performed using the general information from maintenance and final calibration was performed using a series of cases based on bridge drawings and soil exploration descriptions contained in these drawings.

Findings

Computational experiments were conducted to investigate the effects of varying the budget and the relative weights of the two objectives. A set of

critical routes was obtained. Seismic retrofit planning plays an important role in determining the effectiveness of the operational stage of earthquake

response management. At a strategic level, it is dependent on the routes critical for effective emergency response. However, past studies have not addressed the identification of critical routes under budget constraints. This is a very important practical problem for emergency response planners as it involves identifying effective transportation routes while accounting for bridge structural conditions and budget limitations. It implies the identification of bridges to retrofit under earthquake response planning. This study develops a network-level methodology to assist decision-makers in identifying an effective seismic retrofit scheme for the bridges of a network.

This study formally defines the Multicommodity Maximal Covering Network Design Problem (MCNDP) and formulates it. To our knowledge, it represents the first attempt to incorporate the coverage objective in a generic network model, leading to a new class of models with significant practical implications. Past work has used the coverage criterion in identifying a path, tree or tour on a network by considering either the routing or the fixed cost, while the proposed model simultaneously considers the routing and fixed costs subject to a budget constraint. That is, none of the existing models consider routing costs, fixed costs and coverage criterion in a single framework.

From a practical standpoint, the key contribution of this study is its ability to address planning problems faced by emergency response agencies vis-à-vis disaster management. The specific problem addressed here relates to earthquake response management. Under budget constraints, there is a need to determine an effective retrofit plan for the bridges in a seismically-prone region. Most studies in the literature have adopted a local perspective to solving this problem, and do not consider the effect of potential bridge failures on the transportation system performance. The few methodologies that use a systems approach do not consider the influence of retrofit decisions on the quality of the associated routes vis-à-vis effective earthquake response. The proposed model fills this critical gap by considering the total travel time for the key O-D pairs in the network and the total population covered by them as the criteria for determining the critical routes. The

budget limitation on the total bridge retrofit costs for the links constituting the critical routes is represented as a constraint in our model. Hence, this methodology simultaneously determines the set of bridges in a network requiring retrofit and the routes that serve as focal points for earthquake response.

The evolution of the earthquake resistant bridge design requirements contained in the AASHTO Specifications and the existence of a proposed draft seismic design specification being discussed have significant implications in the assessment of the existing vulnerability, its mitigation, and the design of new bridges along the Emergency Routes. Along with this development, the USGS assessment of the seismic risk of southwestern Indiana and a change of format of the definition of the earthquake design ground motion that is integral part of the proposed draft seismic design specification also has important implications on the assessment of the operational capabilities of the Earthquake Emergency Routes of Indiana. The effects of the 2001 Nisqually Earthquake that occurred in the State of Washington gave invaluable insight on the expected situation for the State of Indiana due to a comparable bridge inventory in age and bridge types and a parallel increase of the seismic risk awareness in both regions over the last decades.

Using the information collected in this study, the GIS implementation through FEMA-HAZUS-MH software may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in southwestern Indiana. Vulnerability was evaluated for different levels of ground acceleration in order to obtain behavior patterns for selected bridges. Using these results, seismic behavior patterns were obtained for bridges located in southwestern Indiana included in the INdot Vincennes District maintenance data base. Soil amplification and liquefaction potential were evaluated for the cases having reported SPT measurements from borings. Based on this process, information critical for the identification of upgrade needs for the transportation structures part of the Earthquake Emergency Routes is made available to the INdot.

Implementation

The findings and developments of this research project are presented next in the form of suggested implementation steps by the Indiana Department of Transportation.

1. Formal adoption of the selected emergency routes or some variation of the same by the state is a necessary step. Such routes adopted and maintained by INdot will be affected by issues and policies outside the scope of those considered in this study. For instance, policy decisions, budgetary constraints, new projects, i.e. continuation of I-69, will likely affect the formal adoption of a set of emergency routes and will continue to impose changes in the future. The methodology developed in this study and the information implemented in HAZUS-MH should be of assistance to state decision makers in the future.
2. Future implementation strategies can be studied through simulation studies using appropriately updated information for different earthquake scenarios including earthquake ground motion defined in current AASHTO Standard Specifications, proposed Draft AASHTO LRFD Seismic Provisions, or other. These simulations will provide corresponding bridge retrofit scope for the proposed routes or alternative definitions.
3. The information in the maintenance database should be periodically evaluated and should include information currently available only in the bridge drawings (see Section 7.5 of the final report). This will improve the vulnerability assessment of the bridges refining the mitigation policies to adopt in the future. This expansion of the maintenance database can be extended to other counties and districts not covered in this research project.
4. Establish a program for assessing the liquefaction, soil spread, and embankment failure potential along the routes at locations other than bridge sites by providing a consistent evaluation methodology supported with the information developed in this study. This implementation will require a soil exploration program to obtain geotechnical properties of sites where no information exists or where such information was obtained years ago with different objectives than liquefaction evaluation. In the research conducted in this study, liquefaction evaluations were made using solely boring information contained in selected bridge drawings.
5. To establish procedures for emergency response under different earthquake occurrence scenarios.
6. INdot should consider the formal adoption of HAZUS-MH software for scenario management, mitigation and vulnerability studies and to train appropriate personnel. An important feature in this implementation step is the engagement of the Polis Center in the training of INdot personnel on the use of HAZUS-MH software. The Polis Center is a nationally recognized HAZUS implementation facility.

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<p>The occurrence of a strong earthquake in Indiana or neighboring states, as has occurred in the past, has serious implications for the State Transportation System with respect to adequate response. The definition of Earthquake Emergency Routes for the State of Indiana became a priority for the Indiana Department of Transportation. These Emergency Routes take into account various aspects related to transportation including coverage of population and area and travel time along these routes, and issues related to structural and geotechnical seismic vulnerability of the roads and bridges along these routes.</p> <p>Part I of the overall study report formulates a multi-commodity maximal covering network design problem (MCNDP) for identifying critical routes, for earthquake response and to seismically retrofit bridges. In the MCNDP, routes are sought that minimize the total travel time over the selected routes and maximize the total population covered, subject to a budget constraint on bridge retrofitting costs on the selected routes. The problem is formulated as a two-objective integer programming model and solved using the branch-and-cut module in the CPLEX optimizer. The model performance is analyzed using the transportation network in southwest Indiana. A problem reduction strategy is introduced to reduce computational times by recognizing that the critical routes are not usually circuitous. Thereby, the search for the critical routes for an origin-destination (O-D) pair is confined to a limited geographical region around it. To further reduce computational costs, the formulation is extended to incorporate valid inequalities that exploit the problem structure. Simulation studies are conducted to investigate the effects of varying the budget and the relative weights of the two objectives. Noninferior frontiers that illustrate the trade-offs between the conflicting objectives for different budgets are constructed to provide practical insights to decision-makers. The final selected routes in addition include constraints set by the Study Advisory Committee regarding specific roads to be included in the final set.</p> <p>Part II of the study report covers the structural and geotechnical seismic issues related to the definition of Earthquake Emergency Routes for the State of Indiana. Previous worldwide experience on bridge damage during strong earthquakes has influenced the bridge earthquake design specifications and mitigation procedures. The evolution of the relevant requirements in the AASHTO Specifications and the existence of a proposed draft seismic design specification under development as part of the NCHRP Project 12-49 have significant implications in the assessment of the existing vulnerability, its mitigation, and the design of new bridges along the Emergency Routes. Geographical Information System (GIS) was implemented to assist the evaluation of seismic vulnerability of the proposed Indiana Emergency Routes. Using the available information, the GIS implementation may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in Indiana incorporating information from the Indiana Department of Transportation (Indot) maintenance database and selected structural and geotechnical information from the relevant bridge drawings. Findings from a vulnerability assessment as well as final calibration performed using selected representative bridge data such as drawings and soil exploration descriptions are also reported. Vulnerability was evaluated for different levels of ground acceleration in order to obtain behavior patterns for selected bridges and to illustrate upgrade needs for the transportation structures on the Emergency Routes. The information contained in this report could be used to assist the engineers in conducting simulations that will help Indot to devise mitigation policies for different earthquake occurrence scenarios and to evaluate potential alternative routes.</p>			
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CHAPTER 1 INTRODUCTION

1.1 Background and Motivation

Earthquakes can significantly disrupt societal functioning, cause ecological damage, and lead to loss of human lives and property, warranting a coordinated and efficient response to mitigate their negative impacts. Past experience with earthquakes has demonstrated the vulnerability of the critical infrastructural lifelines and the need for mitigation strategies as well as emergency response planning. For example, the bridge failures under the 1989 Loma Prieta and the 1994 Northridge earthquakes resulted in substantial losses to the regional economies [Chang and Nojima, 2001].

Highway transportation systems are vital to the normal functioning of a society and can be even more so under disasters. Under disasters, emergency response agencies need to transport various commodities, including food, clothing, medicine, medical supplies, machinery and personnel from different points of origin to different locations in the disaster areas in an efficient manner [Haghani and Sei-Chang, 1996]. Damage to the highway system can seriously affect emergency response, recovery operations and disrupt the regional economy; hence the functionality of these systems is critical for post-disaster response. Gordon et al. [1998] estimate that twenty percent of the \$6.5 billion in losses due to the Northridge earthquake resulted from damage to the transportation system. This highlights the need for countermeasures to mitigate the possible risk of damage to the highways and the associated consequences.

Bridges typically represent the elements of a highway system that are most susceptible to failure under earthquakes. Hence, the seismic vulnerability of bridges has been extensively studied [Buckle et al. 1994]. As a consequence, seismic retrofitting of bridges has been the most preferred solution adopted in the pre-disaster strategic planning stage so as to minimize the risk of unacceptable damage to highways in earthquake-prone regions. Due to the significant effort and cost involved in strengthening existing bridges to seismic design standards it is not practical to retrofit every bridge in an earthquake-

prone region, and even if it were possible, retrofitting cannot be done simultaneously across all bridges. Also, seismic risk can vary across bridges due to their physical characteristics, age, and current structural condition. Hence, a key problem faced by earthquake response planners is the identification of the set of bridges for seismic retrofitting. This implies prioritization given budget limitations and the effort involved.

The bridge prioritization problem, that is, the selection of bridges for seismic retrofitting, can be viewed from structural and transportation domains vis-à-vis performance. From a structural perspective, the risk/vulnerability of a bridge to seismic failure is the predominant factor that influences prioritization. In the transportation context, the focus is on ensuring the survival of links (and the bridges on those links) that minimize emergency response times and maximize the population that can be reached, by retaining a network-level perspective. The existing literature predominantly addresses the prioritization problem from a structural perspective. The dominant approach is the use of multi-attribute utility theory [Nojima, 1999] to determine the risk for each bridge in the highway network. This is done by weighing the values of the various attributes to generate a measure of risk. For example, one study [Osaka Municipal Government Report, 1998] uses attributes on seismic capacity (or vulnerability) and importance (or criticality). Seismic capacity, an indicator of the structural sufficiency of a bridge with regard to an anticipated earthquake, is based on the bridge type, structural condition, service life, and geotechnical and seismic hazards. The importance criterion indicates the criticality of the bridge to post-disaster relief operations based on the traffic flow on the associated link, economic impacts due to its failure, and the connectivity with crossroads.

While the seismic capacity criterion has been well-studied in the literature, the importance criterion has not been adequately addressed [Chang and Nojima, 2001]. This is because bridge importance is not readily quantifiable, implying subjectivity in its determination. The issue of accessibility across the transportation network vis-à-vis earthquake response can be influenced by political, social and economic factors in addition to the engineering solutions. Hence, only a few retrofitting schemes exist in practice, and are primarily based on engineering judgment [Buckle, 1994], indicating methodological gaps in the state of knowledge to address this problem. The intuitive

solution to the earthquake response planning problem is the retrofitting of every bridge in the shortest path between the selected origin-destination (O-D) pairs in the network. However, this is typically cost prohibitive. This implies the need for a network level perspective to address the problem which considers the interactions between the failure of a bridge and its influence on the network-level system performance [Werner et al., 1997, Chang and Nojima, 2001]. The Multidisciplinary Center for Earthquake Engineering Research (MCEER) Bulletin (1994) suggests that the system performance considerations be addressed before performing the seismic capacity evaluation of bridges.

Few studies consider network-level transportation system performance in determining prioritization schemes. Basoz and Kiremidjian [1994] develop a network-level procedure to quantify the importance of bridges. It determines the bridges that constitute minimal cuts in a transportation network and ranks the individual bridges within the set. The minimal cut for a given O-D node pair in a network is the minimum number of links whose failure disconnects the two nodes. Performance was tied to the connectivity between critical destinations under disasters. Wakabayashi [1997] performs an importance analysis of the Kobe highway network based on several network realizations. The travel time between Osaka and Kobe represented the performance criterion. The structural vulnerability of the links was not considered.

Nojima [1999] proposed a performance based prioritization method for upgrading network components with limited resources. The performance measure was the reliability of system flow capacity, defined as the maximum flow between a specified O-D pair in the network. The study rank-orders links based on their ability to improve the reliability measure. Chang and Nojima [2001] develop performance measures based on the total length of the surviving network and the accessibility provided by it.

The overview of the past work indicates that several studies exist that address the bridge seismic retrofit aspects of the earthquake response planning problem. Hence, the problem has been addressed primarily in the structural engineering domain. Different criteria have been used to evaluate the bridges in the network. Some of them are ad-hoc in nature and involve considerable subjectivity. Only a few studies exist that address the seismic retrofit problem from a transportation systems perspective. Among those, there

are very few studies that analyze the problem at a network-level. Those that do are primarily concerned with the effect of retrofit decisions on vehicular flow performance measures for the surviving network. Studies addressed from the perspective of earthquake response are sparse, and typically consider a single criterion such as connectivity between O-D node pairs, travel time for a single O-D pair, or accessibility to population. None of these studies explicitly consider budget constraints in the planning problem. This significantly increases the problem complexity due to the dependence of the surviving network under an earthquake on the budget investment decisions. This key gap, significant from a practical standpoint, motivates the development of a new class of models that can aid planners in developing an effective budget allocation scheme to prioritize retrofit decisions. The proposed research addresses this by developing the **MCNDP** model.

1.2 Study Objectives

The primary study objective is to develop a methodology to address the strategic planning problem at a network-level for earthquake response. This methodology enables the decision-makers to identify an effective seismic retrofit scheme vis-à-vis earthquake response for the bridges of a network. The proposed methodology is motivated by the need to consider the key factors for effective earthquake response in a single framework. This is done by first defining the concept of “critical routes”. The critical routes of a transportation network are the set of routes whose functionality is critical to the effectiveness of earthquake response. The associated problem is labeled the critical routes problem. The specific tasks to address the study objective are:

1. Identification of criteria for selection of critical routes. These criteria are based on factors that directly affect the effectiveness of earthquake response.
2. Formally define the critical routes problem. This involves the description of the problem context, its constraints and the criteria used to solve the problem.
3. Development of a model to address the critical routes problem. A mathematical formulation is developed that seeks to optimize the response criteria under budget

limitations and other constraints. This generates the critical routes for the study region of interest.

4. Application of the model to a case study. Sensitivity analyses are conducted with respect to the model parameters to derive insights that assist decision-makers. This is done primarily by identifying a non-inferior frontier, which provides an intuitive practical tool for decision-making. The region of interest in this study is the earthquake-prone southwest region of Indiana. Figure 1.1 depicts the seismic map of the region.

1.3 Organization of the Report

This report is organized as follows. Chapter 2 provides an overview of the relevant literature in network design and coverage. Chapter 3 defines the critical routes problem and formulates it. It further proves that the problem is NP-hard, and identifies valid inequalities to enhance solution efficiency. Chapter 4 discusses the application of the formulation to a case study in the southwest Indiana region, and provides the implementation details. Chapter 5 reports the results from experiments conducted and derives insights. Chapter 6 reports the results from experiments conducted after incorporating the recommendations of the Study Advisory Committee in terms of “required” routes as part of the critical routes subnetwork. Chapter 7 provides some concluding remarks.

Figure 1.1 Seismic map of southwest Indiana

CHAPTER 2 LITERATURE REVIEW

This chapter provides a brief review of the methodological approaches relevant to the problem addressed in this research. As the focus is on network-level transportation-related planning for disaster response, especially under earthquakes, most of the relevant literature is in the urban operations research domain as applied to network models. For the sake of exposition, we first define the two types of link costs that are typically encountered in network design problems, namely fixed costs and routing costs. The fixed cost of a link is a one-time cost incurred with choosing that link for routing commodity flows and is independent of the quantity routed through it. The routing cost of a link is defined as the cost incurred in transporting a unit commodity across that link. For example, in an earthquake response planning context, bridge seismic retrofit costs would represent fixed costs for that link, and the time to traverse that link would correspond to the routing costs.

2.1 Network Design

A broad range of practical network optimization problems occur in the context of transportation, distribution planning, emergency response, telecommunications and computer networking. A specific class of these, labeled as network design problems, arises primarily in the capital investment phase of engineering problems. The network design problem generally seeks to determine a network configuration that minimizes the sum of the fixed costs of the links chosen and the cost of routing different commodities through the network defined by these links. Due to its usefulness in the aforementioned applications, the fixed charge network design problem has been well studied. Magnanti and Wong [1984] provide a comprehensive survey of integer programming-based methods to address network design problems. They develop a generic discrete choice network design formulation that unifies the different types of network design models. Well-known problems in combinatorial optimization, including the shortest path,

minimum spanning tree, Steiner tree and traveling salesman problem have been shown to be special cases of the generic network design model. However, these models have focused primarily on cost minimization and do not consider other important criteria like population coverage, environmental impacts and revenues.

2.2 Network Coverage

Current and Min [1986] emphasize the potential for multiple objectives in transportation planning problems and highlight the conflicting nature of such objectives in the context of public sector decision-making. For example, the shortest path network across O-D pairs may not maximize the accessibility to the population in the region. To overcome these limitations, Current et al. [1985] introduce the notion of coverage to the network design problem by formulating the maximum covering shortest path problem for a single O-D pair with two objectives: (i) to minimize the path cost for that O-D pair, and (ii) to maximize the total population covered by that path. They associate some fraction of the population with each node, which is labeled as the demand of that node. They define a demand node as covered if the path includes the node or passes through another node that is within a pre-specified distance from that node. They also formulate a variation of this problem for hazmat routing applications, called the minimum covering shortest path problem [Current et al., 1988], in which the demand covered is minimized.

Hutson and Revelle [1989] extend the concept of coverage to tree networks by considering two types of coverages: direct and indirect. A demand node is directly covered if there is a link in the tree incident upon it. Indirect coverage is assumed if the demand node is within a prescribed distance from a link in the tree. The maximal direct covering tree problem seeks to minimize the cost of the subtree and maximize the total demand covered by it. Hutson and Revelle [1993] extend the approach to develop the maximal indirect covering tree problem. They suggest the use of these models for problems involving road network construction in sparsely populated areas under resource constraints.

Kim et al. [1989] introduce the subtree r -cover problem which seeks a connected subgraph covering every demand node with minimum total length for a given network. A

cover radius is associated with each demand node. It is used to ensure that the distance between the demand node and the closest point in the subgraph does not exceed a threshold value. The formulation is applied to position troops in a war situation to ensure that troops protect/cover the various demand nodes within pre-specified threshold response times. Kim et al. [1990] consider a special case in which the given network is a tree.

Current and Schilling [1989] adapt the coverage concept to the traveling salesman problem by formulating the covering salesman problem. The problem aims to identify a minimum cost subtour in a network such that every demand node is within a specified distance from a node on the subtour. They extend this model to develop the maximal covering tour problem [1994], in which the objectives are to minimize the total tour length and maximize the demand covered by the tour, subject to a requirement on the number of nodes in the subtour. Suggested application domains include rural health care and overnight delivery systems. Gendreau et al. [1997] solve a variant of the maximal covering tour problem in which a given set of nodes must always be present in a tour, and propose its application to the location of post boxes and the design of collection routes. They also propose classes of valid inequalities for use in a branch-and-cut algorithm to solve large problem instances in a reasonable amount of time.

2.3 Discussion

The overview of the literature indicates that the coverage criterion has been successfully applied to identify a path, tree or tour, all of which are special network structures. This research extends the coverage criterion to a network problem with multiple O-D pairs, labeled the multicommodity maximal covering network design problem (**MCNDP**). Its objectives are to minimize the routing costs over all O-D pairs and maximize the total demand covered, subject to a budget constraint based on the fixed costs incurred on the chosen links. To our knowledge, there exist no prior studies that consider budgetary limitations, routing costs and coverage criteria in a single model with multiple O-D pairs. Such a model is essential for strategic planners to make long-term budget allocation decisions that factor in system-wide impacts of budgetary decisions

over time vis-à-vis performance. Potential applications of the **MCNDP** other than the problem addressed in this study include the design of regional transit systems and the planning of truck shipment routes. With modifications, the **MCNDP** can be used to design electric power networks to build high voltage lines for new power plants or to connect to existing power plants.

CHAPTER 3 METHODOLOGY

This chapter develops the methodology to address the critical routes problem using the **MCNDP**. Section 3.1 illustrates the applicability of the **MCNDP** to the “critical routes problem”. Section 3.2 states the **MCNDP**. Section 3.3 presents the mathematical model. Notation and parameters are introduced in Section 3.3.1, the decision variables are defined in Section 3.3.2, and the formulation is illustrated in Section 3.3.3. Section 3.4 examines the computational complexity of the **MCNDP**. Section 3.5 presents valid inequalities for the formulation.

3.1 The Critical Routes Problem

Network-level disaster management planning is vital for effectively responding to natural calamities and security-related problems. For example, the availability of the transportation network is critical to emergency response [Haghani and Oh, 1996] under earthquakes. It entails the identification of “critical routes” in a planning context that remain functional following an earthquake, to enable the response operators to access as much population as possible in a minimum amount of time. This implies two objectives for the selection of the critical routes: (i) minimizing the total travel time on the routes between the O-D pairs, and (ii) maximizing the total population that can be covered by these routes. The functionality or survivability of a route, implying its availability under an earthquake, is governed significantly by its weakest elements, the bridges. The seismic retrofitting [Cooper et al., 2001] of bridges can enhance the survivability of the associated routes under earthquakes. However, as discussed earlier, due to the significant cost and effort involved in retrofitting, it is impractical to retrofit every bridge in an earthquake-prone region, especially with limited budgets. Hence, the budget serves as a constraint in the determination of the critical routes. Therefore, the critical routes problem

can be viewed as a budget-constrained covering network design problem which seeks low cost routes that cover the maximum population while satisfying budget constraints introduced by the need to retrofit bridges on the critical routes.

3.2 Problem Statement

We formulate the **MCNDP** generically here, and apply it to the critical routes problem in the next chapter. We are given the locations of demand centers of a region and its associated undirected network. The network links have a fixed cost for their usage and a routing cost. There is a budget constraint on the total fixed cost incurred. The **MCNDP** seeks to allocate a limited budget to links such that the total routing costs for a set of O-D pairs is minimized and the total demand covered by the routes connecting them is maximized. The demand of a center is covered if a link in one of the selected routes provides access to it.

3.3 Mathematical Model

This section first introduces the notation, parameters and decision variables, and then proposes an integer programming formulation for the **MCNDP**.

3.3.1 Notation and Parameters

Let $G(N, E)$ denote an undirected network with node set N and link set E . The indices i and j denote a node in the network, $i, j \in N$ and $E \subseteq N \times N$, where $[i, j]$ denotes an undirected link between nodes i and j with a nonnegative fixed cost f_{ij} . Let B to denote the available budget. Each O-D pair in the network is represented as a unique commodity type. Let k represent the commodity type index, $k \in K$, where K denotes the set of all commodities. One unit of flow of commodity k must be transported over the network from its origin $O(k)$ to its destination $D(k)$. To differentiate the direction of flow of a commodity, we consider two directed links (i, j) and (j, i) corresponding to each original undirected link $[i, j]$. Let A denote the set of the directed links; all links are uncapacitated. Let c_{ij}^k be the nonnegative routing cost for a unit of commodity k on link (i, j) , and m

demand the demand center index, $m \in M$, the set of demand centers in the region. Let r_m represent the demand associated with demand center m , and E_m the set of links that are eligible to cover it. We treat the demand centers separately from the nodes in the network, whereas past studies assign demand centers to a node in the network which is a more restrictive approach.

3.3.2 Decision Variables

The formulation contains three types of variables: (i) the arc flow variables denoted by the vector $x = \{ x_{ij}^k \}$, which define the flow of different commodities in each of the selected links, (ii) the design variables denoted by the vector $y = \{ y_{ij} \}$, which define the links selected for the network design, and (iii) the coverage variables denoted by the vector $z = \{ z_m \}$, which define whether or not a demand center is covered. They are defined as follows:

$$x_{ij}^k = \begin{cases} 1, & \text{if there is a unit flow of commodity } k \text{ on link } (i, j) \\ 0, & \text{otherwise} \end{cases}$$

$$y_{ij} = \begin{cases} 1, & \text{if link } [i, j] \text{ is used in a flow path} \\ 0, & \text{otherwise} \end{cases}$$

$$z_m = \begin{cases} 1, & \text{if demand center } m \text{ is accessible from a link of a flow path} \\ 0, & \text{otherwise} \end{cases}$$

3.3.3 The MCNDP Formulation

The **MCNDP** formulation has two objectives: Z_1 , the total routing cost, and Z_2 , the total demand covered where,

$$Z_1 = \sum_{k \in K} \sum_{[i,j] \in E} (c_{ij}^k x_{ij}^k + c_{ji}^k x_{ji}^k) \quad \text{and} \quad Z_2 = - \sum_{m \in M} r_m z_m.$$

The integer programming formulation for the **MCNDP** is expressed as follows:

MCNDP:

$$\text{Minimize } Z = [Z_1, Z_2] \tag{1}$$

subject to

$$\sum_{(i,j) \in A} x_{ij}^k - \sum_{(j,i) \in A} x_{ji}^k = \begin{cases} 1, & \text{if } i = O(k) \\ -1, & \text{if } i = D(k) \\ 0, & \text{otherwise} \end{cases} \quad \forall i, k \quad (2)$$

$$z_m \leq \sum_{k \in K} \sum_{[i,j] \in E_m} (x_{ij}^k + x_{ji}^k) \quad \forall m \quad (3)$$

$$x_{ij}^k \leq y_{ij} \quad \forall k, [i, j] \in E \quad (4)$$

$$x_{ji}^k \leq y_{ij} \quad \forall k, [i, j] \in E \quad (5)$$

$$\sum_{[i,j] \in E} f_{ij} y_{ij} \leq B \quad (6)$$

$$\sum_{(i,j) \in A} x_{ij}^k \leq 1 \quad \forall i, k \quad (7)$$

$$\sum_{(i,j) \in A} x_{ij}^k \leq 1 \quad \forall k, j = D(k) \quad (8)$$

$$\sum_{\{(i,j) \in A / \forall i, j \in Q\}} x_{ij}^k \leq |Q| - 1 \quad \forall k, Q \subseteq N, 2 \leq |Q| \leq |N| - 2 \quad (9)$$

$$x_{ij}^k, x_{ji}^k = 0 \text{ or } 1 \quad \forall k, [i, j] \in E \quad (10)$$

$$y_{ij} = 0 \text{ or } 1 \quad \forall [i, j] \in E \quad (11)$$

$$z_m = 0 \text{ or } 1 \quad \forall m \in M \quad (12)$$

$$x \in S \quad (13)$$

In multiobjective programming, there may never exist a single solution that optimizes all the objectives. Thus, the notion of an optimal solution is not relevant here and is replaced by the concept of a noninferior solution set from which the decision-maker

selects the most preferred solution. A solution to the above formulation is said to be noninferior if there exists no other feasible solution that improves one objective without degrading the other. There are two popular solution approaches to multiobjective programming: preference based techniques and generating techniques [Cohon, 1978]. The latter approach, which has been commonly employed due to its simplicity, generates the entire noninferior solution set or an approximation of it. We adopt this approach. An approximation of the set can be obtained by using a single objective function formed by different convex combinations of the objective functions. Our objective function becomes:

$$Z(w_1, w_2) = w_1 Z_1 + w_2 Z_2 \quad (14)$$

where $w_1 + w_2 = 1$, and $w_1, w_2 \geq 0$.

The use of the weight pair (w_1, w_2) reflects the explicit tradeoffs between the total routing cost and the demand that can be covered. Due to the discrete nature of the MCNDP solution set, the above weighting method is incapable of identifying noninferior solutions that lie in the duality gap of the convex hull of the set. Figure 3.1 illustrates an example of a noninferior frontier which is a discrete set of points. The boundary of its convex hull is obtained by joining its exterior points. However, there can be non-dominated solutions that lie within the convex hull. For example, point B represents such an instance in the figure. Hence, point B is said to lie in the duality gap of the convex hull. Solving a constrained version to enable such identification entails a substantial increase in the computational effort. Also, the number of noninferior solutions is large, though finite, for even small-size discrete optimization problems and in the worst case increases exponentially with problem size. Hence, it is not practical to generate the entire noninferior set. Instead, it is appropriate to focus on generating an approximation to the noninferior solution set.

Constraints 2 denote the network flow conservation constraints that require x to describe a simple path from the origin to the destination for all commodities. Constraints 3 represent the coverage constraints, which imply that a demand center is covered only when at least one of the links providing accessibility to the center is in a flow path.

Constraint sets 4 and 5 are the forcing constraints; they state that no flow is allowed in either direction of link $[i, j]$ unless the associated fixed cost is incurred. The **MCNDP** can be formulated in an alternative way by expressing the forcing constraints in an aggregated form:

$$\sum_{k \in K} x_{ij}^k \leq |K| y_{ij} \quad \forall [i, j] \in E \quad (15)$$

$$\sum_{k \in K} x_{ji}^k \leq |K| y_{ji} \quad \forall [i, j] \in E \quad (16)$$

Using constraint sets 15 and 16 instead of 4 and 5, respectively, significantly reduces the number of constraints in the formulation, and consequently the problem size. However, past efforts suggest that the generic network design models with the less efficient disaggregate formulation perform computationally much better than the more efficient aggregate formulation [Magnanti and Wong, 1984]. This is because the disaggregate version of the forcing constraints better approximates the convex hull of the set of feasible integer solutions of the **MCNDP**, and hence, yields tighter lowerbounds for the linear programming (LP) relaxation of the formulation.

Constraint 6 is the budget constraint; it states that the sum of the fixed costs of the links in any solution should not exceed B .

Due to constraint set 3, any solution can entail looping paths of the kind C1 and C2, as shown in Figure 3.2, for one or more commodities as loops help to achieve extra population coverage. We seek loopless paths and this is enabled through constraint sets 7 and 8. Constraints 7 state that the maximum flow of any commodity type exiting any node in the network should not exceed unity. Constraints 8 prevent the occurrence of loops at destination nodes (similar to C2), which is not precluded by 7.

The coverage constraints can also lead to isolated subtours such as ST (as shown in Figure 3.2) that do not share any link with the corresponding commodity flow paths in the solution. These subtours are prevented by adding subtour elimination constraints 9, in which Q denotes the nodes in a subtour. Since, $|Q|$ can potentially take several values between 2 and $|N|-2$, there are potentially an exponential number of such constraints.

Constraint sets 10, 11 and 12 restrict the flow, design, and coverage variables, respectively, to 0-1 values.

Constraint 13 restricts the flow variables to the set S which represents a restricted solution domain S in which some flow variables are fixed *a priori*. As discussed in the next chapter, this is necessary to restrict the geographical region within which the search is performed to enable computational time savings. Presumably, the commodity paths to be determined would be confined to a subnetwork surrounding the O-D pair, and not circuitous. Hence, it models the topological restrictions upon the commodity flows in the network.

The **MCNDP** is a linear integer program with $(2|K||E|+|E|+|M|)$ integer variables and comprises of $(|K||N|+|M|+2|K||E|+1+|N||K|+|K|)$ constraints and an exponential number ($O(2^{|E|})$) of subtour elimination constraints.

3.4 Computational Complexity of MCNDP

Lemma: The **MCNDP** is NP-hard.

Proof: We prove it by restriction. Consider the instance **MCNDP_R** of problem **MCNDP**: $|M| = 0$, that is, there are no demand centers and the objective function is just the total routing cost of the various commodities. The coverage constraints 3 cannot be imposed as there are no demand centers. Also, the constraints sets 7, 8 and 9 are redundant in the new formulation and can be ignored. Solving this instance, which has only one objective, is equivalent to solving a budget design problem which is known to be NP-hard [Johnson et al., 1978]. Therefore, the **MCNDP** generalizes the budget design problem. It follows that the **MCNDP** is NP-hard.

Hence, the **MCNDP** is rather intractable, implying that the solution approach is typically an enumeration-type procedure.

3.5 Valid Inequalities

Typically, integer programming formulations are solved using branch-and-bound type methods that solve linear programs at the nodes of the branch-and-bound tree. The computational time for such procedures is dependent on the number of tree nodes

enumerated to obtain the solution, as well as the time required to solve the formulation at each enumerated node. The number of nodes to be enumerated is dependent on the quality of the bounds generated by associated LP relaxations. Valid inequalities, which are constraints based on the problem characteristics, are useful in this context. They are redundant in an integer programming formulation but can eliminate non-integer solutions that are optimal for the LP relaxations. Hence, they improve the lower bounds computed by the solution algorithms, thereby generating computational time savings. Valid inequalities can be appended to the **MCNDP** to enhance the formulation.

Proposition 1. The constraint

$$x_{ij}^k + x_{ji}^k \leq y_{ij} \quad \forall k, [i, j] \in E \quad (17)$$

is a valid inequality for the **MCNDP**.

Proof: In any feasible design, a link $[i, j]$ is either chosen or not. Consider the case where link $[i, j]$ is selected, which implies $y_{ij} = 1$. Then, the corresponding set of inequalities for that link in constraints 17 are its subtour elimination constraints for $|Q| = 2$. If that link is not selected, it implies $y_{ij} = 0$. Then, the forcing constraints imply that $x_{ij}^k = 0, \forall k$, and the corresponding constraints in 17 are valid. This completes the proof.

This valid inequality is a virtual constraint vis-à-vis the problem as it is not a direct representation of a physical reality or a logical characteristic. However, it can replace constraints 4 and 5, and part of constraints 9 (that is, for $|Q| = 2$). Hence, it can be used to generate some computational time savings by reducing the problem size.

Proposition 2. The following constraints are valid for the **MCNDP**.

$$y_{ij} \leq z_m \quad \forall m, [i, j] \in E_m \quad (18)$$

$$x_{ij}^k \leq z_m \quad \forall m, k, [i, j] \in E_m \quad (19)$$

$$x_{ij}^k + x_{ji}^k \leq z_m \quad \forall m, k, [i, j] \in E_m \quad (20)$$

Proof: If $x_{ij}^k = 1$ for some k and $[i, j] \in E_m$ then y_{ij} assumes the value 1, and the demand center m is covered implying $z_m = 1$. If $x_{ij}^k = 0 \forall k, [i, j] \in E_m$ then no fixed cost is incurred for the $\forall [i, j] \in E_m$ and the demand center m is not covered. This completes the proof of constraints 18. The proof of constraints 19 follows directly from the forcing constraints, and the result just proved. Constraint set 18 is redundant in the **MCNDP** formulation and directly follows from constraints 17 and 18.

These valid inequalities imply that if a demand center m is not chosen, then links in E_m do not appear in the solution. This simplifies the budget constraint and network flow conservation constraints, leading to potential computational efficiencies.

Proposition 3. Let $m1$ and $m2$ be two demand centers such that $E_{m1} \subseteq E_{m2}$, then the following relation holds.

$$z_{m1} \leq z_{m2} \quad (21)$$

Proof: Suppose demand center $m1$ is covered by flow of some commodity k in arc (i, j) , this flow also covers demand center $m2$. If demand center $m1$ is not covered, constraint 21 reduces to the nonnegativity constraint for the variable z_{m2} . The proof is complete.

This valid inequality can potentially reduce computations by exploiting the problem characteristic that if a demand center is covered by a link (i, j) , it is redundant to search other links to cover this demand center.

In summary, the **MCNDP** is a NP-hard integer programming formulation. It is exacerbated by the need for subtour and looping elimination constraints. In addition, valid inequalities can be proposed to increase computational efficiency by exploiting the problem structure. The next chapter discusses the application of the **MCNDP** to the determination of critical routes for earthquake response planning.

Figure 3.1 Example of noninferior solution in the duality gap

Figure 3.2 Example of a solution with looping paths and a subtour

CHAPTER 4 CASE STUDY: CRITICAL ROUTES FOR EARTHQUAKE RESPONSE PLANNING

This chapter discusses the application of the **MCNDP** formulation to the determination of critical routes for earthquake response in southwest Indiana. It first describes the details of the case study, followed by a summary of the solution procedure and its implementation. A problem reduction strategy is proposed to reduce computational times.

4.1 Case Study

The **MCNDP** is used to determine the critical routes under earthquakes for a network representing southwest Indiana, as shown in Figure 4.1. It consists of 184 nodes, 307 links, and 93 population centers. The demand of a center is set equal to its population as obtained from the US Census 2000 Data [US Census Bureau, 2000]. Fifteen centers have a population greater than 5000 and are denoted as major population centers. The remaining 78 have a population between 3000-5000, and are labeled minor population centers. O-D node pairs, which represent the commodities (that is each O-D pair represents one commodity), are chosen so as to ensure connectivity between the major population centers. From Figure 4.1, the 15 major population centers can be viewed to lie in five different layers from top to bottom: 3 centers each in the first three levels, 2 in the fourth level, and 4 in the fifth level. To enable connectivity across the region it is reasonable to choose O-D node pairs that connect major population centers between adjacent layers, as also those within each layer. Note that the network topology constituted by any feasible set of critical routes is connected. This approach generated 33 O-D node pairs. Using engineering judgment, the paths of 5 O-D pairs are pre-determined. The fixed cost of a link is the total retrofit area of all the bridges on that link. The link routing costs are their free-flow travel times, implying that link capacity is

ignored. This is because the problem addresses the planning stage of earthquake emergency response. The link travel times are assumed to be symmetric. The links that can potentially cover a population center are those that are within some pre-specified threshold distance of it. They were identified by observing its location and the proximity of roads (interstates, US roads and state roads) to it using the geographical map of the region.

4.2 Solving the **MCNDP**

4.2.1 Solution Procedure

The **MCNDP** is solved using the branch-and-cut algorithm in CPLEX [ILOG CPLEX 7.1, 2001]. It is a search technique that uses the tree structure in which CPLEX dynamically adds cut constraints at the tree nodes to reduce the tree size. The solution procedure has two primary computational aspects. The first is the number of the tree nodes searched in the branch-and-cut method since each requires the solution of a linear program (that is, the LP relaxation of the **MCNDP** formulation). The LP relaxation represents the formulation obtained by allowing the integer decision variables to assume continuous values in the range within their upper and lower limits. The optimal value of this formulation provides a lower bound for the **MCNDP** formulation as it is less restrictive than the original formulation. Thereby, larger the number of tree nodes searched, the greater the computational time. This highlights the critical importance of the second computational aspect, the **MCNDP** formulation size, which depends on the number of constraints. In this context, computational savings can be generated by using valid inequalities, and/or by including or excluding the subtour elimination constraints. The solution procedure first excludes most of the subtour elimination constraints to make the problem computationally tractable. The only ones included are those for $|Q|=2$ implying link subtours. This is because there are only a few constraints with $|Q|=2$, and preliminary analysis indicates that the associated subtours can occur frequently in the solution if they are not precluded. Next, the solution procedure identifies subtours in the solution obtained from CPLEX using this relaxed formulation. If subtours exist, it adds

the necessary subtour elimination constraint(s) and reoptimizes the new formulation. This step is repeated until no more subtours are encountered in the solution.

The best-known integer solution, which is the upper bound to the optimal solution, is the final solution obtained from CPLEX. The lower bound to the optimal solution is the best objective function value across all LP relaxation solutions of the branch-and-cut nodes. They are used to determine the percentage optimality gap, which is a measure of the quality of the solution. The percentage optimality gap is defined as $[(\text{upper bound} - \text{lower bound}) * 100] / \text{lower bound}$, where the bounds are obtained from CPLEX. It indicates how close the best-known integer solution is to the optimal solution.

4.2.2 Implementation Details

The computing environment consists of a Sun Ultra Enterprise server E6500 with 26 400Mhz UltraSparc II processors under the multi-user Solaris 7 operating environment with 23GB RAM, 131GB swap space and 8MB cache. A C++ program was implemented to solve the problem and was compiled using GNU, g++ v2.95.3. It invokes subroutines from the CPLEX Callable Library version 7.1 to construct the formulation and solve it with the in-built mixed integer programming (MIP) optimizer that provides a customized branch-and-cut procedure. Experiments are performed by excluding and including the valid inequalities in the formulation.

A preliminary analysis was performed to calibrate the CPLEX parameter settings for different experimental scenarios in terms of the budgets and the relative weights of the two objectives, and for different % optimality gaps. A trial and error approach was adopted in order to determine a good choice for the optimality tolerance level to solve an instance to the maximal permissible closeness to optimality as possible within a reasonable computational time. Our test runs indicated that for very low values w_2 (≤ 0.005), default settings yielded good computational performance. However, some analysis was required to arrive at a good setting for higher values of w_2 . It was based on aggressive probing, best estimate search for the node selection strategy, and variable branching using pseudo reduced costs. The CPLEX preprocessor was set to on, as it was able to achieve reduction in the MIP problem size leading to faster solution times. Also,

based on our preliminary analysis a priority order was assigned to the variables in which branching would be performed first on the x variables, then on the y variables and finally on the z variables. Each instance was solved under exactly identical parameter settings for the set of experiments with or without valid inequalities to compare the performance of the two cases.

4.2.3 Problem Reduction Strategy

The preliminary runs indicated the intractable nature of the **MCNDP**; an instance with budget $B = 3.33$ million sqft and weight pair $(w_1, w_2) = (0.99, 0.01)$ required over 5 days to solve to a 1% optimality gap. To generate significant computational time savings, a strategy to reduce the search domain was incorporated based on the notion that the solution for an O-D pair would lie within some restricted geographical area around it. Therefore, it is reasonable to expect that the path for an O-D pair is confined to a subnetwork around it rather than including circuitous components that are geographically further away. This enables us to set the commodity flow variables to zero for the rest of the network, which provides the restricted solution domain S for the formulation.

Figure 4.1 Road network of southwest Indiana

CHAPTER 5 RESULTS FROM COMPUTATIONAL EXPERIMENTS

This chapter reports the results and derives insights from the computational experiments conducted using the case study. The primary objective of the experiments is to analyze model sensitivity for different budgets and relative weights for the two objectives. We make a few comments on the computational performance. Subsequently we seek to develop noninferior frontiers that illustrate the trade-offs between the conflicting objectives for different budgets to provide practical insights to decision-makers.

5.1 Model Sensitivity and Computational Performance

An effective problem reduction strategy was sought to be identified based on computational efficiency of the test runs conducted. In the runs all the valid inequalities were included for computational efficiency. A strategy which sets 83.46% of the total number of flow variables to zero *a priori*, performed in a robust manner. Hence, further experiments were conducted using this strategy. The formulation was solved for the following weight pairs: (0.999,0.001), (0.99,0.01), (0.95,0.05) and (0.9,0.1) for the budget range 1.43 million sqft to 3.33 million sqft. The value 1.43 million sqft is the minimum budget required for a feasible solution to the formulation. Table 5.1 illustrates the total travel time and the total population covered for the instances solved. The computational times were higher for larger w_2 values because of the need to search more routes due to the increased importance of population coverage in the objective function. When w_2 is smaller, the problem approaches the budget design problem where the network structure can be favorably exploited to yield better computational performance. For low budgets, the problems are more difficult to solve due to interactions between commodities caused by the need to share links in the final solution. This was also observed by Dionne and Florian [1979]. In our experiments, subtours occur in the

intermediate solutions for all budgets with weight pairs (0.95,0.05) and (0.9,0.1), and only for two instances in (0.99,0.01). This is because a larger weight for population coverage leads to physically meaningless isolated subtours in an effort to improve the objective function value.

5.2 Noninferior Frontiers: Travel Time-Population Coverage Trade-offs

Figure 5.1 displays the noninferior solution set for the budget range 1.43 million sqft to 3.33 million sqft. In the figure, the solutions towards the left of the graph for any budget correspond to lower w_2 values; for example, the leftmost points correspond to $w_2 = 0.001$.

In general, for a budget, higher values of w_2 lead to greater population coverage at the expense of increased system travel time. This plot provides flexibility [Hall, 1985] to the decision-maker by generating a range of solutions. Noninferior solution sets, like the ones shown here, can be very useful to the decision-maker as they highlight the tradeoffs between conflicting objectives. They also aid decision-makers to compare the additional benefits accrued in terms of system travel time savings and extra population coverage due to additional budget. For example, the marginal benefits that could be realized from a budget of 2 million sqft instead of 1.67 million sqft would be higher than those under 2.33 million sqft instead of 2 million sqft.

Figures 5.2a-g depict the critical routes for seven of the instances solved above. For the lowest budget, 1.43 million sqft., the subnetwork formed by the critical routes is sparse, but still covers approximately 90% of the total population of the region. For the critical route networks shown in these figures, the travel times and the population covered are higher as the weight for population is increased, keeping the budget fixed. This is because the routes are more circuitous.

Figure 5.1 Noninferior solution set for different budget values

Figure 5.2b Critical routes for the weight pair (0.99,0.01) and budget 1.67 million sqft

Figure 5.2c Critical routes for the weight pair (0.9,0.1) and budget 1.67 million sqft

Figure 5.2d Critical routes for the weight pair (0.99,0.01) and budget 2 million sqft

Figure 5.2e Critical routes for the weight pair (0.95,0.05) and budget 2 million sqft

Figure 5.2f Critical routes for the weight pair (0.99,0.01) and budget 2.33 million sqft

Table 5.1 Objective function values for the instances solved

Budget (million sqft)	(w_1, w_2)	Total travel time (minutes)	Total population covered	% of Total population covered
1.43	(0.999,0.001)	1656.67	409572	89.46
	(0.99,0.01)	1670.96	412062	90.00
	(0.95,0.05)	1670.96	412062	90.00
	(0.9,0.1)	1686.42	412354	90.06
1.67	(0.999,0.001)	1442.33	415148	90.68
	(0.99,0.01)	1510.70	430861	94.11
	(0.95,0.05)	1758.24	437885	95.64
	(0.9,0.1)	1766.11	438561	95.79
2.00	(0.999,0.001)	1350.84	423192	92.44
	(0.99,0.01)	1454.16	441712	96.48
	(0.95,0.05)	1639.79	450129	98.32
	(0.9,0.1)	1903.84	449489	98.18
2.33	(0.999,0.001)	1342.47	433975	94.79
	(0.99,0.01)	1428.82	448727	98.01
	(0.95,0.05)	1601.52	452715	98.88
	(0.9,0.1)	1809.39	455482	99.49
2.67	(0.999,0.001)	1335.77	436682	95.38
	(0.99,0.01)	1407.88	450804	98.47
	(0.95,0.05)	1620.63	456039	99.61
	(0.9,0.1)	1786.13	455953	99.59
3.00	(0.999,0.001)	1335.77	436682	95.38
	(0.99,0.01)	1409.15	452018	98.73
	(0.95,0.05)	1561.31	455904	99.581
	(0.9,0.1)	1713.74	457111	99.85
3.33	(0.999,0.001)	1335.77	436682	95.38
	(0.99,0.01)	1409.15	452018	98.73
	(0.95,0.05)	1584.22	456184	99.64
	(0.9,0.1)	1635.3	456448	99.70

CHAPTER 6 RECOMMENDED CRITICAL ROUTES CONSISTENT WITH INDOT (SAC) NEEDS

This chapter reports the results from the computational experiments conducted using the case study after incorporating the recommendations of the Study Advisory Committee based on the critical route subnetworks identified in Chapter 5. Section 6.1 states the recommendations briefly and discusses them. Section 6.2 discusses the critical routes obtained by incorporating the recommendations.

6.1 Feedback from Study Advisory Committee and Emergency Response Operators

Based on the feedback from the SAC, the following recommendations were proposed for the critical routes:

- 1) Inclusion of the section of I-64 that passes through the study region.
- 2) Provision of connectivity of the critical routes to adjacent states, namely, Illinois and Kentucky.

The first recommendation attempts to address the omission of I-64 in the critical routes shown in Chapter 5. Since I-64 is a freeway, it would potentially provide an efficient critical route subnetwork component due to the superior structural and maintenance characteristics of freeways; that is, it would need minimal budgetary investments to ensure seismic tolerance. Hence, it would seem natural for it to be a part of at least some of the critical routes. To analyze the reasons for its omission in the critical route subnetworks suggested in Chapter 5, we revisit the model presented in chapter 3 that determines the critical routes. This model trades off three factors, namely total travel time, population covered and budget. Observing the figures (figures 5.2a-g) shown in Chapter 5, we note that the population centers close to I-64 that are not covered in most critical routes are Griffin, Elberfield and Lynnville. Their populations are 160,

636 and 781, respectively. Hence, together they constitute only 0.34% of the total population of the study region. Due to the budgetary constraints, the benefits of higher population coverage achievable through the routes other than I-64 are greater than the lower total travel time achieved due to its inclusion. Hence, the model excludes I-64 from the critical routes shown in Chapter 5.

Routes that provide connectivity to adjacent states are important. They aid in the coordination of response plans with adjacent states and also enable the response operators in Indiana to utilize the assistance offered by their counterparts in the adjacent states in the event of an earthquake primarily causing damage in Indiana. This potentially increases response effectiveness.

Both sets of recommendations were incorporated into the determination of the critical routes by adding the associated constraints into the original **MCNDP** formulation. This also means that the new formulation has a more restricted solution set compared to the original one. Therefore, the new objective function value (denoting the optimal solution for the more-constrained problem) incorporating these additional constraints can never be better than that of the original formulation corresponding to the results in Chapter 5.

6.2 Characteristics of the Recommended Critical Routes

After the addition of constraints to incorporate the SAC recommendations, the minimum budget required for a feasible solution increased to 1.67 million sqft. The critical routes were solved for budgets ranging from 1.67 million sqft. to 3.33 million sqft. Table 6.1 shows the total travel time and the total population covered for the various instances solved. Figure 6.1 depicts the noninferior solution frontier obtained. It illustrates that the marginal benefits realizable from a 2 million sqft. budget compared to the 1.67 million sqft. case are higher than those corresponding to the noninferior frontiers illustrated in Chapter 5 for the same budgets. The results (Figures 6.2a-f) suggest that the critical routes identified in Chapter 5 do not differ perceptibly from those computed here. For identical problem parameters, in most cases, the total travel time and the total population covered have worsened due to the recommendations. This is because the addition of new constraints leads to a more restricted solution set.

It may be noted that, in some instances, the results in Table 6.1 seem better than those shown in Table 5.1. This is not because the solution is inherently better in the restricted case. As just stated, the restricted *optimal* solution will always be worse. However, due to the problem complexity (NP-hard problem) discussed in Chapter 3, neither of these two cases is solved to optimality, but only to a certain optimality tolerance level. Hence, when more variables are fixed (as is done in Chapter 6), the problem can in many instances be solved much closer to optimality though the problem is more constrained.

Connectivity was provided at three sections with Illinois and two sections with Kentucky as desired by the SAC.

Figure 6.1 Noninferior solution set for different budget values

Figure 6.2a Recommended critical routes for the weight pair (0.99,0.01) and budget 1.67 million sqft

Figure 6.2b Recommended critical routes for the weight pair (0.9,0.1) and budget 1.67 million sqft

Figure 6.2d Recommended critical routes for the weight pair (0.95,0.05) and budget 2.00 million sqft

Figure 6.2e Recommended critical routes for the weight pair (0.99,0.01) and budget 2.33 million sqft

Table 6.1 Objective function values for the instances solved

Budget (million sqft)	(w_1, w_2)	Total travel time (minutes)	Total population covered	% of Total population covered
1.67	(0.999,0.001)	1662.63	408422	89.21
	(0.99,0.01)	1685.59	410802	89.73
	(0.95,0.05)	1686.69	410802	89.73
	(0.9,0.1)	1715.19	411855	89.96
2.00	(0.999,0.001)	1391.75	413091	90.23
	(0.99,0.01)	1466.27	429665	93.85
	(0.95,0.05)	1646.53	434609	94.93
	(0.9,0.1)	1755.28	434461	94.90
2.33	(0.999,0.001)	1346.02	427829	93.45
	(0.99,0.01)	1398.75	445473	97.30
	(0.95,0.05)	1584.67	449826	98.25
	(0.9,0.1)	1778.19	450736	98.45
2.67	(0.999,0.001)	1337.33	435301	95.08
	(0.99,0.01)	1425.29	450589	98.42
	(0.95,0.05)	1599.52	453614	99.08
	(0.9,0.1)	1695.34	455026	99.39
3.00	(0.999,0.001)	1328.15	436562	95.36
	(0.99,0.01)	1411.76	451015	98.51
	(0.95,0.05)	1592.24	455519	99.50
	(0.9,0.1)	1689.16	456991	99.82
3.33	(0.999,0.001)	1323.77	436682	95.38
	(0.99,0.01)	1407.76	452018	98.73
	(0.95,0.05)	1516.12	456048	99.61
	(0.9,0.1)	1614.49	457661	99.97

CHAPTER 7 CONCLUSIONS

This chapter summarizes the study and highlights its contributions.

7.1 Summary

Seismic retrofit planning plays an important role in determining the effectiveness of the operational stage of earthquake response management. At a strategic level, it is dependent on the routes critical for effective emergency response. However, past studies have not addressed the identification of critical routes under budget constraints. This is a very important practical problem for emergency response planners as it involves identifying effective transportation routes while accounting for bridge structural conditions and budget limitations. It implies the identification of bridges to retrofit under earthquake response planning. This study develops a network-level methodology to assist decision-makers in identifying an effective seismic retrofit scheme for the bridges of a network. It introduces the **MCNDP**, which seeks routes that minimize the total routing costs over the selected routes and maximizes the total demand covered, subject to a budget constraint. An integer programming formulation of the problem is presented, and is shown to be NP-hard. The problem characteristics are exploited to develop some valid inequalities.

In general, the **MCNDP** model is useful in addressing network-level disaster management planning. The **MCNDP** is applied to generate critical routes for earthquake response planning in southwest Indiana, which is part of a seismically active region that includes parts of Illinois, Kentucky and Missouri as well. The branch-and-cut procedure of the CPLEX MIP optimizer is used to solve the problem. A problem reduction strategy is proposed to reduce computational time while ensuring that the resulting solution is close to or equal to the optimal solution. Noninferior frontiers are generated using convex

combinations of the two objectives so as to provide practical insights for decision-makers. These frontiers assist the decision-maker in analyzing the trade-offs among various objectives and making budget allocation decisions.

A new set of critical routes were developed based on the SAC feedback for the first set of critical routes identified. The SAC recommendations were to include the section of I-64 passing through the study region, and to provide connectivity to the adjacent states (Illinois and Kentucky) to ensure seamless inter-state coordination and cooperation in the event of an actual earthquake. These recommendations were incorporated to generate a new set of critical routes through the addition of constraints to the **MCNDP** formulation. The updated critical routes do not differ perceptibly from those obtained from the experiments prior to the SAC recommendations.

7.2 Contributions of the Study

This study formally defines the Multicommodity Maximal Covering Network Design Problem (**MCNDP**) and formulates it. To our knowledge, it represents the first attempt to incorporate the coverage objective in a generic network model, leading to a new class of models with significant practical implications. Past work has used the coverage criterion in identifying a path, tree or tour on a network by considering either the routing or the fixed cost, while the proposed model simultaneously considers the routing and fixed costs subject to a budget constraint. That is, none of the existing models consider routing costs, fixed costs and coverage criterion in a single framework.

From a practical standpoint, the key contribution of this study is its ability to address planning problems faced by emergency response agencies vis-à-vis disaster management. The specific problem addressed here relates to earthquake response management. Under budget constraints, there is a need to determine an effective retrofit plan for the bridges in a seismically-prone region. Most studies in the literature have adopted a local perspective to solving this problem, and do not consider the effect of potential bridge failures on the transportation system performance. The few methodologies that use a systems approach do not consider the influence of retrofit decisions on the quality of the associated routes vis-à-vis effective earthquake response. The proposed model fills this critical gap by

considering the total travel time for the key O-D pairs in the network and the total population covered by them as the criteria for determining the critical routes. The budget limitation on the total bridge retrofit costs for the links constituting the critical routes is represented as a constraint in our model. Hence, this methodology simultaneously determines the set of bridges in a network requiring retrofit and the routes that serve as focal points for earthquake response.

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Final Report: Part II

FHWA/IN/JTRP-2003/22

EARTHQUAKE EMERGENCY ROUTES FOR THE STATE OF INDIANA
ROUTE SEISMIC VULNERABILITY ASPECTS

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INDOT Research

TECHNICAL *Summary*

Technology Transfer and Project Implementation Information

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August 2005
Final Report

Emergency Earthquake Routes for the State of Indiana (A Two-Volume Report)

Introduction

The possibility of very strong earthquakes in the central Mississippi Valley has been known for 200 years in the New Madrid Seismic Zone. Awareness of the seismic risk in the State of Indiana has increased in the last decades. In the last two decades identification of an independent tectonic process occurring in the Wabash River Valley has implications for the seismic risk of the State of Indiana from what has recently been defined as the Wabash Valley Seismic Zone. The occurrence of a strong earthquake in these seismic zones or in Indiana or the neighboring states due to other not so well understood tectonic processes, as has occurred in the past, has serious implications for the State Transportation System. The definition of Earthquake Emergency Routes for the state is a priority for the Indiana Department of Transportation. The identification of Emergency Routes takes into account issues related to transportation including coverage of population and area and travel time along these routes.

This study formulates a multicommodity maximal covering network design problem (**MCNDP**) for identifying critical routes, for earthquake response and to seismically retrofit bridges. The **MCNDP** seeks routes that minimize the total travel time over the selected routes and maximizes the total population covered, subject to a budget constraint on bridge retrofitting costs on the selected routes. The problem is formulated as a two-objective

integer programming model and solved using the branch-and-cut module in the CPLEX optimizer. The model performance was analyzed using the transportation network of a seismically-prone region in southwest Indiana. Thereby, the search for the critical routes for an origin-destination (O-D) pair is confined to a limited geographical region around it.

A Geographical Information System (GIS) system was developed for evaluation of seismic vulnerability of the proposed Indiana Emergency Routes. The GIS based implementation of the technical information developed in this project through FEMA HAZUS-MH software was conducted in collaboration with the Polis Center in Indianapolis, nationally recognized and FEMA authorized HAZUS implementation facility. HAZUS software may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in southwestern Indiana. It uses information from the Indiana Department of Transportation (INDOT) maintenance database and selected structural and geotechnical information from the bridge drawings data base. Vulnerability assessment was performed using the general information from maintenance and final calibration was performed using a series of cases based on bridge drawings and soil exploration descriptions contained in these drawings.

Findings

Computational experiments were conducted to investigate the effects of varying the budget and the relative weights of the two objectives. A set of

critical routes was obtained. Seismic retrofit planning plays an important role in determining the effectiveness of the operational stage of earthquake

response management. At a strategic level, it is dependent on the routes critical for effective emergency response. However, past studies have not addressed the identification of critical routes under budget constraints. This is a very important practical problem for emergency response planners as it involves identifying effective transportation routes while accounting for bridge structural conditions and budget limitations. It implies the identification of bridges to retrofit under earthquake response planning. This study develops a network-level methodology to assist decision-makers in identifying an effective seismic retrofit scheme for the bridges of a network.

This study formally defines the Multicommodity Maximal Covering Network Design Problem (MCNDP) and formulates it. To our knowledge, it represents the first attempt to incorporate the coverage objective in a generic network model, leading to a new class of models with significant practical implications. Past work has used the coverage criterion in identifying a path, tree or tour on a network by considering either the routing or the fixed cost, while the proposed model simultaneously considers the routing and fixed costs subject to a budget constraint. That is, none of the existing models consider routing costs, fixed costs and coverage criterion in a single framework.

From a practical standpoint, the key contribution of this study is its ability to address planning problems faced by emergency response agencies vis-à-vis disaster management. The specific problem addressed here relates to earthquake response management. Under budget constraints, there is a need to determine an effective retrofit plan for the bridges in a seismically-prone region. Most studies in the literature have adopted a local perspective to solving this problem, and do not consider the effect of potential bridge failures on the transportation system performance. The few methodologies that use a systems approach do not consider the influence of retrofit decisions on the quality of the associated routes vis-à-vis effective earthquake response. The proposed model fills this critical gap by considering the total travel time for the key O-D pairs in the network and the total population covered by them as the criteria for determining the critical routes. The

budget limitation on the total bridge retrofit costs for the links constituting the critical routes is represented as a constraint in our model. Hence, this methodology simultaneously determines the set of bridges in a network requiring retrofit and the routes that serve as focal points for earthquake response.

The evolution of the earthquake resistant bridge design requirements contained in the AASHTO Specifications and the existence of a proposed draft seismic design specification being discussed have significant implications in the assessment of the existing vulnerability, its mitigation, and the design of new bridges along the Emergency Routes. Along with this development, the USGS assessment of the seismic risk of southwestern Indiana and a change of format of the definition of the earthquake design ground motion that is integral part of the proposed draft seismic design specification also has important implications on the assessment of the operational capabilities of the Earthquake Emergency Routes of Indiana. The effects of the 2001 Nisqually Earthquake that occurred in the State of Washington gave invaluable insight on the expected situation for the State of Indiana due to a comparable bridge inventory in age and bridge types and a parallel increase of the seismic risk awareness in both regions over the last decades.

Using the information collected in this study, the GIS implementation through FEMA-HAZUS-MH software may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in southwestern Indiana. Vulnerability was evaluated for different levels of ground acceleration in order to obtain behavior patterns for selected bridges. Using these results, seismic behavior patterns were obtained for bridges located in southwestern Indiana included in the INdot Vincennes District maintenance data base. Soil amplification and liquefaction potential were evaluated for the cases having reported SPT measurements from borings. Based on this process, information critical for the identification of upgrade needs for the transportation structures part of the Earthquake Emergency Routes is made available to the INdot.

Implementation

The findings and developments of this research project are presented next in the form of suggested implementation steps by the Indiana Department of Transportation.

1. Formal adoption of the selected emergency routes or some variation of the same by the state is a necessary step. Such routes adopted and maintained by INdot will be affected by issues and policies outside the scope of those considered in this study. For instance, policy decisions, budgetary constraints, new projects, i.e. continuation of I-69, will likely affect the formal adoption of a set of emergency routes and will continue to impose changes in the future. The methodology developed in this study and the information implemented in HAZUS-MH should be of assistance to state decision makers in the future.
2. Future implementation strategies can be studied through simulation studies using appropriately updated information for different earthquake scenarios including earthquake ground motion defined in current AASHTO Standard Specifications, proposed Draft AASHTO LRFD Seismic Provisions, or other. These simulations will provide corresponding bridge retrofit scope for the proposed routes or alternative definitions.
3. The information in the maintenance database should be periodically evaluated and should include information currently available only in the bridge drawings (see Section 7.5 of the final report). This will improve the vulnerability assessment of the bridges refining the mitigation policies to adopt in the future. This expansion of the maintenance database can be extended to other counties and districts not covered in this research project.
4. Establish a program for assessing the liquefaction, soil spread, and embankment failure potential along the routes at locations other than bridge sites by providing a consistent evaluation methodology supported with the information developed in this study. This implementation will require a soil exploration program to obtain geotechnical properties of sites where no information exists or where such information was obtained years ago with different objectives than liquefaction evaluation. In the research conducted in this study, liquefaction evaluations were made using solely boring information contained in selected bridge drawings.
5. To establish procedures for emergency response under different earthquake occurrence scenarios.
6. INdot should consider the formal adoption of HAZUS-MH software for scenario management, mitigation and vulnerability studies and to train appropriate personnel. An important feature in this implementation step is the engagement of the Polis Center in the training of INdot personnel on the use of HAZUS-MH software. The Polis Center is a nationally recognized HAZUS implementation facility.

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<p>The occurrence of a strong earthquake in Indiana or neighboring states, as has occurred in the past, has serious implications for the State Transportation System with respect to adequate response. The definition of Earthquake Emergency Routes for the State of Indiana became a priority for the Indiana Department of Transportation. These Emergency Routes take into account various aspects related to transportation including coverage of population and area and travel time along these routes, and issues related to structural and geotechnical seismic vulnerability of the roads and bridges along these routes.</p> <p>Part I of the overall study report formulates a multi-commodity maximal covering network design problem (MCNDP) for identifying critical routes, for earthquake response and to seismically retrofit bridges. In the MCNDP, routes are sought that minimize the total travel time over the selected routes and maximize the total population covered, subject to a budget constraint on bridge retrofitting costs on the selected routes. The problem is formulated as a two-objective integer programming model and solved using the branch-and-cut module in the CPLEX optimizer. The model performance is analyzed using the transportation network in southwest Indiana. A problem reduction strategy is introduced to reduce computational times by recognizing that the critical routes are not usually circuitous. Thereby, the search for the critical routes for an origin-destination (O-D) pair is confined to a limited geographical region around it. To further reduce computational costs, the formulation is extended to incorporate valid inequalities that exploit the problem structure. Simulation studies are conducted to investigate the effects of varying the budget and the relative weights of the two objectives. Noninferior frontiers that illustrate the trade-offs between the conflicting objectives for different budgets are constructed to provide practical insights to decision-makers. The final selected routes in addition include constraints set by the Study Advisory Committee regarding specific roads to be included in the final set.</p> <p>Part II of the study report covers the structural and geotechnical seismic issues related to the definition of Earthquake Emergency Routes for the State of Indiana. Previous worldwide experience on bridge damage during strong earthquakes has influenced the bridge earthquake design specifications and mitigation procedures. The evolution of the relevant requirements in the AASHTO Specifications and the existence of a proposed draft seismic design specification under development as part of the NCHRP Project 12-49 have significant implications in the assessment of the existing vulnerability, its mitigation, and the design of new bridges along the Emergency Routes. Geographical Information System (GIS) was implemented to assist the evaluation of seismic vulnerability of the proposed Indiana Emergency Routes. Using the available information, the GIS implementation may be used for mitigation of vulnerability, simulation, and response to a strong earthquake in Indiana incorporating information from the Indiana Department of Transportation (Indot) maintenance database and selected structural and geotechnical information from the relevant bridge drawings. Findings from a vulnerability assessment as well as final calibration performed using selected representative bridge data such as drawings and soil exploration descriptions are also reported. Vulnerability was evaluated for different levels of ground acceleration in order to obtain behavior patterns for selected bridges and to illustrate upgrade needs for the transportation structures on the Emergency Routes. The information contained in this report could be used to assist the engineers in conducting simulations that will help Indot to devise mitigation policies for different earthquake occurrence scenarios and to evaluate potential alternative routes.</p>			
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ABSTRACT

Awareness of the seismic risk in the State of Indiana has increased in the last decades. The possibility of very strong earthquakes in the central Mississippi Valley has been known for 200 years in the New Madrid Seismic Zone. In the last two decades identification of an independent tectonic process occurring in the Wabash River Valley has implications for the seismic risk of the State of Indiana from what has recently been defined as the Wabash Valley Seismic Zone. The occurrence of a strong earthquake in these seismic zones, or in Indiana, or in the neighboring states due to other not so well understood tectonic processes as has occurred in the past, has serious implications for the State Transportation System. The definition of Earthquake Emergency Routes for the State of Indiana became a priority for the Indiana Department of Transportation. The definition of these Emergency Routes takes into account issues related to transportation including coverage of population and area and travel time along these routes, and issues related to structural and geotechnical seismic vulnerability of the transportation structures along these routes. The transportation related issues are presented in Part I of this Report, and the structural and geotechnical seismic related issues are presented in Part II. Previous experience in the United States and elsewhere in the world with bridge damage during strong earthquakes has influenced the bridge earthquake resistant specifications and mitigation procedures. The effects of the Nisqually Earthquake that occurred in the State of Washington in 2001 give insight on the expected situation for the State of Indiana due to a comparable bridge inventory in age and bridge types and a parallel increase of the seismic risk awareness in both regions over the last decades. The evolution of the earthquake resistant bridge design requirements contained in the AASHTO Specifications and the existence of a proposed draft seismic design specification being discussed have significant implications in the assessment of the existing vulnerability, its mitigation, and the design of new bridges along the Emergency

Routes. Along with this development, the USGS assessment of the seismic risk of southwestern Indiana and a change of format of the definition of the earthquake design ground motion that is an integral part of the proposed draft seismic design specification also has important implications on the assessment of the operativeness of the Earthquake Emergency Routes of Indiana.

Vulnerability assessment was performed using the general information from inspection and maintenance, and final calibration was performed using a series of cases based on bridge drawings and soil exploration descriptions contained in these drawings. Vulnerability was evaluated for different levels of ground acceleration in order to obtain behavior patterns for selected bridges. Using these results, seismic behavior patterns were obtained for bridges located in southwestern Indiana included in the Indot Vincennes District Inspection and Maintenance Database. Soil amplification and liquefaction potential were evaluated for the cases having reported SPT measurements from borings. Based on this process, information, upgrade needs for the transportation structures part of the Earthquake Emergency Routes were obtained. With this information, it is possible to perform simulations that will help Indot in devising mitigation policies, perform simulations for different earthquake occurrence scenarios, and establish, evaluate, and implement alternatives for response strategies using the Earthquake Emergency Routes.

The Federal Emergency Management Agency (FEMA) has developed HAZUS-MH, a GIS-based software tool that empowers communities with the ability to assess risk from earthquakes, flood, and wind related disasters. While HAZUS-MH can be used to assess risk with the significant default inventory that it includes, it is possible to integrate local data into the HAZUS-MH analysis in order to produce more realistic loss estimates. As a component of the research work conducted in this project and with the approval of the Study Advisory Committee, The Polis Center from Indianapolis was tasked with the integration of bridge and soils data into HAZUS-MH for the purpose of using that tool to conduct earthquake risk analysis. The Polis Center is a FEMA authorized HAZUS-MH earthquake and flood service provider and is thus able to support the goals of INDOT in

this instance. The GIS implementation through FEMA HAZUS-MH software conducted during this study uses information from the Indiana Department of Transportation (Indot) Inspection and Maintenance Database and structural and geotechnical information from the bridge drawings data base.

CHAPTER 1 INTRODUCTION

1.1 Purpose

The purpose of the project was to propose Earthquake Emergency Routes for the State of Indiana with emphasis in the southwestern part of the State, to review the selected routes for earthquake hazards identifying conditions related to the structures along those routes that may affect their functioning in case of occurrence of a strong earthquake in the region, and to propose procedures for mitigation, evaluation and maintenance of their status as emergency routes.

1.2 Scope

1. To recommend Earthquake Emergency Routes for the Vincennes District of the Indiana Department of Transportation (INDOT), located in the southwestern corner of the State, based on travel times and scope of retrofit work necessary to implement these routes with the final decision of which routes to select resting with INDOT (see Part I of this report).
2. To develop a tool based on geographical information systems technology (GIS) that INDOT engineering may use for identifying and maintaining the Earthquake Emergency Routes network of Indiana and for establishing policies for mitigation of earthquake hazard on these routes, to perform simulations for different earthquake scenarios, and to help in the response to an actual emergency caused by the occurrence of a strong earthquake in the region.
3. To assess the seismic vulnerability of selected bridges belonging to the Vincennes INDOT District based on detailed information, and to correlate this vulnerability assessment with one obtained using just the information currently contained in the INDOT Inspection and Maintenance Database. To recommend items relevant to

the vulnerability of the routes to be added to the Inspection and Maintenance Database.

4. To give guidance on how to prioritize the mitigation of the vulnerability of existing bridges according to the analyses performed and their importance in keeping the transportation network system functional with emphasis in the Earthquake Emergency Routes selected.

1.3 Methodology

Given the multidisciplinary nature of the project, the methodology used took advantage of the most advanced resources of each discipline. The methodology corresponding to each of the components is described in detail along this report at the pertinent locations, with Chapter 4 of this report presenting a summary of the vulnerability assessment methodology used.

1.4 Participants

Joint Transportation Research Program – JTRP

Kumares C. Sinha, Director

Study Advisory Committee

Indiana Earthquake Preparedness Committee

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1.5 Organization of the report

This report is presented in two volumes; the first one – Part I – deals with Transportation issues related to definition of the selected Earthquake Emergency Routes; this second volume – Part II – is organized in the following manner:

Chapter 2 presents background on seismic vulnerability of transportation networks. It includes the reasons behind the need of defining earthquake emergency routes. Discusses the seismic vulnerability issues linked to transportation structures. A summary of the performance of transportation networks in past earthquakes is presented. Finally, it presents and discusses the development of the specifications that govern their design including associated geotechnical issues.

Chapter 3 discusses the tectonic setting of the southwestern corner of Indiana and how it fits within the two main regional seismogenic structures – the New Madrid Seismic Zones and the Wabash River Faulting System – commenting on the earthquakes that have affected the region. The earthquake mean return period is presented and the importance of its definition for earthquake resistant design of structures is brought into

perspective by describing the values being used in current and proposed earthquake resistant bridge design specifications. The implications of adopting a draft seismic design specification by AASHTO are discussed.

Chapter 4 presents the methodology for evaluation of seismic vulnerability of the selected Indiana Emergency Routes, the available information, the Geographical Information System (GIS) implementation and its use for mitigation of vulnerability, simulation, and response to a strong earthquake in southwestern Indiana.

Chapter 5 is devoted to geotechnical sources of vulnerability for transportation structures in general and, in particular, the approach used in defining and assessing these sources of vulnerability for the Earthquake Emergency Routes of the State of Indiana.

Chapter 6 parallels the previous chapter, with the focus on bridges. The methodology used and its application is discussed in detail.

Chapter 7 defines the upgrade needs for the transportation structures part of the Earthquake Emergency Routes.

Chapter 8 presents the conclusions and recommendations of the study.

Chapter 9 lists the bibliographic references relevant to the study.

CHAPTER 2 BACKGROUND

This chapter describes previous background information related to earthquake emergency routes and performance during earthquakes of highway systems, transportation facilities, and structures with special reference to the State of Indiana. This background information has been divided in information related to highway systems, bridges, and issues associated with geotechnical aspects.

2.1 Transportation systems

The Federal Response Plan (FRP) of the United States [FEMA, 2003] created under the auspices of the Robert T. Stafford Act, which is the enabling legislation for emergency response in the United States, provides the legal mandate, the resources, and the mechanisms for an integrated emergency response after the declaration of any disaster by the President. The Federal Response Plan establishes 12 Emergency Support Functions (ESF) to facilitate the implementation and coordination of a broad spectrum of integrated emergency response activities after natural and technological disasters.

ESF No. 1 is “Transportation” and is defined as access to the disaster area. It is based on the assumptions that the transportation infrastructure in the area will sustain damage, which will influence the means and accessibility of relief services and supplies to protect people and property. The disaster responses requiring transportation capacity will be difficult to coordinate effectively during the immediate post-disaster period. Clearing of debris and completion of repairs will be gradual, in spite of best efforts, disrupting access routes for a significant period.

Transportation systems in the Central U.S. – including highways, bridges, railways, waterways, ports, and airports – are vulnerable to the effects of a damaging earthquake in the New Madrid Seismic Zone (NMSZ) [CUSEC, 2000]. Furthermore, damages to transportation systems may extend to several states, which present transportation officials in government and the private sector with unique problems and challenges.

2.3.1 Ground motion amplification due to soil profile characteristics

Association of earthquake damage with poor soil conditions have been known for many years. Early approach to the problem was manifest in seismic codes through a difference in seismic design forces associated with bearing capacity used in foundation design (see 2.2.2). With the increase deployment of strong ground motion accelerometers from the 1950's on, instrumental evidence of the influence of the underlying soil profile permitted interpretations of earthquake damage in several events such as the 1957 San Francisco, California, Earthquake; the 1957 Mexico City Earthquake; the 1964 Anchorage, Alaska, Earthquake; and the 1967 Caracas, Venezuela, Earthquake.

From the mid 1970's on, a clearer picture emerged and different approaches were adopted to account for the effect of amplification by the soil profile. For the first time in the 1974 SEAOC Blue Book Lateral Force Requirements for Buildings [SEAOC, 1974] included a coefficient S , called then Coefficient for Site-Structure Resonance having values between 1.0 and 1.5 depending on the ratio between the fundamental period of the structure, T , and a characteristic site period T_s . The 1974 SEAOC Requirements included an Appendix for obtaining a range of values for the soil deposit characteristic period. This procedure was adopted by the 1976 Uniform Building Code and was kept for several issues of this Code.

Studies conducted for moderate magnitude events in the western U.S. [Seed et al., 1976a] permitted grouping different soil profiles into broad categories [Seed et al., 1976b]. This type of soil profiles and the corresponding amplifications were combined with studies of spectral shapes and were included in the ATC-3-06 project [ATC, 1978] for incorporation in model building earthquake resistant regulations. This classification comprised three types of soil profiles namely: Type 1 – Stiff soils and rock, Type 2 – Deep cohesionless or stiff clay soils, and Type 3 – Soft to medium clays and sands. As part of the requirements a soil coefficient, S , was introduced. This coefficient amplified the medium to long period part of the acceleration spectrum in rock to model the effects of the soil according to the soil profile type by factors of 1.0, 1.2, and 1.5 respectively. This is the same soil profile classification that was used in the seismic bridge design recommendation ATC-6 project [ATC, 1981].

The Central U.S. Earthquake Consortium (CUSEC) is a nonprofit organization, funded by the Federal Emergency Management Agency, which is dedicated to reducing deaths, injuries, damage to property and economic losses resulting from earthquakes occurring in the Central United States. Its members are the seven states that are most vulnerable to earthquakes in this region: Arkansas, Illinois, Indiana, Kentucky, Mississippi, Missouri, and Tennessee.

Fig. 2-1 – Map showing hypothetical maximum intensities for a magnitude 7.6 earthquake in the Central United States region [CUSEC, 2000]

The document [CUSEC, 2000] discusses the transportation system vulnerability and the earthquake risk in the Central United States taking into account the multi-state impact of future damaging earthquakes in the region including associated problems with earthquake induced hazards such as faulting, liquefaction, slope instability, dam or levee failure, and hazardous materials spills. The effects of earthquakes on the transportation system including: highway, railroad, waterway, and air transportation and liquid fuel transport; the steps that must be taken for mitigation of vulnerability of transportation systems in the region; and response and recovery policies are discussed in this important document. It assigns the following consequences of failure in a transportation system due to an earthquake or other natural disaster:

- Direct loss of life due to collapse or structural failure of the lifeline.
- Indirect loss of life due to an inability to respond to secondary catastrophes, such as fires, and/or provide emergency medical aid.
- Delayed recovery operations.
- Release of hazardous products (e.g., losses from tank cars derailed by track failure, gas leaks from ruptured utility lines) and environmental impacts.
- Direct loss of property and utility service (e.g., the collapse of a bridge carrying utilities).
- Losses due to interruption of access (e.g., export losses due to port damage).
- Disruption of economic activity across the region and nation as well as in the community directly affected.

The [CUSEC, 2000] document proposes a New Madrid Transportation Plan and Strategy, which would be the product of an intergovernmental-private sector planning process, aimed at addressing the following:

1. Loss estimates and functionality assessments for selected earthquake scenarios.
2. Common set of planning assumptions for federal, state, and local governments.
3. Criteria for decisions on establishing field operations in a multi-state, multiple region disaster.
4. Proper allocation of resources to multiple impacted areas.

5. State versus Federal roles in determining priorities and planning for infrastructure repair.
6. Procedures and criteria for conflict resolution in meeting requests for resources.
7. Pre-disaster determination of conditions that must be present before federal transportation support is withdrawn.

Similar tasks have been studied in other regions of the U.S., but there has been none requiring such a broad coordination between different States and the Federal Government. The role played by CUSEC and the State Departments of Transportation in implementing this plan can not be underestimated. At present, CUSEC has compiled the Emergency Routes shown in Fig. 2-2.

Fig. 2-2 – Earthquake Emergency Routes and staging facilities of the CUSEC State Departments of Transportation

These routes are currently under study individually by each State. Detailed literature review on transportation issues is presented in Part I of this report.

2.2 Bridges

2.2.1 Bridge performance during earthquakes

In general, the likelihood of bridge damage increases if the ground motion is particularly intense, the soils are soft, the bridge was constructed before modern codes were implemented, or the bridge configuration is irregular [Moehle and Eberhard, 2000]. Depending on these conditions, bridge damage could be grouped in the following general categories:

1. *Unseating of the superstructure* — Short seat lengths at span hinges or simple supports permit the partial or total collapse of the superstructure if the displacements caused by the ground motion exceed the support length. Curved bridges are especially vulnerable for this condition. Restrainers have been used for several years to mitigate this problem; although, there were cases in which they were not effective during the Northridge Earthquake [EERI, 1995a].
2. *Column failure* — Lack of adequate confining transverse reinforcement leads to brittle modes of failure of reinforced concrete columns and piers. Short lap splices or inadequate embedment length of anchorage at the foundation of vertical reinforcement has led to numerous failures. For steel columns, local buckling may allow damage leading to collapse.
3. *Damage to abutments* — The abutment is affected by the underlying soil conditions, the type of foundation, and the demands imposed by the superstructure movement. Usually, abutments are elements where it is difficult to provide ductility and toughness.
4. *Damage to bearings* — Bearings used in bridges designed only for gravity effect are especially prone to failure under horizontal displacement of the superstructure transverse to the direction of thermal expansion.

5. *Structural configuration* — Unique complex structures with curved or significant skew geometry, or bridges having unique features or details, are more susceptible to damage induced by ground motion.
6. *Age, Modifications and Maintenance* — Bridge design specifications have changed in recent years as a result of experience from earthquakes and structural research. The correlation between bad behavior and old bridges is high as observed in many recent earthquakes. Changes introduced after original construction sometimes affect behavior. Numerous cases in which protection barriers or walls reduce the clear height of columns have led to failures under earthquake ground motions. Deterioration of the bridge due to corrosion and other causes has had an influence in the observed behavior.
7. *Geotechnical conditions* — Liquefaction, soil spread, slope instability, proximity to the fault, approach settlement, and other causes of distress influence the response, and damage of the bridge.

The following publications describe extensively bridge behavior and damage during earthquakes [ACI, 1999], [Moehle and Eberhard, 2000], [Penzien, 2000], and [Priestley et al., 1996]. Reports from earthquakes where damage to bridges is specially described are: San Fernando, California, 1971 Earthquake [Lew et al., 1971]; Chile 1985 Earthquake [EERI, 1986]; Whittier Narrows, California, 1987 Earthquake [EERI, 1988]; Loma Prieta, California, 1989 Earthquake [EERI, 1990] and [NRC, 1994]; Costa Rica 1991 Earthquake [EERI, 1991]; Northridge, California, 1994 Earthquake [EERI, 1995a] and [CRSI, 1994], the Kobe, Japan, 1995 Earthquake [EERI, 1995b]; the Turkish Earthquakes of 1999 [EERI, 2000]; the Taiwan Earthquake of 1999 [EERI, 2001], and the 2001 Nisqually, Washington State, Earthquake [Ranf et al., 2001], and [Nisqually EQ Clearinghouse Group, 2001]. Just a summary will be presented limiting it to cases of bridges designed using U.S. or comparable specifications. Direct reference is made to the relevant U.S. earthquakes. Even with these limitations, several things have to be kept in mind in translating this information for application in the State of Indiana. These issues will be discussed after the performance information is presented.

Although there is record of previous damage to bridges both in the U.S. and abroad, the February 9, 1971, San Fernando, California, Earthquake constitutes the first case on intense damage to bridges of modern design that belonged to the Interstate system [Lew et al., 1971]. The 1971 San Fernando Earthquake affected the greater Los Angeles area and specially the northern San Fernando Valley. Post-earthquake reports by the California State Highway Commission indicated that the earthquake damaged approximately 11 miles of multilane freeways and 6 miles of conventional state highways, in addition to numerous city and county streets. Approximately 1/5 of the total damage cost to transportation infrastructure was represented in damage to sixty bridges. Several of these bridges either collapsed or were severely damaged to the point of having to be replaced. Considerable disruption of traffic was caused by the bridge damage. The heavily traveled Interstate 5 was reopened to traffic using emergency detours within five days of the earthquake occurrence. Approximately 80% of the bridge damage cost was concentrated in 4 miles of highway at the intersection of I-5, I-210, and State Road 14. It is interesting to note that one of the collapsed curved bridges rebuilt after the San Fernando Earthquake collapsed during the Northridge 1994 Earthquake.

Fig. 2-3 – Bridge collapse during the February 9, 1971, San Fernando Earthquake

The Loma Prieta, California, 1989 Earthquake [EERI, 1990] provided insight on the influence of geotechnical conditions on bridge behavior by the preponderance of damage in the perimeter of San Francisco Bay where relatively deep and soft soil deposits amplified the ground motion. The collapsed portions of the two-story Cypress Street viaduct (see Fig. 2-4) coincide with the soft soil sites. Soil conditions may have played a role in damaging a structure of 1950's with detailing common at that time that did not provide toughness.

Fig. 2-4 – Collapse of the Cypress Street Viaduct during the Loma Prieta, California, 1989 Earthquake

Loma Prieta Earthquake also highlighted the vulnerability of adjacent short and long spans imposing differential deformation demands and producing the collapse of the short span next to a large span (see Fig. 2-5). During this earthquake there were five bridge collapses (counting the extensive Cypress Viaduct as one) and four other bridges sustained major damage.

Fig. 2-5 – Collapse of a span of the San Francisco-Oakland Bay Bridge during the Loma Prieta, California, 1989 Earthquake

The Costa Rica 1991 Earthquake [EERI, 1991] confirmed the influence of soil conditions on bridge behavior especially in abutment failures, embankment settlement, and liquefaction. Collapses caused by unseating of skewed spans were also observed.

The 1994 Northridge Earthquake, [EERI, 1995a] and [CRSI, 1994], produced high accelerations throughout the Los Angeles metropolitan area where the California Department of Transportation owns and maintains approximately 2,200 bridges. Approximately 1,200 were located in zones that experienced horizontal accelerations more than $0.25g$ and several hundred bridges were subjected to accelerations more than $0.5g$ [ACI, 1999].

Five bridges presented partial or complete collapse. All these structures were of reinforced and/or prestressed concrete, with construction completion dates ranging from 1964 through 1976. Several had been retrofitted with hinge restrainers following the 1971 San Fernando Earthquake. Collapse causes in all cases appear to involve column flexural or shear failures (see Fig. 2-6), or unseating at in-span or abutment hinges (see Fig. 2-7). Three reinforced and/or prestressed concrete bridges sustained major to moderate damage

to their columns but did not collapse. There was damage to bridge abutments throughout the epicentral region.

Fig. 2-6 – Failure of flared column in the Route 118, Mission Gothic Undercrossing during the Northridge, California, 1994 Earthquake

The observed abutment damage included shear key cracking and failure, spalling or failure of abutment backwalls and wingwalls, approach settlement, and approach slab buckling. Damage in steel bridges comprised pounding damage between adjacent elements, buckling of cross bracing, bending of cross-brace gussets, bearing damage including anchor bolt and restrainer fractures, and damage to supporting abutments and pier walls.

The epicentral region contained 132 bridges that had been retrofitted using post San Fernando, 1971, details (Caltrans Phase I), and 63 with post Loma Prieta retrofit details (Caltrans Phase II). Retrofit consisted in hinge restrainers and/or column jackets. Most of the retrofitted bridges performed adequately although, in some cases, hinge restrainers

presented problems. Column jackets, whether made of steel or fiberglass, performed well in all cases.

Fig. 2-7 – Collapse of the Gavin Canyon Undercrossing during the Northridge, California, 1994 Earthquake

The Los Angeles County Department of Public Works reported serious damage in only four out of 1,500 bridges on the county system. The City of Los Angeles reported that 62 out of a total of 800 spans were damaged, and two required closure due to column damage in one case, and fill settlement in the other. Other damage to city bridges consisted of approach-fill settlement, pavement cracking, shear key damage, superstructure rotation, rocker bearing damage, and architectural damage.

Several important observations were made regarding the performance of the transportation system [EERI, 1995a].

- In this earthquake, all bridge collapses were associated with poor performance of older columns with inadequate transverse reinforcement and short seat widths, both of which have been identified as sources of vulnerability in previous earthquakes. The progress in knowledge and design practice was evident in the good performance of bridges constructed or retrofitted to current standards.

- Architectural column flares affected the pier behavior. Earlier design practice assumed that flares spalled off, whereas actually they act as an integral part of the column.
- Current and older design procedures may not adequately represent the distribution of design forces in long, multiframe bridges. Failure and damage in end frames indicates that the stiffer end frames may attract larger forces than anticipated in design.
- As with other construction forms, structural changes can occur during construction and maintenance that invalidate the design assumptions. It is important to verify that as-built conditions are consistent with design assumptions, and not just the design drawings. Improved documentation of design assumptions may be desirable.
- Caltrans Phase I hinge restrainer retrofits had mixed performance. There were many examples of restrainer unit failure (cable fracture, fitting failure, and diaphragm punching).
- Older bridges with 6-inch and 8-inch seats should be reevaluated, even if retrofitted with restrainers.
- Caltrans Phase II retrofits appeared to perform well in all instances. (Its focus was on retrofitting columns in single-column bridges because of the perception that they were more vulnerable than multicolumn bridges.)
- Several skewed bridges collapsed. An apparent cause was torsional response associated with skewed geometries. Methods are needed for improved design of skewed bridges, possibly including elimination of the skew where feasible, elimination of in-span hinges, and lengthening of seats.
- Damage to abutments and approaches was widespread. Considering the extent of this damage for a moderate earthquake, it seems that the current design strategy to accept abutment damage should be reevaluated.
- Major damage and collapse in multicolumn bridges indicates that the increased reliability associated with redundancy of multicolumn bents is not necessarily sufficient to avoid collapse. Retrofit priorities given to multicolumn bridges and

high-traffic-volume bridges may need to be reconsidered. Furthermore, damage and collapse required closure of bridges that usually carry heavy traffic volume.

- Modern traffic management techniques were effective in managing traffic following the earthquake. Furthermore, emergency procedures, including strong incentive and disincentive clauses, were effective in achieving rapid reconstruction of the freeways.

The February 28, 2001, Nisqually, Washington State, Earthquake affected the Seattle, Tacoma, Olympia region [Ranf et al., 2001], and [Nisqually EQ Clearinghouse Group, 2001]. No bridge collapses were observed during this earthquake. In total, 78 bridges were damaged. A total of 10 bridges sustained moderate damage, 16 had mild damage, and 52 bridges had minor damage. The event had a moment magnitude of 6.8 and a depth of focus of 52 km on the Juan de Fuca Plate. The distance from the hypocenter to Seattle was 78 km, 57 km to Tacoma, and 54 km to Olympia.

Fig. 2-8 – Damage to the I-5 Holgate Street overpass, Seattle, during the 2001 Nisqually, Washington State, Earthquake

The Nisqually Earthquake caused moderate ground motion throughout the Puget Sound region. Of the 31 stations for which preliminary information was available, only 13 showed peak ground accelerations more than 10%g and only 2 stations recorded values more than 25%g. Several correlations of damage with bridge characteristics and ground motion intensity were made [Ranf et al., 2001]. For 72 bridges that were damaged (6 movable bridges were excluded because of their unusual characteristics) types of damage were: 48 with damage to concrete, 6 with damage to steel, 11 with damage to beams, restrainers or joints, and 7 with damage due to settlement.

In order to help interpret the correlation of damage with age of the bridge, Figure 2-9 shows the age of construction of the total bridge inventory in the region. The number of bridges built increased from the beginning of the 1950's and then decreased at the beginning of the 1980's coinciding with the construction of the interstate highway system. Figure 2-10 shows the correlation of age of construction with damage observed. From this figure it is evident that bridges built between 1910 and 1940 suffered more damage.

Fig. 2-9 – Age of the total bridge inventory affected by the 2001 Nisqually, Washington State, Earthquake

Figure 2-11 shows the spectral accelerations in percentage of g ($T = 0.3$ s) for each bridge site for the total bridge inventory in the area and Fig. 2-12 correlates the damage observed with the spectral acceleration. Correlation of spectral acceleration and bridge damage is evident from these figures.

Fig. 2-10 – Percentage of damaged bridges by decade of construction for the 2001 Nisqually, Washington State, Earthquake

Fig. 2-11 – Spectral acceleration (%g at $T = 0.3$ s) at the bridge site for the total bridge inventory affected by the 2001 Nisqually, Washington State, Earthquake

Fig. 2-12 – Percentage of bridge damaged for different values of spectral acceleration (%g at $T = 0.3$ s) at the bridge site during the 2001 Nisqually, Washington State, Earthquake

The bridge behavior observed during the Nisqually Earthquake is relevant because it may be similar to what would be expected in southwestern Indiana for a large earthquake in the New Madrid zone. The accelerations expected are of the same order of magnitude of those recorded during Nisqually Earthquake. The bridge inventory has similar characteristic with respect to age and type of bridges. Awareness of the seismic risk in the Pacific Northwest evolved about the same time as that for awareness of the seismic risk in the Central U.S. Earthquake resistant design of new bridges started about the same time in Central U.S. and Pacific NW.

2.2.2 Bridge seismic design criteria evolution

The early days

In 1931 the American Association of State Highway Officials (AASHO) published the first edition of the *Standard Specifications for Highway Bridges* [AASHO, 1931]. These specifications were – and still are – for ordinary bridges having span lengths less than 500 feet. No reference to considering earthquake effects on bridges was made in the *Standard Specifications* until the fifth edition (1949) in which a statement requiring that earthquake effects should be considered was included without guidance on how to account for them. The same statement was kept in the sixth (1953) and seventh (1957) editions.

The eighth edition (1961) of the *Standard Specifications* [AASHO, 1961] was the first to specify an earthquake loading for design (EQ) to be applied statically in any horizontal direction as part of Group VII load combination which included along with earthquake effects: dead loads, earth pressure, buoyancy, and stream flow effects. This load combination was to be used in the working stress design (WSD) procedure with a 1/3 increase in allowable stress permitted for occasional loads. The earthquake load was defined using a seismic coefficient (C) that multiplied the dead load (D):

$$EQ = C \cdot D \quad (2-1)$$

Values of C were 0.02 for structures supported on spread footings where the soil bearing capacity was rated to be greater than 4 t/ft², 0.04 for structures supported on

spread footings where the soil bearing capacity was rated to be less than 4 t/ft², and 0.06 for structures founded on piles. No seismic zone factors or seismic zoning map was provided, leaving to the State Bridge Commissioner or the State Highway Department the decision of considering the State, or regions within the State, as seismic. These seismic provisions were an extension of the lateral force requirements for buildings developed prior to 1961 by the Structural Engineers Association of California (SEAOC). The seismic loading provisions of the eighth (1961) edition of the *Standard Specifications* were kept, without modification, in the ninth (1965), tenth (1969) and eleventh (1973) editions.

The 1975 AASHTO Interim

As a result of the 1971 San Fernando, California, Earthquake, during which many highway bridges were severely damaged and some even collapsed – as described previously – the California Department of Transportation (Caltrans) issued, in 1973, a new seismic design procedure for bridges which formed the basis of the *1975 AASHTO Interim Specifications for Highway Bridges* [AASHTO, 1975]. This document specified an equivalent static lateral force:

$$EQ = C \cdot F \cdot W \quad (2-2)$$

The procedure was restricted to bridges having supporting members of approximately equal lateral stiffness. The equivalent static lateral force was to be applied in any horizontal direction as part of the same Group VII load combination used in the eighth (1961) *Standard Specifications* edition in a working stress design procedure permitting a 1/3 increase in allowable stress. In Eq. (2-2), W represents dead load, F is a framing factor assigned the values of 1.0 for single columns and 0.8 for moment resisting frames with the horizontal force acting along the frame, and C was a combined response coefficient expressed by:

$$C = \frac{A \cdot R \cdot S}{Z} \quad (2-3)$$

in which A denotes the maximum expected peak ground acceleration (PGA) associated with each seismic risk zone of the map of the United States shown in Fig. 2-13, R is a normalized ($\text{PGA} = 1g$) acceleration response spectral value for a rock site, S is a soil amplification factor, and Z is a force reduction factor depending upon structural-component type which accounts for the allowance of inelastic deformations.

The numerical values specified for A were $0.09g$, $0.22g$, and $0.50g$ in seismic zones numbered 1, 2, and 3 respectively in the map of Fig. 2-13.

Fig. 2-13 – Seismic Risk Map contained in the
1975 AASHTO Interim Specifications for Highway Bridges

Numerical values for R , S , and Z were not provided in the *1975 AASHTO Interim Specifications for Highway Bridges*; instead four plots of C as a function of period T were given for different values of A . Each of these plots represents a different depth range of alluvium to rock-like material. The ranges were 0-10 ft, 11-80 ft, 81-150 ft, and greater than 150 ft. Figure 2-14 shows the response coefficient C values as a function of period T for 81 to 150 ft depth of alluvium. Minimum values for C were set at 0.10 for values of A greater than or equal to $0.3g$, and 0.06 for values of A less than $0.3g$.

Fig. 2-14 – Response coefficient C for 81-150 ft depth of alluvium contained in the 1975 AASHTO Interim Specifications for Highway Bridges

Period T was to be evaluated using the relationship

$$T = 0.32 \sqrt{\frac{W}{P}} \quad (2-4)$$

in which P was equal to the total uniform static load required to cause a 1-inch lateral deflection of the whole structure. For complex or irregular structures the 1975 Interim Specifications required use of the modal response spectrum analysis method to obtain design loads. For structures adjacent to active faults, sites with unusual geologic conditions, unusual structures, and structures having a fundamental period greater than 3 seconds it required that the design should be made using current seismicity, soil response, and dynamic analysis techniques. The 1975 AASHTO Interim Specifications for Highway Bridges also included a procedure for design of restraining features to limit the displacement of the superstructure including hinges, ties, and shear blocks.

The seismic design criteria contained in the *1975 AASHTO Interim Specifications* were kept in the twelfth (1977), thirteenth (1983), and fourteenth (1989) editions of *AASHTO's Standard Specifications*. The only modification was to allow the use of load-factor design (LFD) or ultimate strength design (USD). For WSD the same Group VII load combination was specified with a 1/3 allowable stress increase – see Eq. (2-5) –. When using LFD, the Group VII combination was as shown in Eq. (2-5).

$$\begin{aligned}\text{Group VII (WSD)} &= D + E + B + SF + EQ \\ \text{Group VII (LFD)} &= \gamma [\beta_D \cdot D + \beta_E \cdot E + B + SF + EQ]\end{aligned}\tag{2-5}$$

where D denotes dead load; E earth pressure; B buoyancy; and SF stream flow; $\gamma = 1.3$; $\beta_D = 0.75$ when checking columns for minimum axial load and maximum moment or eccentricity, $\beta_D = 1.0$ when checking columns for maximum axial load and minimum moment, and $\beta_D = 1.0$ for flexure and tension members; and $\beta_E = 1.3$ for lateral earth pressure and $\beta_E = 0.5$ for positive moment in rigid frames.

Although the introduction of these requirements for computing seismic forces and their use in design was an improvement with respect to previous practice, no corresponding changes were introduced in the detailing requirements for the structural materials within the *Standard Specifications*. Seen from a more recent perspective, the lack of requirements to introduce toughness to the structural elements as was being required at the same time for the earthquake resistant design of buildings [ACI, 1977] would turn later to be one of the major sources of seismic vulnerability for transportation structures designed and built during this era.

The ATC 6 and Division I-A of AASHTO Standard Specifications

In the aftermath of the February 9, 1971, San Fernando, California, Earthquake the Structural Engineers Association of California (SEAOC) established the Applied Technology Council (ATC), an independent, nonprofit, tax-exempt corporation for the purpose of implementing current technological developments into active structural engineering practice. Their initial efforts were aimed at earthquake resistant design of buildings. In the late 1970's with the sponsorship of the National Science Foundation and

the National Bureau of Standards the Applied Technology Council published document ATC-3-06 [ATC, 1978] devoted to building earthquake resistant design. This document was the draft of what was to become current Model Code seismic requirements in the United States and many countries. In 1981 ATC, with the sponsorship of the Federal Highway Administration, Department of Transportation, published document *ATC-6 Seismic Design Guidelines for Bridges* [ATC, 1981]. These guidelines with minor revision made by the National Center for Earthquake Engineering Research (NCEER) in Buffalo, NY, under the sponsorship of the National Cooperative Highway Research Program (NCHRP), were adopted as the earthquake resistant design requirements for bridges in the fifteenth (1992) edition of *AASHTO Standard Specifications*. These requirements were kept without modification in the sixteenth (1996) and in the current seventeenth (2002) edition [AASHTO, 2002] as *Division I-A Seismic Design*.

Under the requirements of *Division I-A Seismic Design* of the current 17th Edition of the *Standard Specifications* the acceleration coefficient A for the bridge site must be obtained from the acceleration contour maps provided. The northern Midwest portion of the map of horizontal acceleration A (expressed as a percent of gravity) in rock with 90 percent probability of not being exceeded in 50 years – mean return period 475 years – is shown in Fig. 2-15. This map was prepared by the U.S. Geological Survey (USGS) for the 1988 edition of the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* [NEHRP, 1988]. Although the values of A in this map do not differ in the order of magnitude from those contained in the original *ATC-6* 1981 report for the same geographical regions, the 1988 map uses a finer resolution by assigning values of A in a county by county basis and presenting them in smaller acceleration increments.

Once the acceleration coefficient is obtained, the bridge structure must be classified as either “Essential” or “Other” in accordance with given guidelines. An importance classification ($IC=I$) shall be given to Essential bridges and ($IC=II$) shall be given to ordinary bridges. The classification of “Essential” must be given to all bridges located in zones where the value of A is greater than $0.29g$ and on the basis of social, survival, security, or defense requirements. Guidance is given in the commentary of *ATC-6* for

including bridges in either importance group. The bridge also must be assigned to a Seismic Performance Category (SPC) A, B, C or D as a function of the acceleration coefficient A as defined in Table 2-1.

Fig. 2-15 – Northern Midwest portion of the map of horizontal acceleration A (g) in rock with 90 percent probability of not being exceeded in 50 years (*Division I-A Seismic Design of the current 17th Edition of the Standard Specifications*)

Table 2-1 – Seismic Performance Categories (SPC) *Division I-A Seismic Design of the current 17th Edition of the Standard Specifications*

Acceleration Coefficient	Bridge Classification	
	Essential	Other
$A \leq 0.09g$	A	A
$0.09g < A \leq 0.19g$	B	B
$0.19g < A \leq 0.29g$	C	C
$0.29g < A$	D	C

The effects of site conditions on bridge response are determined from a Site Coefficient (S) based on soil profile types summarized in Fig. 2-16.

Fig. 2-16 – Soil Profile Types
(Division I-A Seismic Design of the current 17th Edition of the Standard Specifications)

No dynamic analysis is required in *Division I-A Seismic Design* of the current 17th Edition of the Standard Specifications for bridges having single spans, regardless of the value of the site acceleration coefficient A , and for all bridges in SPC A. All other bridges, regular or irregular, having two or more spans must be analyzed by at least one of two dynamic analysis procedures, namely, the single-mode spectral method (SMSM) or the multi-mode spectral method (MMSM). The SMSM is specified as minimum for

regular bridges in SPC B, C, and D; while the MMSM is specified as minimum for irregular bridges in these same categories. An "irregular" bridge is defined as one having abrupt or unusual changes in mass, stiffness, and/or geometry from abutment to abutment; a "regular" bridge is one not meeting the definition of an "irregular" bridge.

The seismic input in any horizontal direction to be used in each of these minimum dynamic analysis procedures is specified in terms of an elastic seismic response coefficient, C_s , as expressed for the single-mode spectral SMSM method for a bridge period T by:

$$C_s = \frac{1.2 \cdot A \cdot S}{T^{2/3}} \quad (2-6)$$

The value of C_s need not exceed $2.5 \cdot A$. For sites with soil profile types III and IV in areas where A is greater or equal to $0.3g$, C_s need not exceed $2.0 \cdot A$.

Fig. 2-17 – Seismic response coefficient C_s for different soil profiles normalized with respect to acceleration coefficient A (*Division I-A Seismic Design of the current 17th Edition of the Standard Specifications*)

For the multi-mode spectral method (MMSM) the same relationship for C_s is used and for each individual vibration mode “ m ” the mode period T_m must be used in Eq. (2-6)

as T . In this case a reduction of C_s is allowed for modes other than the fundamental and periods shorter than 0.3 sec, with the value of C_s tending to the value of A at a period equal to zero. For long periods greater than 4 sec a minimum value of C_s is required.

Deformations in the nonlinear range of response of the bridge elements are expected to occur when subjected to the earthquake design ground motion prescribed in the *Division I-A Seismic Design* of the current 17th Edition of the *Standard Specifications*. Response modification factors (R) are prescribed for the cases listed in Table 2-2.

Table 2-2 – Response Modification Factors (R) *Division I-A Seismic Design* of the current 17th Edition of the *Standard Specifications*

Substructure	R	Connections	R
Wall-Type Pier	2	Superstructure to abutment	0.8
Reinforced concrete pile bents		Expansion joints within a span of the superstructure	0.8
a. Vertical piles only	3		
b. One or more battered piles	2		
Single columns	3	Columns, piers, or pile bents to cap beam or superstructure	1.0
Steel or composite & steel concrete pile bents		Columns or piers to foundation	1.0
a. Vertical piles only	5		
b. One or more battered piles	3		
Multiple-column bent	5		

In a great departure from the earthquake resistant design of buildings in which the response modification factor is assigned for the structure as a whole by dividing the design forces by R before performing the analysis; in the *ATC-6* document the analysis, static or dynamic, is performed for the bridge without reducing the design forces by the response modification factor. Here it is used at the element level by dividing the design forces by R just when computing modified values (EQM) that replace values of (EQ) in Eq. (2-5). This form of use for the response modification factor in bridges has been kept up to *Division I-A Seismic Design* of the current 17th Edition of the *Standard Specifications*.

Other major departure from previous bridge design practice introduced by the *ACT-6* document was to assign differential detailing requirements depending on the Seismic Performance Categories (SPC). This leads to increased structural toughness from

category A to D, with category A corresponding to the *Standard Specifications* requirements without any seismic provisions beyond using seismic forces in analysis and design and no special detailing. The reinforced concrete requirements for SPC B to D were based on corresponding requirements of the ACI 318 Code [ACI, 1977] for buildings. This may be considered a landmark in improved expected performance of transportation structures subjected to strong earthquake ground motion.

Other significant improvement with respect to previous practice was the introduction of dimension of minimum support-width length to be used at expansion ends of girders in all bridges; regardless of seismic performance category and number of spans (see Fig. 2-18). Prescribed minimum support lengths depend on angle of skew of the support, distance between expansion joints, pier height, and seismic performance category.

FIGURE 3.10 Dimensions for Minimum Support Length Requirements

Fig. 2-18 – Minimum support length definition (*Division I-A Seismic Design* of the current 17th Edition of the *Standard Specifications*)

AASHTO LRFD

Reinforced concrete design departed from the working-stress design (WSD) philosophy in the mid 1950's and by the early 1970's had moved completely into ultimate strength design (USD) philosophy. Other materials lagged but by the mid 1980's

a push for probabilistic design methods coined a new term for USD as load and resistance factor design (LRFD) which was adopted especially by the steel industry and later by AASHTO. Between 1988 and 1993 with the sponsorship of the Federal Highway Administration under the National Cooperative Highway Research Program (NCHRP) developed and published in 1994 the first edition of *AASHTO LRFD Bridge Design Specifications*, reviewed later for a second edition in 1998 [AASHTO, 1998], and recently in a third edition in 2004 [AASHTO, 2004].

The AASHTO LRFD Bridge Design Specifications, First (1994), Second (1998), and Third (2004) Editions, requires that each bridge component and connection satisfy all limit states in accordance with the relation

$$\eta \sum (\gamma_i \cdot Q_i) \leq \phi \cdot R_n \quad (2-7)$$

in which η is a factor related to a ductility factor η_D , a redundancy factor η_R , and an operational importance factor η_i in accordance with $\eta = \eta_D \cdot \eta_R \cdot \eta_i$; γ_i is a statistically-based load factor applied to force effect Q_i ; and ϕ is a statistically-based resistance factor applied to the nominal resistance R_n . The numerical values to be used for these factors can be found in the LRFD Specifications (AASHTO LRFD, 1994, 1998, and 2004).

The value of Q_i for the value of i representing an extreme seismic event, designated EQ, is found using the same procedure described above for *Standard Specifications*, Fifteenth (1992), Sixteenth (1996), and seventeenth (2002) Editions.

An additional bridge classification, "Critical," has been added to the LRFD Specifications; and the number of substructure response modification factors R , have been increased and made different in all three classifications, "Critical," "Essential," and "Other" as indicated in Table 2-3. The response modification factors for connections remained the same in the *LRFD Specifications* than in the *Standard Specifications*.

Although, at first glance it may look that the impact of adoption of *AASHTO LRFD Specification* to replace the *AASHTO Standard Specifications* with respect to seismic design is minor because they are based on the same requirements, this is not true for bridges in the "Critical" and "Essential" categories where seismic design forces described

by *EQM* (*EQ/R*) increase by a factors ranging from 1.33 to 3.33 for “Critical” bridges and from 1.33 to 1.5 for “Essential” bridges.

Table 2-3 – Response Modification Factors (*R*) in *AASHTO LRFD Specifications*

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers-larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
• Vertical piles only	1.5	2.0	3.0
• One or more battered piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite & steel concrete pile bents			
• Vertical piles only	1.5	3.5	5.0
• One or more battered piles	1.5	2.0	3.0
Multiple-column bent	1.5	3.5	5.0

The presented material describes current *AASHTO Specifications* both *Standard* and *LRFD*. Notwithstanding, several important changes on seismic bridge specifications are being discussed in the corresponding drafting committees and it is important to be aware of their implications, if approved. These changes come basically from three sources:

- the first one corresponds to changes in the California Department of Transportation (Caltrans) seismic design criteria,
- the second one from changes in the way to define the design ground motion both in the description of the movement at rock by the U.S. Geological Survey (USGS) that is discussed in Chapter 3, and
- the third in the way to assess the amplification caused by the soil profile by the Building Seismic Safety Council (BSSC) as part of the National Earthquake Hazard Reduction Program (NEHRP) for incorporation in the NEHRP Recommended Provision for Seismic Regulations for New Buildings that is discussed in 2.3.1.

Some of these new trends have already been adopted as regulations in the corresponding earthquake resistant design requirements for new buildings. A brief description of these trends follows.

Caltrans Seismic Design Criteria

In the aftermath of the Loma Prieta Earthquake of 1989, the California Department of Transportation (Caltrans) decided to sponsor a project to conduct a critical review and to recommend changes where needed in the seismic design part of the Caltrans Bridge Design Specifications (BDS) also known as the California Seismic Design Criteria (SDC). The result of this project was presented in document ATC-32 [ATC, 1996] which provided a number of recommended improvements to seismic bridge design practice in California. The project focused on issues related to seismic loading, structural response analysis, and component design. Special attention was given to design issues related to reinforced concrete components, steel components, foundations, and conventional bearings. In addition, the specifications were revised to give a bigger role to the evaluation of displacement and to use it as a tool to define performance criteria. Although a new definition of the design ground motion was contained, the production of new seismic risk maps was considered then outside the scope of the project.

The ATC-32 was, as the title indicated, a recommendation. It took some time for Caltrans to update the *Caltrans SDC*. In July 1999 Version 1.1 of the *Caltrans SDC* was published [Caltrans, 1999], with Version 1.2 in December of 2001 [Caltrans, 2001], and Version 1.3 in February 2004 [Caltrans, 2004].

The *Caltrans SDC* adopts a performance-based approach specifying minimum levels of structural system performance, component performance, analysis, and design practices for ordinary standard bridges. For bridges with non-standard features or operational requirements above and beyond the ordinary standard bridge a greater degree of attention than specified by the SDC is required. Many of the methodologies contained in the SDC evolved from the seismic retrofit program and some of the procedures are major departures from previous practice while others are slight modifications to current practice. The most significant change in design philosophy for new bridges is a shift from

a force-based assessment of seismic demand to a displacement-based assessment of demand and capacity. The displacement approach is based on comparing the elastic displacement demand to the inelastic displacement capacity of the primary structural components while insuring a minimum level of inelastic capacity at all potential plastic hinge locations.

The SDC document [Caltrans, 2004] warns about its applicability in places different from California: *“This document is intended for use on bridges designed by and for the California Department of Transportation. It reflects the current state of practice at Caltrans. This document contains references specific and unique to Caltrans and may not be applicable to other parties either institutional or private.”* One important aspect that makes its direct applicability elsewhere difficult is the way the earthquake design ground motion is defined. California has unique tectonic characteristics with a relatively thin earth’s crust producing predominantly shallow earthquakes. The fault mapping of California is comprehensive; leading to definition of design ground motion supported on relatively well-known faults that lend themselves to establish earthquake mean recurrence periods with somewhat less uncertainty than in other places in the U.S. (this will be discussed later in Chapter 3 with respect to the implication for the State of Indiana). This means that the design ground motion proposed by *Caltrans SDC* is particular for the tectonic setting and the types of soil profiles common in California. Other important aspect is that the *Caltrans SDC* is focused on concrete bridges. The *Caltrans SDC* specifies target ductility demands, defined as the ratio of the estimated global frame displacement demand to the yield displacement of the subsystem from its initial position to the formation of plastic hinge, for different structural components. For instance, for multi-column bents a value of 5 is prescribed while for wall piers in the strong direction a value of unity ($= 1$) is given. The structure must be designed to resist the internal forces generated when the structure reaches its Collapse Limit State. The Collapse Limit State is defined as the condition when a sufficient number of plastic hinges have formed within the structure to create a local or global collapse mechanism.

The local displacement capacity of a member is based on its rotation capacity (see Fig. 2-19 for cantilever column and Fig. 2-20 framed columns), which in turn is based on

its curvature capacity. The curvature capacity must be determined from a moment-curvature ($M-\phi$) analysis. Each ductile member shall have a minimum local displacement ductility capacity of 3 to ensure dependable rotational capacity in the plastic hinge regions regardless of the displacement demand imparted to that member.

Fig. 2-19 – Local displacement capacity – cantilever column with fixed base
(*Caltrans Seismic Design Criteria* [Caltrans, 2004])

Fig. 2-20 – Local displacement capacity – framed column assumed as fixed-fixed
(*Caltrans Seismic Design Criteria* [Caltrans, 2004])

Caltrans SDC defines how the expected material properties can be obtained, how to compute the plastic moment capacity for ductile concrete members, how to perform the moment curvature ($M-\phi$) analysis, and how to obtain the shear capacity of the

components. The designer is required to check that the nominal capacity of the superstructure longitudinally and of the bent cap transversely are sufficient to ensure that the columns respond in the nonlinear range prior to the superstructure or bent cap reaching its expected nominal strength. *Caltrans SDC* requires assessing the force and deformation demands and capacities on the structural system, and its individual components, using analysis. The document prescribes for ordinary standard bridges the use of equivalent static analysis and linear elastic dynamic analysis for estimating the displacement demands, and inelastic static analysis (push-over) for establishing the displacement capacities. The *Caltrans SDC* document gives guidance with respect to the use of appropriate cracked sections for each type of analysis.

The geotechnical engineer is required to provide the following information relative to the bridge site: seismicity, fault distance, earthquake magnitude, peak rock acceleration, soil profile type, liquefaction potential, and foundation stiffness or the soil parameters necessary for determining the force deformation characteristics of the foundation. The document contains standard acceleration response spectra for preliminary design, but in general requires a site specific spectrum for design. The standard acceleration and displacement response spectra provided are function of soil profile type and design earthquake magnitude. Magnitude ranges from 6.5 to 8, and envisioned peak ground accelerations in rock as high as $0.7g$ are presented. Acceleration amplification caused by the soil profile follows latest BSSC and NEHRP recommendations (see 2.3.1). The foundation subsystem response must be evaluated based on the quality of the surrounding soil. Soils are classified as competent, poor, or marginal.

In the *Caltrans SDC* the bridge designer is required to maintain the ratio of effective stiffness between any two bents within a frame or between any two columns within a bent within prescribed limits. Hinge seat width are prescribed to accommodate the anticipated thermal movement, prestress shortening, creep, shrinkage, and the relative longitudinal earthquake displacement demand between adjacent frames. As a rule adequate seat width must be provided to prevent unseating as a primary requirement. Hinge restrainers are considered secondary members to prevent unseating. Moment resisting connections between the superstructure and the column must be designed to

transmit the maximum forces produced when the column has reached its flexural and shear overstrength capacity. *Caltrans SDC* considers bearings as sacrificial elements for ordinary standard bridges. Typically bearings must be designed and detailed for service loads. However, bearings must be checked to insure their capacity and mode of failure are consistent with the assumptions made in the seismic analysis. The designer must detail bearings so they can be easily inspected for damage and replaced or repaired after an earthquake. Columns flares require special treatment to insure proper behavior of the column. At the foundation, the size and number of piles and the pile group layout must be designed to resist service level moments, shears, and axial loads and the moment demand induced by the column plastic hinging mechanism. The linear elastic demand analysis model must include an effective abutment stiffness that accounts for expansion gaps, and incorporates realistic values of the embankment fill response. Seat type abutments must be designed to resist elastically all transverse service load and moderate earthquake demands.

Draft AASHTO LRFD Update

The *Caltrans SDC* imposed a trend that was picked up by the National Cooperative Highway Research Program (NCHRP) for incorporation in an update of the *AASHTO LRFD Specifications*. A research project sponsored by AASHTO in cooperation with the Federal Highway Administration (FHWA) produced a draft update of the AASHTO LRFD Specifications that is summarized and described in NCHRP Report 472 [NCHRP, 2002] and presented in final form in four reports published by the Applied Technology Council: “Recommended LRFD guidelines for the seismic design of highway bridges – Part I: Specifications” [ATC/MCEER, 2003a], “Recommended LRFD guidelines for the seismic design of highway bridges – Part II: Commentary and Appendices” [ATC/MCEER, 2003b], “Liquefaction Study Report for Recommended LRFD guidelines for the seismic design of highway bridges” [ATC/MCEER, 2003c], and “Design Examples for Recommended LRFD guidelines for the seismic design of highway bridges” [ATC/MCEER, 2003d]. The research was performed by a joint venture of the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake

Engineering Research (MCEER), Buffalo, NY. The project purpose was to develop new specifications for the seismic design of bridges, considering all aspects of the design process. These aspects include the following: (1) design philosophy and performance criteria, (2) seismic loads and site effects, (3) analysis and modeling, and (4) design requirements. The new specification should be nationally applicable with provisions for all seismic zones. The emphasis was on design of new bridges rather than on retrofit of existing ones. The *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* contains a number of new concepts and additions, as well as some major modifications to existing provisions. These are summarized in the following paragraphs.

The definition of the design earthquake has been moving away from the 10% probability of exceedance in a 50-year life span of the structure towards a definition of a maximum considered earthquake with a lower (3%) probability of exceedance and sometimes a longer life span (75 years). On the other hand, the use of effective peak ground acceleration as the mapped parameter to define the design ground motion has been substituted in the specifications for design of new buildings for a direct definition of the design response spectrum including the site effects. The USGS has developed maps for these new definitions as will be explained in Chapter 3 of this report. The *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* uses these new definitions and maps.

The proposed provisions provide performance objectives and damage states for two design earthquakes with explicit design checks to ensure that the performance objectives are met (see Table 2-4).

In Table 2-4 the upper-level event, termed the “rare” or Maximum Considered Earthquake (MCE), describes ground motion that, for most locations, is defined probabilistically and has a probability of exceedance of 3% in 75 years (approximately 2500-year mean return period). The frequent or expected earthquake has a probability of exceedance of 50% in 75 years (approximately 100-year mean return period). For the service level: “Immediate” means full access to normal traffic and the bridge must be available for traffic after an inspection, and “Significant Disruption” means limited access (reduced lanes, light emergency traffic) may be possible after shoring but the

bridge may need to be replaced. Detailed geometric constraints on service level are presented in the draft. For the damage levels: “None” means evidence of movement may be present but no notable damage; “Minimal” means some visible signs of damage produced by minor nonlinear response (narrow flexural cracking), no permanent deformations, and repair can be performed under non-emergency conditions, “Significant” means no collapse, but permanent offsets, cracking, reinforcement yielding, and major spalling of concrete may require closure for repairs. Partial or complete replacement of columns may be required in some cases.

Table 2-4 – Design Earthquakes and Seismic Performance Objectives in the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)*

Probability of exceedance for design earthquake ground motion		Performance level	
		Life Safety	Operational
Rare earthquake (MCE) 3% in 75 years	Service	Significant Disruption	Immediate
	Damage	Significant Disruption	Minimal
Frequent or expected earthquake 50% in 75 years	Service	Immediate	Immediate
	Damage	Minimal	Minimal to none

The design ground motion in *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* is described through a design response spectrum that can be obtained from the general procedure or from a site-specific study. The site-specific study is required if the bridge is considered a major or very important structure, when the site is located within 10 km of an active fault, or when the site has a Type F soil profile (see 2.3.1) with peats or highly organic soils, high plasticity clays, or very thick deposits of soft to medium stiff clays.

The general procedure defines response spectra for the rare MCE earthquake and for the frequent earthquake based on maps produced (or being produced in the case of the frequent earthquake) by the USGS. The acceleration spectrum for 5% of critical damping is obtained using a two point procedure. The spectral design acceleration at 0.2-second

period, S_{DS} , and at 1-second period, S_{D1} , are obtained from Equations (2-8) and (2-9) respectively.

$$S_{DS} = F_a \cdot S_S \quad (2-8)$$

$$S_{D1} = F_v \cdot S_1 \quad (2-9)$$

where S_S and S_1 are the 0.2-second and 1-second period spectral accelerations on rock (Class B site, see 2.3.1) from ground motion maps and F_a and F_v are site coefficients associated with the site class and mapped acceleration value. The maps for the Maximum Considered Earthquake contained in the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* are the same maps produced by the USGS for BSSC to be included in the 1997 NEHRP recommendations for new buildings [NEHRP, 1997]. These maps are presented and discussed in Chapter 3. For the frequent earthquake there were no maps available when the draft was produced and the draft suggests obtaining them by interpolation from the hazard curves produced by the USGS compatible with the maps for the MCE. The design spectrum is obtained from the values given by Equations (2-8) and (2-9). Two control periods are given: T_s and T_0 defined in Equations (2-10) and (2-11) respectively. Figure 2-21 shows the design spectrum construction using the two-point method.

$$T_s = \frac{S_{D1}}{S_{DS}} \quad (2-10)$$

$$T_0 = 0.2 \cdot T_s \quad (2-11)$$

For periods less than or equal to T_0 , the design response spectral acceleration is defined by Eq. (2-12). Note that as T tends to $T = 0$ seconds, the resulting value of S_a is equal to the peak ground acceleration (PGA).

$$S_a = 0.60 \cdot \frac{S_{DS}}{T_0} \cdot T + 0.4 \cdot S_{DS} \quad (2-12)$$

For periods greater than or equal to T_0 and less than or equal to T_s , the design response spectral acceleration, S_a , is defined by Eq. (2-13).

$$S_a = S_{DS} \quad (2-13)$$

For periods greater than T_s the design response spectral acceleration, S_a , is defined by Eq. (2-14).

$$S_a = \frac{S_{D1}}{T} \quad (2-14)$$

Fig. 2-21 – Design Response Spectrum – Probabilistic general procedure
(Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49))

When a site-specific response spectrum is used, a probabilistically based spectrum must be obtained using a characterization of the seismic sources and ground motion attenuation that includes current scientific interpretations, including uncertainties in seismic source and ground motion models and parameter values. If the spectrum obtained this way exceeds the limits shown in Fig. 2-22, a deterministic spectrum may be used.

The deterministic design response spectrum may be used in regions having known active faults if the deterministic spectrum ordinates are less than those of the probabilistic spectrum. The deterministic spectrum must be the median-plus-standard-deviation spectrum calculated for characteristic maximum magnitude earthquakes on known active faults (the corresponding building requirements require 150% instead of the 184% for bridges), but must not be less than the spectrum shown in Fig. 2-22. If there is more than one active fault in the site region, the deterministic spectrum must be computed as the

envelope of spectra for the different faults. Alternatively a deterministic spectrum may be computed for each fault, and each spectrum, or the spectrum that govern bridge response, may be used for the analysis of the bridge. When the design response spectrum is determined from a site-specific study, the spectrum ordinates must not be less than 2/3 of the ordinates of the probabilistic spectrum obtained following the general procedure. The draft permits also the use of acceleration time histories instead of a design response spectrum and provides guidance on how to define them. Design for vertical acceleration effects are prescribed in detail for bridges located less than 50 km from an active fault.

Fig. 2-22 – Minimum Deterministic Design Response Spectrum
(Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49))

The *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* defines a Seismic Hazard Level (SHL) classification into four groups (I to IV). The bridge must be assigned in the larger SHL obtained from Table 2-5 or 2-6.

Table 2-5 – Seismic Hazard Level based on $F_v S_1$
Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)

Seismic Hazard Level	Value of $F_v S_1$
I	$F_v S_1 \leq 0.15g$
II	$0.15g < F_v S_1 \leq 0.25g$
III	$0.25g < F_v S_1 \leq 0.40g$
IV	$0.40g < F_v S_1$

Table 2-6 – Seismic Hazard Level based on $F_a S_S$
Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)

Seismic Hazard Level	Value of $F_a S_S$
I	$F_a S_S \leq 0.15g$
II	$0.15g < F_a S_S \leq 0.35g$
III	$0.35g < F_a S_S \leq 0.60g$
IV	$0.60g < F_a S_S$

The Seismic Design Level is used, in turn, to define the Seismic Design and Analysis Procedure (SDAP) and the Seismic Detailing Requirements (SDR). Table 2-7 defines the Seismic Design and Analysis Procedure.

Table 2-7 – Seismic Design and Analysis Procedure (SDAP)
Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)

Seismic Hazard Level	Life Safety	Operational
I	A1	A2
II	A2	C/D/E
III	B/C/D/E	C/D/E
IV	C/D/E	C/D/E

Brief description of the different Seismic Design and Analysis Procedures (SDAP):

- SDAP A — Observance of minimum seat widths, checking of connection design forces for bearings, and use of minimum shear reinforcement in concrete piles are the only requirements. The horizontal connection design force in the restrained direction must be 10% of the vertical reaction for SDAP A1 and 25% for SDAP A2. No rigorous analysis is required instead these default values are used as design forces.
- SDAP B — Does not require a seismic demand analysis. Capacity design principles and minimum design detailing are required. There are certain restrictions to use SDAP B associated with span length (maximum 80 feet), skewness of the bridge (maximum 30°), bent stiffness, and element cross-section dimension and reinforcement, and other.
- SDAP C — Requires the use of the capacity design spectrum approach. This procedure applies only to bridges that behave and can be modeled as a single-

degree-of-freedom system. The procedure is also restricted to bridges with a very regular configuration, bridges not having more than six spans, spans not greater than 60 feet, maximum skewness of 30°, and other.

- SDAP D — Requires the use of the elastic response spectrum method. Cracked sections are required. The procedure is divided into a single-mode equivalent lateral load procedure, and a multimode dynamic analysis procedure.
- SDAP E — Consist in an elastic response spectrum analysis used for design plus a displacement capacity verification performed afterwards.

Response modification factor, R_B , to be used in SDAP D and E are presented in Table 2-8 for the substructure. Response modification factor for connections are all $R = 0.8$ but are not intended for cases where the design of the connection is made using capacity design principles.

Table 2-8 – Base Response Modification Factors, R_B , for the substructure
Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)

Substructure Element	Performance Objective			
	Life Safety		Operational	
	SDAP D	SDAP E	SDAP D	SDAP E
Wall pier – larger dimension	2	3	1	1.5
Columns – Single and multiple Ductile detailing	4	6	1.5	2.5
Pile bents and drilled shafts – vertical piles – above ground	4	6	1.5	2.5
Pile bents and drilled shafts – vertical piles – 2 diameters below ground level – no owner's approval required	1	1.5	1	1
Pile bents and drilled shafts – vertical piles – in ground – owner's approval required	N/A	2.5	N/A	1.5
Pile bents with batter piles	N/A	2	N/A	1.5
Seismically isolated structures	1.5	1.5	1	1.5
Steel braced frame – ductile	3	4.5	1	1.5
Steel braced frame – Nominally ductile	1.5	2	1	1
All elements for frequent or expected earthquake	1.3	1.3	0.9	0.9

The value of R must be obtained from Eq. (2-15)

$$R = 1 + (R_B - 1) \cdot \frac{T}{T^*} \leq R_B \quad (2-15)$$

where $T^* = 1.25 T_s$ and T_s is defined in Eq. (2-10).

Table 2-9 defines the Seismic Detailing Requirements (SDR) for each Seismic Hazard Level (SHL) and Table 2-10 defines the component detailing requirements for each SDR.

Table 2-9 – Seismic Detailing Requirements (SDR)
Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)

Seismic Hazard Level	Life Safety	Operational
I	1	2
II	2	3
III	3	4
IV	5	6

Minimum seat width must not be less than 1.5 times the displacement of the superstructure at the seat including P-Δ effects, Δ_m , obtained using Eq. (2-16), but can not be less than the value of N obtained from Eq. (2-17):

$$\Delta_m = R_d \cdot \Delta$$

$$R_d = \begin{cases} \left(1 - \frac{1}{R}\right) \cdot \frac{T^*}{T} + \frac{1}{R} & \text{for } T < T^* \\ 1 & \text{for } T \geq T^* \end{cases} \quad (2-16)$$

$$N = \left[0.1 + 0.0017L + 0.007H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(2\frac{B}{L}\right)^2} \right] \cdot \frac{1 + 1.25F_v S_1}{\cos \alpha} \geq 1.25 \cdot \Delta_m \quad (2-17)$$

where,

Δ = displacement demand from the seismic analysis

Δ_m = nonlinear displacement demand

R_d = displacement amplification factor

L = distance between joints in m

H = height of the tallest pier between the joints in m
 B = width of the superstructure in m
 α = skew angle

In Eq. (2-17) the ratio B/L need not be taken greater than 3/8.

The proposed LRFD design provisions contain an incentive from a design and construction perspective for performing a more sophisticated “pushover analysis” [NCHRP, 2002]. The response modification factor (R) increases approximately 50 percent when a pushover analysis is performed, primarily because the analysis results will provide a greater understanding of the demands on the seismic resisting elements. The analysis results are assessed using additional plastic rotation limits on the deformation of the substructure elements to ensure adequate performance.

The proposed LRFD provisions provide a mechanism to permit the use of some seismic resisting systems and elements — termed earthquake resisting systems (ERS) and earthquake resisting elements (ERE) — that are not permitted in current AASHTO provisions. Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. The classification of ERS and ERE into three categories: permissible, permissible with owner’s approval, and not recommended, is done to trigger due consideration of seismic performance that leads to the most desirable outcome.

The state of the art in earthquake resistant bridge design has been presented. The more recent changes in mandatory and proposed specifications derive from behavior experiences during recent earthquakes; from the intensive seismic retrofit programs in some states, specially in the West Coast; and from recent research in seismic behavior of bridges.

Table 2-10 – Component Detailing Requirements for SDR's — *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)*

Component		SDR 1	SDR 2	SDR 3	SDR 4	SDR 5	SDR 6
Seat width		Use Eq. (2-17)					
Bearing	Conventional	0.10 · DL	0.25 · DL	Capacity design procedure or elastic forces with $R = 0.8$			
	Isolation	Detailed and tested for 1.1 times 5% in 75-years forces and displacements					
Column	Flexure	Non-seismic requirements		SDAP B and C – non-seismic or minimum steel or P-Δ or 50% in 75-years forces for C. SDAP D and E – Moment demand divided by R or minimum steel or P-Δ			
	Shear	Non-seismic requirements	Minimum shear reinforcement	From capacity design procedures or elastic forces with $R = 0.67$			
Connection of column to superstructure, bent beam, footing/pile cap		N/A except bearing above	N/A except bearing above	Design forces from capacity design using over-strength ratios of 1.5 and 1.4 for concrete and steel respectively			
Soil and pile aspects of foundation design	Concrete	N/A	Piles top 3D minimum reinforcement	Design forces from capacity design using over-strength factor of 1.0	Design forces from capacity design and overstrength factor of 1.5 and 1.4 for concrete and steel respectively. Minimum shear reinforcement apply		
	Steel	N/A	N/A				
Pile bents	Concrete	N/A	Min. shear reinf. down to 10D below surface	Design forces from capacity design using over strength ratio of 1.5 and 1.4 for concrete and steel respectively. Minimum shear reinforcement as per SDR 2.			
	Steel	N/A	N/A				
Abutments		N/A	N/A	SDAP B and C – non-seismic req. SDAP D and E – Detailed req.	Detailed requirements		
Liquefaction		Only if requested by owner		If $M_w < 6$ not required. If > 6 full procedure	Full procedure		
ESR/ERE		N/A	N/A	Use procedure	System requiring owner’s approval not permitted		
Approach/settlement slab		N/A	N/A	N/A	Encouraged but not mandated	N/A	Required

Bridge retrofit

The leader in seismic retrofit programs has been the State of California. There are more than 12,000 bridges in the California State Highway system, plus an additional 12,000 city and county bridges. Since the 1971 San Fernando earthquake struck the Los Angeles area Caltrans has been engaged in an ongoing bridge retrofit program. Initially, Caltrans Seismic Retrofit program consisted of restraining sections of 1,262 bridges with steel cables. The Seismic Retrofit program now involves strengthening the columns of existing bridges by encircling certain columns with a steel casing or, in a few instances, an advanced woven fiber casing. In addition to the column casing, some of the bridge footings are made bigger and given more support by placing additional pilings in the ground or by using steel tie-down rods to better anchor the footings to the ground. In a few projects bridge abutments are made larger and the existing restrainer units are made stronger because encasing the columns make them stiffer and can change the way forces are transmitted within the bridge. Many Seismic Retrofits involve "hinge seat extensions" which enlarge the size of the hinges that connect sections of bridge decks and helps prevent them from separating during severe ground movement.

The Seismic Retrofit program was divided into Phase 1 and Phase 2. Phase 1 included 1,039 bridges identified for strengthening after the Loma Prieta quake. Phase 2 identified an additional 1,364 bridges for strengthening following the January 1994 Northridge earthquake. Currently the state's bridge earthquake strengthening program will involve approximately 2,200 additional structures, including the state's toll bridges. In addition a total of 1,114 city and county-owned bridges have been identified as candidates for seismic strengthening.

Washington State has carried similar programs. Between 1999 and 2001 more than 350 bridges were retrofitted statewide. After the 2001 Nisqually, Washington State, Earthquake the scope of the retrofit program was expanded.

Since 1983 the Federal Highway Administration has published three documents on seismic retrofit of highway bridges [FHWA, 1983, 1987, and 1995]. The latest, 1995 document, is compatible with current *Division I-A of the AASHTO Standard*

Specifications [AASHTO, 2002]. This document covers procedures for preliminary screening of the bridges, detailed evaluation, and design of retrofit measures.

Fig. 2-23 – Caltrans Seismic Retrofit Program Poster

The preliminary screening – Chapter 2 – emphasizes the need of developing an inventory of bridge seismic information accompanied by a seismic rating system, and gives guidance for establishing them. Using the vulnerability rating a priority index of retrofit is computed.

The detailed evaluation guidelines – Chapter 3 – define the different evaluation methods starting with the analysis procedures: analysis procedures for capacity/demand method, and analysis procedure for lateral strength method. Then, gives detailed requirements for performing a capacity/demand evaluation of the bridge, and establishes

the principles for applying the lateral strength method. Using the results of the two procedures an overall assessment of the bridge components is described including assessment of: bearings and expansion joints; columns, walls, and footings; abutments; and the possibility of liquefaction. Potential retrofit measures are discussed for each of the assessed components.

The rest of the document [FHWA, 1995] is devoted to detailed retrofit measures for the assessed components including: Chapter 4 – discusses seismic retrofit strategies including conventional and innovative earthquake protective systems; Chapter 5 – deals with the retrofit of bearings, seats, and expansion joints; Chapter 6 – covers retrofit of columns, cap beams, and joints; Chapter 7 – discusses the retrofit measures for foundations including footings and abutments; Chapter 8 – covers retrofit measures for bridges on hazardous sites including bridges across or near active faults and bridges on liquefiable soils discussing potential remedial solutions for these cases ; and Chapter 9 – discusses protective measures using seismic isolation. The appendices cover: (A) detailed indication for obtaining capacity/demand ratios for bridge components, (B) the assessment of members strength and deformation capacity, and (C, D, and E) worked examples for two bridges and the use of cable restrainers.

Although this document was developed a few years ago, it covers all the basic procedures. This document [FHWA, 1995] combined with some of the detailed concepts covered by the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] makes an excellent seismic bridge retrofit guide that is totally up-to-date.

2.3 Geotechnical issues

In the previous section when discussing the evolution of the bridge seismic design criteria the parameters associated with geotechnical issues were presented without explaining and discussing their background. The following sections present the corresponding background and relevant information that that will be needed for the assessment for geotechnical issues of the earthquake emergency routes of Indiana as presented in Chapter 4.

During the 1985 Mexico City Earthquake very large soil amplifications were recorded. This brought the introduction of an additional soil profile type for deep soft clay deposits that was termed Type 4 and was assigned a soil coefficient $S = 2$. With this addition this corresponds to the soil profile classification used when the ATC-6 requirements were modified to be adopted as *Division I-A Seismic Design* of the fifteenth (1992) edition of *AASHTO Standard Specifications*. These requirements have been kept without modification in the sixteenth (1996) and in the current seventeenth (2002) edition [AASHTO, 2002] of the *AASHTO Standard Specifications*. The soil profile definition and soil factor, S , were shown previously in Fig. 2-16.

Information mainly from the 1989 Loma Prieta, California, Earthquake, and the 1985 Mexico City event, in addition to data from other cases in other parts of the world permitted in the early 1990's to restudy the ground motion amplification due to soil profile characteristics and to propose a new classification of soil profile types and soil amplification factors [Whitman, 1992]. These new procedures take into account the level of shaking, rock stiffness, and soil type, and the stiffness and depth effect on the amplification of ground motions at short and long periods. These new soil profile types and soil amplification factors were incorporated into the *1994 NEHRP Provisions* [NEHRP, 1994]. Among the more important changes are larger amplification factors for lower levels of acceleration caused by the soil responding in the linear range with less damping as compared with larger amplitude movement responding in the nonlinear range and inducing greater damping. Some adjustments were introduced in the *1997 NEHRP Provisions* [NEHRP, 1997] with the change of the definition of the design ground motion introduced then, and were kept in the *2000 NEHRP Provisions* [NEHRP, 2000].

The soil site must be assigned into one of six soil profile classes labeled A to F. The definition of the soil profile is based on averaged soil properties for the upper 100 feet (30 m) of soil profile. The properties used for this definition are: average shear wave velocity ($\bar{v}_s$), average standard penetration resistance ($\bar{N}$), average standard penetration resistance for the cohesionless soils only ($\bar{N}_{ch}$), and average undrained shear strength in case of cohesive soils ($\bar{s}_u$). These averages are weighted with respect to each layer thickness. Table 2-11 indicates how these parameters are used to define the soil profile.

Table 2-11 – Site Classification [NEHRP, 2000]

Site class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A Hard rock	>5000 ft/s (> 1500 m/s)	N/A	N/A
B Rock	2500 to 5000 ft/s (760 to 1500 m/s)	N/A	N/A
C Very dense soil or soft rock	1200 to 2500 ft/s (360 to 760 m/s)	> 50	> 2000 psf (> 100 kPa)
D Stiff soil	600 to 1200 ft/s (180 to 360 m/s)	15 to 50	1000 to 2000 psf (50 to 100 kPa)
E Soft soil	< 600 ft/s (< 180 m/s)	< 15	< 1000 psf (< 50 kPa)
	Any profile with more than 10 ft (3 m) of soil having <ul style="list-style-type: none"> • Plasticity Index $PI > 20$, • Moisture content $\omega \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf (25 kPa) 		
F Soils requiring site- specific evaluation	<ol style="list-style-type: none"> 1. Soils vulnerable to potential failure or collapse (liquefiable, quick- or highly-sensitive clays, collapsible weakly cemented soils). 2. More than 10 ft (3 m) of peat and/or highly organic clays. 3. More than 25 ft (7.5 m) of very high plasticity clays ($PI > 75$). 4. More than 120 ft (37 m) of soft to medium clays. 		

N/A =Not applicable.

Figure 2-24 shows the values of the coefficient F_a as a function of the site class and the short period maximum considered earthquake spectral acceleration S_s . Figure 2-25 shows

the values of the coefficient F_v as a function of the site class and the 1-second period maximum considered earthquake spectral acceleration S_1 . For sites class F a site-specific procedure must be used for determining the maximum considered earthquake spectral acceleration.

Fig. 2-24 – Values of F_a [NEHRP, 2000]

It is important to note that the reference accelerations for obtaining F_a and F_v , as shown in Figure 2-24 and 2-25, were changed in the *1997 NEHRP Provisions* [NEHRP, 1997] from what was presented originally in the *1994 NEHRP Provisions* [NEHRP, 1994]. In the 1994 Edition the abscissa of Figures 2-24 and 2-25 were mapped values of effective peak ground acceleration A_a (EPA) and acceleration A_v derived from effective peak ground velocity (EPV), respectively, based on a 10% probability of being exceeded in a 50-year life span of the structure — 475 years mean return period —. In the 1997 Edition the definition of the design ground motion was changed to the spectral values of the maximum considered earthquake — 2500 years mean return period — described through S_5 and S_1 respectively, as used in Figures 2-24 and 2-25.

Fig. 2-25 – Values of F_v [NEHRP, 2000]

Although this methodology for evaluation of site effects was originally introduced in documents for earthquake resistant building design, it is as well applicable to bridge design, as recognized in the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002]. This last document requires a site-specific evaluation for Site Class F soils, when the bridge is considered a major or very important structure requiring a higher degree of confidence of meeting the seismic performance objectives, or when the site is within 10 km of an active fault. The *Draft* document includes Appendix 3A containing guidelines for conducting site-specific geotechnical investigations and dynamic response analyses. Appendix 3A gives guidance on the site-specific geotechnical investigation to be performed, the modeling of the soil profile, the selection of the ground motion at bedrock to use in the analysis, the procedure of the analysis, and how to interpret the results.

2.3.2 Liquefaction

Liquefaction is a process caused by the earthquake ground motion vibration by which sediments below the water table temporarily lose strength and behave as a viscous liquid

rather than a solid. The type of sediments most susceptible are clay-free deposits of sand and silts; occasionally, gravel liquefies [Seed and Idriss, 1982]. Seismic waves, primarily shear waves, passing through saturated granular layers, distort the granular structure, and cause loosely packed groups of particles to readjust. This readjustment increases the pore-water pressure between particles if drainage cannot occur. If the pore-water pressure raises to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid producing what is called liquefaction of the soil.

In the liquefied condition, soil may deform with little shear resistance. Deformations large enough to cause damage to structures are called ground failures. The ease with which a soil can be liquefied depends primarily on the looseness of the soil, the amount of cementing or clay between particles, and the amount of drainage restriction. The amount of soil deformation following liquefaction depends on the looseness of the material, the depth, thickness, and extension measured in area of the liquefied layer, the ground slope, and the distribution of loads applied by the structure.

Fig. 2-26 – Damage due to liquefaction on Bridge 002/6s-w
2001 Nisqually, Washington, Earthquake

Liquefaction is restricted to certain geologic and hydrologic environments primarily recent deposited sand and silt in areas with high ground water levels. Generally, the younger and looser the sediment and the higher the water table, the more susceptible the soil is to liquefaction. Sediments most susceptible to liquefaction include Holocene (less than 10,000-year-old) delta, river channel, flood plain, and aeolian deposit, and poorly

compacted fills. Liquefaction has been most abundant in areas where ground water lies close — generally within 30 ft — to the ground surface; few instances of liquefaction have occurred in areas with ground water deeper than 60 ft. Dense soils, including well-compacted fills, have low susceptibility to liquefaction.

Only when liquefaction is accompanied by some form of ground displacement or ground failure it is destructive to the built environment. For engineering purposes, it is not the occurrence of liquefaction that is of prime importance, but its severity or its capability to cause damage. Adverse effects of liquefaction can take many forms. These include: flow failures; lateral spreads; ground oscillations; loss of bearing strength; settlement; and increased pressure on retaining walls.

Flow failures — Is evident by lateral displacement of large masses of soil. Flows may consist on completely liquefied soil or blocks of intact material riding on a layer of liquefied soil. It occurs on sites having loose saturated sands or silts lying on steep slopes.

Fig. 2-27 – Turnagain Heights, Anchorage, a sector approximately 2,600 m by 270 m slid 21 m toward Cook Inlet. Sand lenses liquefied and moved down slope. Slope failure began 2 minutes after the start of ground motion – 1964, Alaska, Earthquake

Lateral spread — Lateral spreads involve lateral displacement of large, surficial blocks of soil as a result of liquefaction of a subsurface layer. Displacement occurs in response to the combination of gravitational forces and inertial forces generated by an earthquake. Lateral spreads generally develop on gentle slopes (most commonly less than 3 degrees) and move toward a free face such as an incised river channel. Horizontal displacements commonly range up to several meters. The displaced ground usually breaks up internally, causing fissures, scarps, horsts, and grabens to form on the failure surface. Damage caused by lateral spreads is severely disruptive. For example, during the 1964 Alaska Earthquake, more than 200 bridges were damaged or destroyed by spreading of floodplain deposits toward river channels. The spreading compressed the superstructures, buckled decks, thrust stringers over abutments, and shifted and tilted abutments and piers [NAS, 1973].

Fig. 2-28 – Damage caused by lateral spreading at Sunset Lake trailer park in Tumwater – 2001 Nisqually, Washington, Earthquake

Loss of Bearing Strength — When the soil supporting the structure liquefies and loses strength, large deformations can occur within the soil which may allow the structure to settle and tip. Conversely, buried tanks and piles may rise buoyantly through the liquefied soil. For example, many buildings settled and tipped during the 1964 Niigata, Japan,

Earthquake. The most spectacular bearing failures during that event were in an apartment complex where several four-story buildings tipped as much as 60 degrees. Apparently, liquefaction first developed in a sand layer several meters below ground surface and then propagated upward through overlying sand layers. The rising wave of liquefaction weakened the soil supporting the buildings and allowed the structures to slowly settle and tip. In many cases, the weight of a structure will not be great enough to cause the large settlements associated with soil bearing capacity failures. However, smaller settlements may occur as soil pore-water pressures dissipate and the soil consolidates after the earthquake. The eruption of sand boils (fountains of water and sediment emanating from the pressurized, liquefied sand) is a common manifestation of liquefaction that can also lead to localized differential settlements.

Fig. 2-29 – Settlement of pile cap in liquefied soil beneath industrial building south of downtown Seattle – 2001 Nisqually, Washington, Earthquake

Increased Lateral Pressure on Retaining Walls — If the soil behind a retaining wall liquefies, the lateral pressures on the wall may greatly increase. As a result, retaining walls maybe laterally displaced, tilt, or structurally fail, as has been observed for waterfront walls retaining loose saturated sand in a number of earthquakes.

Now we turn our attention to the evolution of the engineering procedures to evaluate the liquefaction potential. The number of variables that should be taken into account to properly evaluate the liquefaction potential can be grouped into three broad groups [Seed and Idriss, 1982]: soil properties (dynamic shear modulus, damping characteristics, unit

weight, grain characteristics, relative density, and soil structure), environmental factors (procedure of soil formation, seismic history, geologic history – aging, cementation –, lateral earth pressure coefficient, depth of water table, and effective confining pressure), and earthquake characteristics (intensity of ground shaking, and duration of ground shaking). A combination of in-situ test and laboratory tests of undisturbed samples may be used to obtain properties that can be used to define the liquefaction potential using engineering judgment to properly account for the large intrinsic variability of the measured properties — because of the difficulty of retrieving and testing really undisturbed samples — and unknown or difficult to estimate parameters, specially those associated with the earthquake ground motion.

Because of the noted reasons, since the mid 1970's a methodology termed the “simplified procedure” has evolved to be the accepted standard of practice for evaluating liquefaction resistance of soils. This procedure is based on regressions of data from field observations and laboratory measurements. Recently an update of the procedures involved was made by a panel of experts under the sponsorship of the National Center for Earthquake Engineering and the National Science Foundation [Youd and Idriss, 2001]. The proposed updated procedure constitutes the method of choice in which an all-encompassing procedure is not employed.

The procedure requires the computation of two variables that describe the seismic demand on a soil layer — cyclic stress ratio (*CSR*) — and the capacity of the soil layer to resist liquefaction — cyclic resistance ratio (*CRR*) — respectively. The factor of safety to liquefaction of the soil layer is then obtained as:

$$FS = \frac{CRR}{CSR} \quad (2-18)$$

The cyclic stress ratio, *CSR*, is obtained from:

$$CSR = \left(\frac{\tau_{av}}{\sigma'_{vo}} \right) = 0.65 \cdot \left(\frac{a_{max}}{g} \right) \cdot \left(\frac{\sigma_{vo}}{\sigma'_{vo}} \right) \cdot r_d \quad (2-19)$$

where a_{max} = peak ground acceleration at the ground surface generated by the earthquake; g = acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively; τ_{av} = induced shear stress; and r_d = stress reduction coefficient accounting for flexibility of the soil profile.

For a depth below ground surface z in m, r_d can be obtained from:

$$r_d = \begin{cases} 1.0 - 0.00765 \cdot z & \text{for } z \leq 9.15 \text{ m} \\ 1.174 - 0.0267 \cdot z & \text{for } 9.15 \text{ m} < z \leq 23 \text{ m} \end{cases} \quad (2-20)$$

Four field tests are recommended for routine evaluation of liquefaction resistance CRR : the cone penetration test (CPT), the standard penetration test (SPT), shear-wave velocity (V_s) measurements, and for gravelly sites the Becker penetration test (BPT). Each test has its advantages and limitations with the CPT providing the most detailed soil stratigraphy and robust field-data based liquefaction resistance curves now available. CPT testing should always be accompanied by soil sampling for validation of soil type identification. The SPT has a longer record of application and provides disturbed soil samples from which fines content and other grain characteristics can be determined. Measured shear-wave velocities provide fundamental information on small-strain soil behavior that is useful beyond analyses of liquefaction resistance. V_s is also applicable at sites, such as landfills and gravelly sediments, where CPT and SPT soundings may not be possible or reliable. The BPT test is recommended only for gravelly sites and requires use of rough correlations between BPT and SPT, making the results less certain than other tests. Where possible, two or more test procedures should be applied to assure adequate definition of soil stratigraphy and a consistent evaluation of liquefaction resistance.

When using the standard penetration test (SPT) the blow count must be normalized to take into account the overburden pressure and the hammer energy. This is done by adjusting the blow count values, N , to a normalized value termed $(N_1)_{60}$ that corresponds to an overburden pressure, P_o , of approximately 100 kPa (2 kip/sq ft) and a hammer efficiency of 60%. The cyclic resistance ratio (CRR) is first defined for an ideal clean-sand base curve for magnitude 7.5 earthquakes:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200} \quad (2-21)$$

Equation (2-21) is valid for $(N_1)_{60}$ values less than 30. For greater values clean granular soils are too dense to liquefy and are classified as non-liquefiable. A series of adjustments must be made to $CRR_{7.5}$, as follows:

Fines content adjustment — CRR increase with soil fines content (FC) by adjusting $(N_1)_{60}$ to an equivalent clean sand value $(N_1)_{60cs}$. This is performed using the following empirical equation:

$$(N_1)_{60cs} = \alpha + \beta \cdot (N_1)_{60} \quad (2-22)$$

and α and β are obtained from:

$$\alpha = \begin{cases} 0 & \text{for } FC \leq 5\% \\ e^{\left(1.76 - \frac{190}{FC^2}\right)} & \text{for } 5\% < FC \leq 35\% \\ 5.0 & \text{for } 35\% < FC \end{cases} \quad (2-23)$$

$$\beta = \begin{cases} 1.0 & \text{for } FC \leq 5\% \\ 0.99 + \frac{FC^{1.5}}{1,000} & \text{for } 5\% < FC \leq 35\% \\ 1.2 & \text{for } 35\% < FC \end{cases}$$

Other corrections to SPT — Several corrections in addition to the fines content influence SPT results. They are incorporated by:

$$(N_1)_{60cs} = N_m \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S \quad (2-24)$$

where N_m = measured SPT blows; C_N = factor to take into account the overburden stress; C_E = factor to take into account hammer energy ratio; C_B = factor to take into account the borehole diameter; C_R = factor to take into account the rod length; and C_S = correction for samplers with or without liners. Table 2-12 lists the values for these corrections factors.

Table 2-12 – Corrections to SPT [Youd and Idriss, 2001]

Factor	Equipment variable	Term	Correction
Overburden pressure	—	C_N	$(P_o / \sigma'_{vo})^{0.5}$
Overburden pressure	—	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5 – 1.0
Energy ratio	Safety hammer	C_E	0.7 – 1.2
Energy ratio	Automatic trip Donut	C_E	0.8 – 1.3
Borehole diameter	65-115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	< 3 m	C_R	0.75
Rod length	3 – 4 m	C_R	0.8
Rod length	4 – 6 m	C_R	0.85
Rod length	6 – 10 m	C_R	0.95
Rod length	10 – 30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler w/o liners	C_S	1.1 – 1.3

The recommended procedure [Youd and Idriss, 2001] presents procedures to obtain CRR based on cone penetration test (CPT) and on shear wave velocity (V_s). They were not used in present research due to lack of CPT and V_s information for bridge sites in Indiana with few exceptions [Bobet et al., 2001]. The interested reader is referred to [Youd and Idriss, 2001].

Magnitude correction — An earthquake magnitude scaling factor (MSF) must also be applied using Eq. (2-25).

$$FS = \left(\frac{CRR_{7.5}}{CSR} \right) \cdot MSF \quad (2-25)$$

The magnitude scaling factor, MSF , for moment magnitude, M_w , is presented in Eq. (2-26).

$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \quad (2-26)$$

The proposed *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] requires a liquefaction potential evaluation, as indicated in Table 2-10, for the different component detailing requirements (SDR). For SDR 1 and 2 no evaluation is required unless requested by the owner. For SDR 3 to 6 its must be performed unless requested by the owner or one of the conditions in Table 2-13 are met.

Table 2-13 – Cases for SDR 3 to 6 for which liquefaction potential evaluation need not be performed in *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)*

Magnitude for the 3% probability of exceedance 75-year event	Normalized SPT blow count, $(N_1)_{60}$	$F_a \cdot S_s$
$M_w < 6.0$	N/A	N/A
$6.0 \leq M_w < 6.4$	> 15	N/A
	> 10	$0.375g > F_a \cdot S_s \geq 0.25g$
	> 5	$0.250g > F_a \cdot S_s$

Maps of mean earthquake magnitude are provided. These maps were prepared by the USGS using the maximum magnitude that can affect the site for all relevant seismic sources to produce the design ground motion with 3% of being exceeded in 75 years. If liquefaction potential must be evaluated, the procedures of Appendix 3B must be used.

Appendix 3B is devoted to liquefaction and other geological hazards. It uses the same principles presented [Youd and Idriss, 2001], and gives guidance for making an in depth evaluation of liquefaction potential. The results of the liquefaction assessment are used to evaluate the potential severity of three liquefaction related hazards to the bridge: (1) flow failures involving large slope failures, (2) limited lateral spread, and (3) ground settlement. It permits to evaluate these hazards on the basis of safety factor for liquefaction. If the safety factor is less than 1.0 to 1.3 the potential related hazards must be evaluated following detailed guidelines for each one of them.

In general the procedures included require improving the soil or locating the support foundation elements below the liquefiable layer and to design the bridge to meet the

performance objectives while being subjected to the settlements and/or lateral displacements imposed by the lateral spread.

2.3.3 Fault rupture

The morphology of rivers is associated with geological fault setting in many cases. The possibility of having a bridge located along the path of a geological fault is real. Two recent cases of bridge damage caused by the bridge crossing the fault path are shown in Figures 2-30 and 2-31.

Fig. 2-30 – Collapse of Bei-Feng Bridge located on the earthquake causing fault
Chi-Chi, Taiwan Earthquake, September 21, 1999

Fig. 2-31 – Bolu Viaduct, crossed by the North Anatolian Fault
November 12, 1999 Duzce, Turkey, Earthquake

In many cases the fault existence and location is difficult to detect. The degree of activity of a fault is subjective and estimation the amount of displacements and recurrence period is difficult to forecast. There are very few acceleration records obtained very close to a fault and the expected ground motion is difficult to estimate. Design for these circumstances is challenging and has a high degree of uncertainty in meeting any performance objective. The proposed *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] recommends avoiding bridge construction across faults.

2.3.4 Embankment failure

High embankments in bridge approaches may fail due to different type of problems. Liquefaction may cause lateral spread or settlement, leading to failure of the embankment. The ground motion, without liquefaction occurring, may cause the failure of the slopes. Techniques developed for embankment dams and slope stability may be used to assess the possibility of failure of the road embankment or the bridge approach.

2.3.5 Landslides

Landslides present a significant hazard to roadways in seismically active areas and can pose a hazard for bridges. Damage can be in the form of ground movement either at the abutment or extending to the central piers of the bridge. Bridges located near steep slopes, or places with a history of rock falls, or avalanches.

Fig. 2-32 – Landslide induced by the earthquake. A nearby bridge had major structural damage – Morgan Hill, California, Earthquake of April 24, 1984

Design must include dynamic slope stability analysis or other procedures of stability assessment of nearby slopes, such as Newmark sliding block method. It is important to estimate the order of magnitude of displacement during the landslide.

CHAPTER 3 SEISMIC RISK OF SOUTHWESTERN INDIANA

3.1 Tectonic setting

The great majority of the World's seismic activity occurs at the tectonic plate boundaries. Continental U.S. is contained within the North American Plate with Indiana practically in the middle of it and far away from the plate boundaries. Notwithstanding, earthquakes have been felt in Indiana since colonization began by the French in Louisiana and from the east after the Louisiana Purchase. Intraplate earthquakes can be caused by a number of different kind of stresses — large scale glacial rebound (the slow flexure of the crust back up after a large sheet of ice is removed), for example, or the broad compressional stress caused within eastern North America by the compression forces from the mid-Atlantic spreading center.

In recent decades, earth scientists have collected evidence that strong earthquakes in the central Mississippi Valley have occurred repeatedly in the geologic past. Small earthquakes happen in the region frequently. The area in which most of these quakes occur is referred to as the New Madrid seismic zone (NMSZ) which stretches from just west of Memphis, Tennessee, into southern Illinois. Several times in the past century, moderate earthquakes have been widely felt in southern Illinois and southwestern Indiana. In the last decade geologic evidence that the Wabash Valley faults, initially considered the northern portion of the New Madrid Seismic Zone, is a different system with its own tectonic setting has led to define it as an independent seismic zone that has been named the Wabash Valley Seismic Zone (WVSZ).

The New Madrid Seismic Zone (NMSZ) — Geologic structures related to earthquakes in the central Mississippi Valley region have been deeply buried over hundreds of millions of years by thick layers of sediment. Geophysical studies have revealed a major buried northeast-trending feature known as the Reelfoot Rift [Hildenbrand and Hendricks, 1995], which formed more than 500 million years ago by a process of

extension in the middle of the North American Plate. A rift structure is created when geologic forces begin to pull the Earth's crust apart. If this process continues long enough, the crust separates to form an ocean basin, as happened to form the Atlantic Ocean basin. The Reelfoot Rift is called a failed rift because the Earth's crust did not separate enough to create a new ocean basin. However, the crust was disturbed enough to form major faults that mark the axis and margins of the rift and now contribute to the occurrence of earthquakes in the NMSZ.

Fig. 3-1 – The Reelfoot Rift in the New Madrid Region

A sequence of powerful earthquakes struck the mid-Mississippi River Valley, central United States, in the winter of 1811-1812. The first one occurred December 16, 1811, Intermittent strong shaking continued through March 1812 and aftershocks strong enough to be felt occurred through the year 1817. The initial earthquake of December 16 was followed by two other principal shocks, one on January 23, 1812, and the other on February 7, 1812. Judging from newspaper accounts of damage to buildings, the February 7 earthquake was the biggest of the three. On the basis of the large area of damage (600,000 km²), the widespread area of perceptibility (5,000,000 km²), and the complex physiographic changes that occurred, the Mississippi River valley earthquakes

of 1811-1812 rank as some of the largest in the United States since its settlement by Europeans [Stover and Coffman, 1993]. The estimated magnitudes for these events vary considerably from different sources (7.2 to 8.8 depending on the parameters used to define the magnitude and the scale in which it is reported).

Fig. 3-2 – Isoseismal map for the Arkansas earthquake of December 16, 1811, first of the 1811-1812 New Madrid series [Stover and Coffman, 1993]

The association of the 1811-1812 earthquakes to a causing fault has been hindered by lack of information from a scarcely inhabited region at the time of occurrence. Since the early 1970's a great amount of multidisciplinary research have been devoted to find and define characteristics of the events and the tectonic setting in order to understand the tectonic process taking place and validate seismic risk assessments for the region.

From the information gathered from different sources it is accepted (with dissent) that the causing faults were those shown in Fig. 3-3. The USGS along with the Geological

Surveys of the CUSEC member States and several universities in the region have to a certain degree reached consensus on what is known with certainty and what needs more research to frame adequately the seismic risk assessment for the region. The primary types of evidence are geological — paleoseismological, stratigraphic, geomorphic, structural, and sedimentological information — and seismological, instrumental and historical.

Fig. 3-3 – The New Madrid seismic zone showing the three faults that ruptured in 1811-1812 and earthquakes recorded in recent years [Hough, 2002]

Paleoliquefaction investigations have provided constraints on recurrence in the NMSZ and have been conducted at 44 sites. From these studies it is known that two pre-1811 episodes of liquefaction comparable to that of 1811-1812 can be interpreted [Participants, 2000]. Best estimates of the dates of these events are 1450 and 900 (Fig. 3-4), which leads to a median recurrence interval ranging between 267 and 644 years. The volume of sand mobilized at many of the liquefaction sites in the NMSZ implies they could not have been formed by local events of moderate magnitude. Further indication of

very strong shaking comes from geotechnical measurements at liquefaction sites in the NMSZ, which show the shallow sub-surface materials to be only moderately liquefiable.

Fig. 3-4 – Estimated ages of paleoliquefaction features (vertical axis) arranged by site location, from north (left) to south (right), excluding the Current River sites. Vertical gray shaded segments indicate most probable ages, with brackets showing approximate two standard deviation uncertainties. Shaded horizontal bars indicate the inferred timing of paleoearthquakes. Dashed horizontal line is at 1811-1812. [Participants, 2000]

The paleoliquefaction interpretations also suggest that recurrence rates for events with magnitude on the range of 5 and the largest earthquakes cannot be simply related. Magnitude regressions of the large events for the approximate periods of recurrence implied from the paleoliquefaction record indicate that the mean return period for the smaller magnitude events should be smaller (more frequent). Quoting from [Participants, 2000]: “In short, the largest earthquakes occur much more frequently than the rate of smaller earthquakes would imply. Finally, the paleoliquefaction record suggests that the clustering of earthquakes that occurred in 1811-1812 also occurred in prior events. Multiple units of vented material are evident in many of the sandblows and have been interpreted as resulting from major events that occurred within weeks to months of each other. The lack of soil development, but evidence for bioturbation in materials between the units, constrains the timing between clustered events.”

The length of the active fault system and the lack of relative motion measured across the system make the use of models that work well for earthquakes occurring in plate-boundary settings not appropriate. Models for intraplate tectonics have recently been proposed but their use is just being implemented in the NMSZ. Magnitude estimates of large New Madrid earthquakes rely on uncertain conversions of observations of their effects measured as Modified Mercalli intensities derived mainly from newspapers accounts. It is evident that the spread of intensity is much larger than for comparable events in the West Coast, as shown in Fig. 3-5. The magnitude conversion may be biased by different causes, but one that has been pointed out several times is that soil profile amplification may have increased the reported magnitudes because most towns at the time were settlements along rivers.

Fig. 3-5 – Comparison of areas of damage from the New Madrid and San Francisco Earthquakes

Slip rates for the known faults in the region estimated mainly using characteristics of secondary features show that three episodes of deformation are apparent in folded fluvial deposits overlying the Reelfoot fault with their ages close to the three major events evident in the paleoliquefaction record. The inferred rates of slip are compatible with the short recurrence interval for major earthquakes evident in the paleoseismic record. The

potential for currently classified as non-active faults within and beyond the NMSZ producing moderate-to-large earthquakes remains unknown, although Quaternary surface faulting has been documented on the periphery of the NMSZ [Participants, 2000].

Wabash Valley Seismic Zone — The Wabash Valley Fault System is about 90 km long and 50 km wide in southeastern Illinois, southwestern Indiana, and northwestern Kentucky near the deepest part of the Illinois Basin. The fault system consists of subparallel, high-angle normal faults that have vertical displacements as great as 150 m. The faults bound horsts and grabens, and commonly overlap one another. Major fault plains dip at angles ranging from 50 to 85 degrees. Individual fault blocks are only slightly tilted, and drag is generally absent or weakly expressed.

Fig. 3-6 – Wabash River Valley Faults [René and Stanonis, 1995]

In the past decade, there has been increasing awareness that the seismic hazard in this zone may be greater than the historical earthquake record would suggest. Numerous prehistoric large magnitude earthquakes have struck southern Indiana and Illinois; some may have been larger. Geologic evidence for these earthquakes in the form of sand-and-gravel intrusions in river sediments has been discovered at more than 200 sites in the Wabash River Valley and along its tributaries in Indiana and Illinois. This information is the result of USGS studies done in cooperation with the Indiana Geological Survey, Indiana University, Illinois Department of Natural Resources-Illinois State Museum and the Illinois State Geological Survey. Clastic dikes filled with sand and gravel, interpreted to be the result of earthquake induced liquefaction, occur throughout much of southern Indiana and adjacent parts of Illinois. At least seven and probably eight prehistoric earthquakes have been documented during the Holocene, as well as, at least one during the latest Pleistocene (see Fig. 3-7) [Munson et al., 1997]. Nearly all of these liquefaction features originated from earthquakes centered in southern Indiana and Illinois, and not further south in the nearby source region of the great 1811-12 New Madrid earthquakes [Crone and Wheeler, 2000].

Fig. 3-7 – Map of southern two-thirds of Indiana showing sites where ancient sandblows have been found, and showing areas of liquefaction for six major prehistoric earthquakes. [Kirby, 2001]

In the absence of well-determined data on the timing of paleoevents and the amount of tectonic slip associated with those events, it is impossible to estimate reliable or even meaningful Holocene or late Quaternary slip rates [Crone and Wheeler, 2000]. In summary the tectonics of the Wabash Valley Seismic Zone it not well understood at

present. No historical earthquakes in the Wabash Valley region have been strong enough to cause liquefaction. It is likely that numerous other magnitude 6 to 7 Holocene earthquakes have struck the region, but did not leave a record because of the lack of liquefiable deposits in large parts of the region.

3.2 Earthquake history of Indiana

Earthquakes originated within Indiana and from neighboring seismic zones have been felt in Indiana since the start of the colonization. Figure 3-8 shows the seismicity for the New Madrid and Wabash Valley Seismic Zones. The following description of the seismic history of Indiana was abridged from [Stover and Coffman, 1993].

Fig. 3-8 – Earthquakes in the New Madrid and Wabash Valley, red circles indicate earthquakes that occurred from 1974 to 2002 with magnitudes larger than 2.5, green circles denote earthquakes that occurred prior to 1974 [USGS, 2002]

The most damaging Indiana earthquake originating within the State occurred on September 27, 1909, near the Illinois border between Vincennes and Terre Haute. Some chimneys fell, several building walls were cracked, light connections were severed, and pictures were shaken off the walls. It was strong in Indianapolis and Oakland City. It was felt over an area of 80,000 km² (see Fig. 3-9) including the southwestern half of Indiana, all of Illinois and parts of Iowa, Kentucky, Missouri, Arkansas, and probably in parts of Kansas.

Fig. 3-9 – Isoseismal map for the Wabash River Valley, Indiana, Earthquake of September 27, 1909.

The latest important earthquake originated within the State occurred June 18, 2002, with magnitude 4.6 (USGS, but $m_b = 5.0$ according to other sources) near Evansville. There was very little damage. The location of the earthquake and its hypocentral depth (14-18 km) indicated ongoing deformation along reactivated Precambrian and Paleozoic basement structures, in a zone of recurring seismic activity, and in an area of possibly heightened neotectonic strain [Hamburger et al., 2002]. Figure 3-10 shows the intensity

map for this earthquake obtained from the Community Internet Intensity Map USGS program.

Fig. 3-10 – Iseismal map for the Evansville, Indiana, Earthquake of June 18, 2002

Other damaging earthquakes originated in Indiana include the April 29, 1899, Earthquake with rated intensity VI to VII on the Modified Mercalli Scale. It was strongest at Jeffersonville and Shelbyville; at Vincennes, chimneys were thrown down and walls cracked. It was felt over an area of 110,000 km².

In 1876, twin shocks fifteen minutes apart were felt over an area of 160,000 km². A shock in 1887 centered near Vincennes was felt over 200,000 km² and an 1891 shock damaged property and frightened people in church at Evansville.

Indiana has also suffered from damage caused by earthquakes originating in neighboring States. The worst occurred on November 9, 1968, and centered near Dale in southern Illinois. The shock, a magnitude 5.3, was felt over 1,500,000 km² and 23 States including all of Indiana. Intensity VII was reported from Cynthiana where chimneys were cracked, twisted, and toppled; at Fort Branch where groceries fell from shelves and a loud roaring noise was heard, and at Mount Vernon, New Harmony, Petersburg, Princeton, and Stewartsville, all of which had similar effects.

Almost exactly ten years earlier on November 7, 1958, an earthquake originating near Mt. Carmel, Illinois, caused plaster to fall at Fort Branch. Roaring and whistling noises were heard at Central City and the residents of Evansville thought there had been an explosion or plane crash. It was felt over 90,000 km² of Illinois, Indiana, Missouri, and Kentucky.

On March 2, 1937, a shock centering near Anna, Ohio, threw objects from shelves at Fort Wayne and some plaster fell. Plaster was also cracked at Indianapolis. Six days later, another shock originating at Anna brought pictures crashing down and cracked plaster in Fort Wayne and was strongly felt at Lafayette.

The great New Madrid earthquakes of 1811 and 1812 must have strongly affected the State, particularly the southwestern part, but there is little information available.

3.3 Design earthquake return period

For the foregoing discussion on the seismic risk of Indiana it is important to establish a common base for comparing the probability of exceedance of the design ground motion, the earthquake mean return period, and the lapse of exposure or life of the structure.

The probability, q , of having ground motion that exceeds in one year a pre-fixed value of a descriptive parameter, a_o , – for example peak ground acceleration – is:

$$q = P(a \geq a_o, \text{ in one year}) \quad (3-1)$$

The probability of not exceeding the value of a_o in t years, under the assumption of statistical independence between events, is:

$$q_t = (1-q) \underset{1}{(1-q)} \underset{2}{(1-q)} \underset{3}{(1-q)} \cdots \underset{t}{(1-q)} \quad (3-2)$$

Then:

$$q_t = (1-q)^t \quad (3-3)$$

The probability, q_o , of having at least one event with a value greater than a_o in a time span of t years is:

$$q_o = 1 - q_t = 1 - (1-q)^t \quad (3-4)$$

The return period is defined as the mean time, in years, between events producing a value of the descriptive parameter greater or equal to a_o :

$$\bar{T} = \frac{1}{q} \quad (3-5)$$

Then:

$$q_o = 1 - q_t = 1 - \left(1 - \frac{1}{\bar{T}}\right)^t \quad (3-6)$$

For example, if the design earthquake has been defined as an event that produces a value of the descriptive parameter having a probability of 3% of being exceeded in a 75 year interval; using the formulation presented: $q_o = 0.03$ and $t = 75$ years. Applying Eq. (3-6):

$$0.03 = 1 - \left(1 - \frac{1}{\bar{T}}\right)^{75} \quad \text{then} \quad 0.97 = \left(1 - \frac{1}{\bar{T}}\right)^{75} \quad \text{and} \quad \left(1 - \frac{1}{\bar{T}}\right) = 0.97^{\left(\frac{1}{75}\right)} = 0.999594$$

It follows that $\frac{1}{\bar{T}} = 1 - 0.999594 = 0.00040604$, and the mean period of recurrence for this case is, then:

$$\bar{T} = \frac{1}{0.00040604} = 2463 \text{ years}$$

Table 3-1 – Design Ground Motion Mean Return period used in different documents

Document and reference	Structure life-span (years)	Probability of exceedance	Mean Return Period (years)
AASHTO Standard Specifications for Highway Bridges 5 th Edition (1949)	N/A	N/A	N/A
AASHTO Standard Specifications for Highway Bridges 8 th Edition (1961)	N/A	N/A	N/A
AASHTO 1975 Interim Specifications for Highway Bridges [AASHTO, 1975]	N/A	N/A	N/A
ATC-3-06 Recommendations for Buildings [ATC, 1978]	50	10%	475
ATC-6 Design Guidelines for Bridges [ATC, 1981]	50	10%	475
Division I-A of AASHTO Standard Specifications (15 th to current 17 th Edition) [AASHTO, 2002]	50	10%	475
AASHTO LRFD Specifications (1 st , 2 nd , and 3 rd Edition) [AASHTO, 1998]	50	10%	475
Caltrans Seismic Design Criteria [Caltrans, 2001]	N/A	N/A	N/A
NEHRP Recommendations for New Buildings [NEHRP, 1997] – Maximum Considered Earthquake (see Note)	50	2%	2475
Draft AASHTO LRFD Seismic Provisions [NCHRP, 2002] – Maximum Considered Earthquake	75	3%	2463
Draft AASHTO LRFD Seismic Provisions [NCHRP, 2002] – Frequent Earthquake	75	50%	109

Note – In NEHRP 1997 the Design Ground Motion for buildings corresponds to 2/3 of the Maximum Considered Earthquake ground motion. This reduction is not used in the Draft AASHTO LRFD seismic provisions for bridges although the definition of the Maximum Considered Earthquake is practically the same.

Figure 3-11 shows the sensitivity to the mean return period when changing the life span of the structure or the probability of exceedance.

Fig. 3-11 – Mean return period as a function of the probability of exceedance and the life span of the structure in years

3.4 Seismic risk assessment for bridge design in Indiana

The awareness of the seismic risk associated with the New Madrid Seismic Zone and more recently the Wabash Valley Seismic Zone, as presented before, has been increasing in the last decades with the corresponding increase in the seismic forces and related requirements used in design of building structures and bridges. The following sections present the seismic risk parameters presented by the bridge specifications in the last decades in order to set a common frame of reference to be used in the vulnerability assessment of the transportation structures within the selected Earthquake Emergency Routes of Indiana.

Figure 3-12 shows the 1949 revision of the seismic risk map developed by the U.S. Coast and Geodetic Survey [Housner, 1970]. Table 3-2 presents the peak ground accelerations suggested by Housner to be used with the map of Fig. 3-12.

Fig. 3-12 – U.S. Coast and Geodetic Survey Seismic Probability Map of the United States (Revised 1949)

Table 3-2 – Maximum zonal acceleration to be used with map of Fig. 3-12 [Housner, 1970]

	Maximum Acceleration	M
Zone 3 (near a great fault)	0.50g	8.5
Zone 3 (not near a great fault)	0.33g	7.0
Zone 2	0.16g	5.75
Zone 1	0.08g	4.75
Zone 0	0.04g	4.25

This map was developed before seismic zoning was introduced in the building or bridge design requirements. Although the map was referred as a seismic probability map it actually was based on Mercalli intensity and no guidance was given on the corresponding return period or probability of exceedance. Seen from a modern point of view this map contains features such as a neighboring Zone 1 and 3 areas without a transition Zone 2 in between, and other that currently would be frowned at. Notwithstanding, the general location of the zones and envisioned peak ground acceleration do not differ much from more recent maps. In this map southwestern Indiana

is located in a Zone 2 corresponding to a maximum acceleration of 0.16g and a corresponding magnitude of 5.75 for the design event. The rest of Indiana, with the exception of a small Zone 2 bordering Ohio, was located in a Zone 1 (maximum expected acceleration of 0.08g and magnitude of 4.75 for the design event. No design response spectrum was defined. This map was never mandatory in Indiana for bridges or buildings.

3.4.1 1975 AASHTO Interim Specifications for Highway Bridges

In Section 2.2.2 the *1975 AASHTO Interim Specifications* [AASHTO, 1975] were presented. Figure 3-13 shows the northern Midwest portion of the map including Indiana (the whole U.S. map is shown in Fig. 2-13). The seismic design criteria contained in the 1975 AASHTO Interim Specifications was included in the twelfth (1977), thirteenth (1983), and fourteenth (1989) editions of AASHTO's Standard Specifications.

Fig. 3-13 – Northern Midwest portion of the map included with the *1975 AASHTO Interim Specifications* [AASHTO, 1975]

No more detailed map was included with the 1975 Interim. The southwestern corner of Indiana containing Gibson, Posey and Vanderburgh counties was a Zone 3 with a peak ground acceleration $A = 0.5g$. A region south of an approximate line linking Terre Haute, Indianapolis and Fort Wayne was included in a Zone 2 ($A = 0.22g$ PGA). The rest of the State was located in a Zone 1 ($A = 0.09g$ PGA).

Table 3-3 – Maximum values for the Response Coefficient (C) in Indiana
1975 AASHTO Interim Specifications [AASHTO, 1975]

Depth of Alluvium	Zone 1 ($A = 0.09g$)		Zone 2 ($A = 0.22g$)		Zone 3 ($A = 0.50g$)	
	C max	T at peak	C max	T at peak	C max	T at peak
0 – 10'	0.060	-	0.072	0.30 s	0.163	0.30 s
11' – 80'	0.066	0.40 s	0.120	0.35 s	0.203	0.30 s
80' – 150'	0.080	0.60 s	0.110	0.55 s	0.155	0.50 s
> 150'	0.065	0.65 s	0.090	0.55 s	0.120	0.50 s

3.4.2 ATC 6 and AASHTO Division I-A

The *ATC-6* [ATC, 1981] recommendations were modified to be incorporated in the fifteenth (1992) edition of *AASHTO Standard Specifications* as *Division I-A Seismic Design*. These requirements were kept without modification in the sixteenth (1996) and in the current seventeenth (2002) edition [AASHTO, 2002]. The *ATC-6* document used for peak ground acceleration in rock, A , the map of A_v originally proposed for the ATC-3-06 building requirements project.

The ATC-3-06 requirements for buildings were taken over by the BSSC and became the NEHRP recommended provisions. By the time the *ATC-6* proposed bridge requirements were incorporated into *AASHTO Standard Specifications* the maps had been updated in the 1988 NEHRP provisions [NEHRP, 1988] by the USGS [Algermissen et al., 1990]. The map included in the 15th Edition AASHTO Standard Specifications and still contained in the current 17th Edition correspond to the A_v map as modified in 1988. Figure 3-14 shows the State of Indiana portion of this map. The values given correspond to peak ground acceleration in rock expressed as percentage of the acceleration of gravity for a probability of exceedance of 10 percent in 50 years, which is approximately equal to a 15 percent probability of exceedance in the 75-year design life now advocated for a typical highway bridge (see Fig. 3-11).

The southwestern corner of Indiana is located within contours labeled for peak ground acceleration of $A = 0.075g$ and $A = 0.10g$; including totally (t) or partially (p) the following 20 counties: Brown (p), Clay (p), Daviess (t), Dubois (p), Gibson (t), Greene (t), Knox (t), Lawrence (p), Martin (t), Monroe (p), Morgan (p), Owen (t), Pike (t), Posey (t), Putnam (p), Spencer (p), Sullivan (t), Vanderburgh (t), Vigo (p), and Warrick (t)..

Section 3.2 of *Division I-A Seismic Design* indicates that “*Local maxima (and minima) are given inside the highest (and lowest) contour for a particular region. Linear interpolation shall be used for sites located between contour lines and between a contour line and local maximum (or minimum).*”

The rest of the south half of the state is located within a contour line for peak ground acceleration $A = 0.05g$ and local maxima labeled as $A = 0.06g$. This zone includes Terre Haute and Indianapolis. The northern half of the State, including the cities of Lafayette and Fort Wayne, located within the contour line for peak ground acceleration $A = 0.05g$ and local maxima labeled as $A = 0.04g$.

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Fig. 3-14 – Values for peak ground acceleration for the State on Indiana in *Division I-A Seismic Design of 17th Edition of AASHTO Standard Specifications* [AASHTO, 2002]

All editions of the AASHTO LRFD Specifications (1994, 1998, 2004) [AASHTO, 2004] include the same map but contain variations in the design requirements that may lead to a different design although based on the same seismic risk assessment (see 2.2.2).

3.4.3 USGS 1996 Maps for the Draft AASHTO LRFD Seismic Provisions

In 1993, the USGS embarked on a major project to prepare updated national earthquake ground motion maps. The result of that project was a set of probabilistic maps published in 1996 that cover several rock ground motion parameters (peak ground acceleration — PGA — and elastic response spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 sec) and three different probability levels or return periods (10% probability of exceedance in 50 years, 5% probability of exceedance in 50 years, and 2% probability of exceedance in 50 years — which is approximately equal to 3% probability of exceedance in 75 years). In addition to the maps, the ground motion values at any specified latitude and longitude can be obtained via the Internet.

Fig. 3-15 – 1996 USGS Map of peak ground acceleration in rock for a probability of exceedance of 10% in 50 years

These maps form the rock ground motion basis for seismic design using the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002]. Figure 3-15 shows the 1996 USGS Map of peak ground acceleration in rock for a probability of exceedance of 10% in 50 years that although not used in the *Draft AASHTO LRFD Seismic Provisions*, is given here for reference with previous requirements.

Fig. 3-16 – 1996 USGS Map of peak ground acceleration in rock for a probability of exceedance of 3% in 75 years (= to 2% in 50 years)

The upper-level event, termed the “rare” or Maximum Considered Earthquake (MCE), describes ground motions that, for most locations, are defined probabilistically and have a probability of exceedance of 3% in 75 years (mean return period approximately 2500 years). Figure 3-16 shows the 1996 USGS Map of peak ground

acceleration in rock for a probability of exceedance of 2% in 50 years (equivalent to 3% in 75 years as shown in Fig. 3-11) that although not used also in the *Draft AASHTO LRFD Seismic Provisions*, is given here for reference with previous requirements using a different return period.

For locations close to highly active faults, the mapped Maximum Considered Earthquake ground motion was deterministically bounded so that the levels of ground motion did not become unreasonably high. Deterministic bounds on the ground motion were calculated by assuming the occurrence of maximum magnitude earthquakes on the highly active faults. It is equal to 150% of the median ground motion for the maximum magnitude earthquake, but not less than 1.5g for the short-period spectral acceleration plateau and 0.6g for 1.0-sec spectra acceleration (see Fig. 2-22). Deterministic bounds were applied in high-seismicity portions of the West Coast, Alaska, and Hawaii leading to a design ground motion lower than the ground motion for 3% probability of exceedance in 75 years. The Maximum Considered Earthquake governs the limits on the inelastic deformation in the substructures and the design displacements for the support of the superstructure.

The lower level design event, termed the “expected” or “frequent” earthquake, defines a ground motion corresponding to 50% probability of exceedance in 75 years. This event ensures that essentially elastic response is achieved in the bridge substructure for the more frequent or expected earthquake. This design level is similar to the 100-year flood and has similar performance objectives. An explicit check on the strength capacity of the bridge substructure is required. Parameter studies made as part of the development of the draft provisions show that the lower level event will only impact the strength of columns in parts of the western United States.

With respect to Indiana the USGS 1996 Maps used the then recently identified paleoearthquakes in southern Indiana and Illinois based on widespread paleoliquefaction features as described in 3.1. An areal zone with a higher $M_{\max}$ of 7.5 to account for such large events was used. The Wabash Valley $M_{\max}$ zone that was used in the maps was based on the Wabash Valley fault zone.

Fig. 3-17 – 1996 USGS Map of spectral acceleration S_0 in rock for a probability of exceedance of 3% in 75 years (= to 2% in 50 years)

Fig. 3-18 – 1996 USGS Map of spectral acceleration S_1 in rock for a probability of exceedance of 3% in 75 years (= to 2% in 50 years)

The *Draft AASHTO LRFD Seismic Provisions* uses maps that define spectral ordinates at periods of 0.2 sec. (S_S) and 1.0 sec. (S_1) to be used as a two point procedure to obtain the design response spectrum (see Fig. 2-21). Figures 3-17 and 3-18 show the maps for S_S and S_1 in rock respectively for the Maximum Considered Earthquake having a 3% probability of exceedance in 75 years.

No maps were developed by USGS for the expected frequent earthquake, defining a ground motion corresponding to 50% probability of exceedance in 75 years. The *Draft AASHTO LRFD Seismic Provisions* suggests the values of S_S and S_1 for this earthquake may be obtained from interpolation from information provided by USGS either in the CD-ROM published with the maps or in the Internet by using latitude and longitude for the site or the ZIP code of the location. The web site to perform these operations is: <http://eqint.cr.usgs.gov/eq/html/deaggint.html>

Fig. 3-19 – Results obtained for Vincennes, IN, for spectral acceleration S_S for the frequent earthquake, in rock for 50% probability of exceedance in 75 years

Figure 3-19 show the results obtained for Vincennes for spectral acceleration S_S in rock for 50% probability of exceedance in 75 years as required for the frequent earthquake. The values obtained were $S_S = 0.06g$ and $S_1 = 0.01g$.

3.4.4 2002 Update of the 1996 USGS Maps

In 2002 changes were introduced by the USGS [Frankel et al., 2002] to the maps developed in 1996. Numerous changes were introduced with varying effects on the mapped values. Some of these changes affected the parameters used in the New Madrid Seismic Zone and the Wabash Valley Seismic Zone. Among them the more important are changes in the mean recurrence time, the characteristic magnitude, and the spatial concentration of New Madrid sources of large earthquakes, and the incorporation of additional attenuation relations. Two versions of the maps exist one from January and the other from October. Only the later version maps are be presented here.

The 2002 update uses a shorter mean recurrence time for characteristic earthquakes in New Madrid than was used in the 1996 maps, as well as a smaller median magnitude than that applied in 1996. The three equally weighted “fictitious sources” used in the 1996 maps were substituted for three sources consisting in a fault trace matching recent micro-earthquake activity and two adjacent sources that are situated near the borders of the Reelfoot Rift. It is important to note that the probabilistic ground motions for the 10% probability of exceedance level have increased markedly around the New Madrid area, compared to the 1996 maps. This is caused by the shorter mean return time of 500 years for characteristic earthquakes used in the 2002 maps. The $M_{\max}$ 7.5 zone assigned to the Wabash Valley area was enlarged to include the most likely rupture zones from paleoearthquakes with magnitudes above about 7.0.

Significant differences between the 1996 and 2002 maps are caused by the inclusion of additional attenuation relations in the 2002 maps. These changes affected mainly the Atlantic seaboard. There is little change for the probabilistic ground motions at 2% probability of exceedance in 50 years for most of the central U.S. During 2003, changes were made twice to the 2002 maps but these changes only affect some areas of the West Coast.

Fig. 3-20 – 2002 USGS Map for Peak Ground Acceleration in rock with 10% probability of exceedance in 50 years

Fig. 3-21 – 2002 USGS Map for Peak Ground Acceleration in rock with 3% probability of exceedance in 75 years (= to 2% in 50 years)

Fig. 3-22 – 2002 USGS Map for S_5 Spectral Acceleration in rock with 3% probability of exceedance in 75 years (= to 2% in 50 years)

Fig. 3-23 – 2002 USGS Map for S_1 Spectral Acceleration in rock with 3% probability of exceedance in 75 years (= to 2% in 50 years)

In April 2004 the USGS included in their website the possibility of producing maps for latitude and magnitude pairs. The Indiana PGA map for 475-year return period is presented in Fig. 3-20 and for the 2500-year return period in Fig. 3-21, and the maps for S_S and S_1 for 2500-year return period in Figures 3-22 and 3-23 respectively.

3.4.5 JTRP Project SPR 2812 – Analysis of Seismic Hazard Assessment for Indiana

A Joint Transportation Research Program Project (SPR 2812) lead by Purdue Professors J. Haase and R. Nowack is currently working on an assessment of the seismic hazard in Indiana with specific application to transportation structures. The results of this research were not available to be used in the Emergency Routes Project but undoubtedly will have a large impact in the application of future bridge design specifications for the Indiana Department of Transportation and seismic rehabilitation policies to be used with existing bridges. Once the *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* is adopted by AASHTO, and later by INDOT, the need for having an in-house State assessment of the seismic risk to be coordinated with the USGS will become very important.

The *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* includes in Commentary to Section 3.10.2.1 a clause that reads: “*In lieu of using the national ground motion maps referenced in the Specification, ground motion response spectra may be constructed based on approved State ground motion maps. To be accepted the development of State maps should conform to the following:*

1. *The definition of design ground motion should be the same as described in Article 3.10.1.2 and Table 3.10.1-1.*
2. *Ground motion maps should be based on a detailed analysis demonstrated to lead a quantification of ground motion at a regional scale that is as or more accurate than achieved at the scale of the national maps. The analysis should include: characterization of seismic sources and ground motion that incorporates current scientific knowledge; incorporation of uncertainty in seismic source and ground motion models and parameter values used in the analysis; detailed documentation of map development; detailed peer review. The peer review process should*

preferably include one or more individuals from the U.S. Geological Survey who participated in the development of the national maps.”

3.4.6 Comments on the results from the different maps and bridge design requirements

Mapped values for acceleration in rock in Bloomington, Evansville, Indianapolis, Jasper, New Albany, Terre Haute, and Vincennes (see Fig. 3-24 for location) are shown as read from the different seismic risk maps in Table 3-4.

Fig. 3-24 – Location where acceleration mapped values are compared for the different seismic risk map versions (see Table 3-4). Base map shows INDOT districts.

In comparing the values shown in Table 3-4 it must be noticed that the values from the 1949 map developed by the U.S. Coast and Geodetic Survey [Housner, 1970] and in the 1975 AASHTO Interim [AASHTO, 1975] are higher, as a rule, than the values for peak ground acceleration contained in *Division I-A Seismic Design* of the current 17th Edition of the *AASHTO Standard Specifications* [AASHTO, 2002]. It may be argued that the values contained in Division I-A correspond to EPA (Effective Peak Ground Acceleration) which takes into account several additional parameters such as duration of

the strong ground motion and are not directly comparable to the instrumental peak ground acceleration recorded in an accelerometer, in addition to the acceleration in the 1949 and 1975 documents not being defined clearly with respect to mean return period.

Table 3-4 – Seismic Zone definition and mapped values of acceleration for selected cities in Indiana for the different maps

Map	City (County) → Parameter Presented ↓	Bloomington (Monroe)	Evansville (Vanderburgh)	Indianapolis (Marion)	Jasper (Du Bois)	New Albany (Floyd)	Terre Haute (Vigo)	Vincennes (Knox)
1949 USCGS	Zone	2	2	1	2	1	2	2
	PGA	0.16g	0.16g	0.08g	0.16g	0.08g	0.16g	0.16g
AASHTO 1975 Interim	Zone	2	3	2	2	2	2	2
	A (PGA)	0.22g	0.50g	0.22g	0.22g	0.22g	0.22g	0.22g
AASHTO Div. I-A	A (PGA) 10% PE 50y	0.078g	0.085g	0.063g	0.075g	0.068g	0.070g	0.090g
1996 USGS Maps	PGA 10% PE 50y	0.045g	0.130g	0.032g	0.070g	0.039g	0.056g	0.110g
	PGA 2% PE 50y	0.14g	0.49g	0.08g	0.20g	0.12g	0.08g	0.40g
	S_s 2% PE 50y	0.28g	0.80g	0.20g	0.60g	0.22g	0.36g	0.80g
	S_1 2% PE 50y	0.12g	0.30g	0.10g	0.16g	0.14g	0.14g	0.20g
2002 USGS Maps	PGA 10% PE 50y	0.046g	0.120g	0.034g	0.070g	0.043g	0.060g	0.100g
	PGA 2% PE 50y	0.12g	0.40g	0.10g	0.20g	0.12g	0.16g	0.30g
	S_s 2% PE 50y	0.28g	0.80g	0.20g	0.60g	0.28g	0.36g	0.60g
	S_1 2% PE 50y	0.12g	0.20g	0.10g	0.16g	0.12g	0.12g	0.16g

The maps in current bridge design requirements contained in *Division I-A* of the *AASHTO Standard Specifications* were developed by the USGS in the 1970's and

updated in 1988 and they are comparable in respect of having the same mean return period with the 1996 and 2002 PGA USGS maps for 10% probability of exceedance in 50 years (Figures 3-15 and 3-20). The impact of the knowledge that has been accumulated about the New Madrid and Wabash Valley Seismic Zones is evident in the more detailed acceleration values reported. The values of PGA for the same return period in the new 1996 and 2002 maps are lower for Bloomington, Indianapolis, Japer, New Albany and Terre Haute, and higher for Evansville and Vincennes. This is consistent with the reported new understanding of the tectonic process of the Wabash Valley Seismic Zone and if the new maps were made part of *Division I-A* of the *AASHTO Standard Specification* the impact on the design of new bridges in Indiana would be marginal if not less demanding in many cases.

In order to assess the impact of adopting the proposed *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] as seen solely from the design acceleration point of view the following aspects come into play: (a) a change from a 475-year to a 2500-year return period design earthquake, (b) a change from peak ground acceleration (effective or accelerographic reported) description of the design ground motion to be routinely converted into a design spectrum to a new direct definition of the design spectrum, (c) site effects that amplify design acceleration in a different way including now short period accelerations along with long period ones both of them specially affecting the low acceleration range where precisely Indiana sites are located, and (d) use of a “frequent” earthquake for serviceability assessment.

The pertinent comments will follow in the same order listed. Although, the research presented in this report deals mainly with existing bridges — the reference for any bridge upgrade has been customarily related to current or in the process of being adopted requirements for new bridges — the approach adopted minimizes the dependence on the seismic risk assessment and focus on the expected behavior for increasingly severe ground motions without putting a tap on them.

(a) *Impact of change of design event return period* — It is debatable that with a relatively short-timed documented record of events proper extrapolations can be performed. Extrapolation is common in other civil engineering disciplines with

acceptable results from the point of view of proper design parameters and public awareness of the problems involved in distinguishing between acceptable risk and forecast. Flood control is a good example of the differences involved and the acceptability of the design parameters involved. On the other hand, both extreme wind design and earthquake resistant design have come into public scrutiny because of the gap between code design parameters and reported measured values. This has brought a trend to define design parameters in the same order of magnitude of the reported measured values. The basis for this change is well documented in the related literature and the real impact on the design procedures has been moderate because the reference to actual behavior during catastrophic events is the judging parameter in most cases. The new seismic risk maps presented in the USGS 1996 and 2002 maps describe a Maximum Considered Earthquake (MCE) that is supposed to produce measurable parameters such as peak ground acceleration of the same order of magnitude of those that may be recorded in the occurrence of a catastrophic event. How these values are made compatible with acceptable design parameters — from the economic and code requirements perspectives — have been solved in different ways.

The building design requirements were the first to adopt this approach [NEHRP, 1997]. The MCE was accepted but the design ground motion was defined as $2/3$ that produced by the MCE. The end result was design parameters that did not differ much from what was being used. The proposed *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] does not introduce the same reduction, although changes the accepted Response Modification Factors (R) to higher values that may bridge the difference (please compare values listed in Tables 2-3 and 2-8). Other subtle difference between the approach for buildings and for bridges is that in the case of site specific studies the building requirements require that the deterministic spectrum be obtained from 150% the mean of the spectra computed for characteristic maximum magnitude earthquakes while for bridges it requires the median-plus-one-standard-deviation which leads to 184% of the mean spectrum. One important conclusion is that in the future the basic seismic risk design factors will be different for buildings and for bridges although based on the same seismic risk maps.

(b) *Impact of change from PGA to spectral description* — The description of the design ground motion through a response spectrum was formalized in the ATC 3-06 project [ATC, 1978] based on the work of N. Newmark. The spectrum description was based on two parameters defined as Effective Peak Acceleration (EPA) and Effective Peak Velocity (EPV) that were related to the spectral ordinates as shown in Fig. 3-25 with EPA made equal to parameter A_a and EPV proportional to A_v . This procedure was the one adopted in ATC 6 for bridges and later in *Division I-A* of the *Standard AASHTO Specification* and is the basis of current 17th edition with the acceleration parameter A corresponding to the mapped value of A_v of the 1988 maps[NEHRP, 1988] by the USGS [Algermissen et al., 1990].

Fig. 3-25 – $EPA = S_a/2.5$ and $EPV = S_v/2.5$ obtained from a response spectrum with 5% damping as prescribed by ATC 3-06 [ATC, 1978]

The use of two different parameters to define the design response spectrum one based on acceleration and the other based on velocity was a wise decision because attenuation with distance for the two parameters is not the same. The 2.5 proportional constant was developed by Newmark based on the study of many accelerograms and their corresponding response spectrum and in general works well for most records but the commentary of the ATC-3-06 made it clear that effective peak acceleration and effective peak velocity were different from the peak acceleration in the accelerogram, and the peak velocity obtained by integration of the accelerogram.

The proposed *Draft AASHTO LRFD Seismic Provisions* moves away from this definition of the design ground motion and directly states the design spectrum severing the link with the effective peak ground acceleration and velocity, although it states a way to obtain the peak ground acceleration as $0.4S_{DS}$ as was shown in Fig. 2-21 thus apparently keeping the same 2.5 ratio. As shown in Table 3-4 for the seven Indiana sites, in the 2002 USGS maps the ratio between S_S and PGA for the same return period reports ratios between 2 and 3 with a mean of 2.27. Although these ratios are not fixed and will probably change in the future as new versions of the maps appear, the impact of moving from a PGA definition to a directly defined spectrum as the description of the design ground motion is minor.

Figures 3-26 and 3-27 show the combined impact of the change in return period and the spectral definition from the current *Division I-A* seismic design requirements response spectrum to the one required in the proposed *Draft AASHTO LRFD* requirements using the 2002 USGS maps.

Fig. 3-26 – Bridge Design Spectra in rock for Bloomington, Evansville, Jasper, and Vincennes in current *Division I-A* and proposed *Draft AASHTO LRFD* requirements

Fig. 3-27 – Bridge Design Spectra in rock for Indianapolis, New Albany, and Terre Haute in current *Division I-A* and proposed *Draft AASHTO LRFD* requirements

Table 3-5 – Analysis and Design requirements for selected cities in Indiana for the current *Division I-A* and proposed *Draft LRFD* requirements (spectral values in rock)

Specs	City (County) → Requirements ↓	Bloomington (Monroe)	Evansville (Vanderburgh)	Indianapolis (Marion)	Jasper (Du Bois)	New Albany (Floyd)	Terre Haute (Vigo)	Vincennes (Knox)
AASHTO Div. I-A	SPC (Essent./Other)	A/A	A/A	A/A	A/A	A/A	A/A	A/A
Proposed Draft LRFD	SHL	II	IV	II	III	II	III	III
	SDAP (Life S. /Oper.)	A2/ CDE	CDE/ CDE	A2/ CDE	BCDE/ CDE	A2/ CDE	BCDE/ CDE	BCDE/ CDE
	SDR (Life S. /Oper.)	2/3	5/6	2/3	3/4	2/3	3/4	3/4

Table 3-5 shows the implication from the point of view of the corresponding associated design analysis and detailing requirements presenting the Seismic Performance Category (SPC) (see Table 2-1) of the current requirements and for the

proposed Draft LRFD requirements the Seismic Hazard Level (SHL) (see Tables 2-5 and 2-6), the Seismic Design and Analysis Procedure (SDAP) (see Table 2-7), and Seismic Detailing Requirements (SDR) (see Tables 2-9 and 2-10).

(c) *Impact of change of site effects evaluation procedure* — The USGS 1996 and 2002 maps are defined at the B-C soil profile type interface (see Table 2-11) which is the dividing line between rock and very dense soil or soft rock. The soil amplification factors are both $F_a = 1.0$ and $F_v = 1.0$ for soil profiles type B. This is similar to the Soil Profile Type I of *Division I-A* of the current 17th Edition of the *Standard AASHTO Specifications* for which has a Site Coefficient $S = 1.0$ (see Fig. 2-16). Up to this point the procedures are similar. When other soil profile types come into play, the difference is large because the proposed *Draft AASHTO LRFD Seismic Provisions (NCHRP 12-49)* [NCHRP, 2002] has soil amplification factors F_a that affects the short period part of the design spectrum. AASHTO current Standard Specifications does not have a comparable soil dependent factor. Both F_a and F_v are acceleration dependent with comparatively larger values for lower accelerations (see Figures 2-24 and 2-25). Just for comparison purposes a site, common in Indiana, with more than 30 ft of soft to medium stiff soil under current *Division I-A* requirements would be a Type III soil profile and would have a Site Coefficient $S = 1.5$ that amplify the medium to long period part of the design spectrum. The same soil would probably be classified as a Type D Stiff Soil when using the proposed *Draft AASHTO LRFD Seismic Provisions*. Table 3-6 shows the values obtained for the cases being use for comparison.

For the short period portion of the design spectrum under the proposed *Draft Seismic Provisions* the amplification caused by the soil profile would be 18% to 60% percent higher than the current requirements with no amplification required. For the medium to long period portion of the design spectrum the values under the proposed Draft Seismic Provisions the amplification caused by the soil profile would be 33% to 60% percent higher than the current requirements. The reason for this large impact is linked to Indiana having mapped accelerations in the low range where the soil amplification effect is relatively larger.

Table 3-6 – Values of the site coefficient for a Type III soil profile under current *Division I-A* requirements and a Type D soil under proposed *Draft LRFD Provisions*

Specs	City (County) → Parameter Presented ↓	Bloomington (Monroe)	Evansville (Vanderburgh)	Indianapolis (Marion)	Jasper (Du Bois)	New Albany (Floyd)	Terre Haute (Vigo)	Vincennes (Knox)
AASHTO Div. I-A	A (PGA) 10% PE 50y	0.09g	0.09g	0.06g	0.09g	0.06g	0.06g	0.09g
	Site Coefficient S	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Draft AASHTO LRFD plus 2002 USGS Maps	S_s 2% PE 50y	0.28g	0.80g	0.20g	0.60g	0.28g	0.36g	0.60g
	Soil Type D Factor F_a	1.58	1.18	1.60	1.32	1.58	1.51	1.32
	S_1 2% PE 50y	0.12g	0.20g	0.10g	0.16g	0.12g	0.12g	0.16g
	Soil Type D Factor F_v	2.32	2.00	2.40	2.16	2.32	2.32	2.16

Figures 3-28 and 3-29 show the combined impact of the change in return period, the spectral definition and the soil amplification requirements for a Type III soil profile from the current Division I-A seismic design requirements design spectrum to the one required in the proposed Draft AASHTO LRFD requirements for a Type D profile using the 2002 USGS maps.

Table 3-7 shows the implication from the point of view of the corresponding associated design analysis and detailing requirements presenting the Seismic Performance Category (SPC) (see Table 2-1) of the current requirements and for the proposed Draft LRFD requirements the Seismic Hazard Level (SHL) (see Tables 2-5 and 2-6), the Seismic Design and Analysis Procedure (SDAP) (see Table 2-7), and Seismic Detailing Requirements (SDR) (see Tables 2-9 and 2-10).

Fig. 3-28 – Bridge Design Spectra in soil for Bloomington, Evansville, Jasper, and Vincennes in current *Division I-A* and proposed *Draft AASHTO LRFD* requirements

Fig. 3-29 – Bridge Design Spectra in soil for Indianapolis, New Albany, and Terre Haute in current *Division I-A* and proposed *Draft AASHTO LRFD* requirements

It is evident from the values presented for rock and for soil that the impact of the proposed Draft LRFD requirements is significant both for the design forces and for the analysis and detailing requirements. Under the current Division I-A requirements the analysis and detailing are the simplest (SPC-A) while for the proposed Draft LRFD the most strict covered (SHL-IV) are required for Evansville and Vincennes in both cases (rock and soil) studied and for Jasper in soil. In no case the simpler (SHL-I) is required.

The implications from the point of view of complexity of design and cost of construction are significant.

Table 3-7 – Analysis and Design requirements for selected cities in Indiana for the current *Division I-A* and proposed *Draft LRFD* requirements (spectral values in soil)

Specs	City (County) → Requirements ↓	Bloomington (Monroe)	Evansville (Vanderburgh)	Indianapolis (Marion)	Jasper (Du Bois)	New Albany (Floyd)	Terre Haute (Vigo)	Vincennes (Knox)
AASHTO Div. I-A	SPC (Essent./Other)	A/A	A/A	A/A	A/A	A/A	A/A	A/A
Proposed Draft LRFD	SHL	III	IV	II	IV	III	III	IV
	SDAP (Life S. /Oper.)	BCDE/ CDE	CDE/C DE	A2/ CDE	CDE/ CDE	BCDE/ CDE	BCDE/ CDE	CDE/ CDE
	SDR (Life S. /Oper.)	3/4	5/6	2/3	5/6	3/4	3/4	5/6

(d) *Impact of use of “frequent” earthquake* — The proposed *Draft AASHTO LRFD Seismic Provisions* introduces a frequent earthquake to be used in verification of serviceability of the bridge. Figure 3-19 shows the value 0.06g obtained for S_S at Vincennes for this earthquake based on results of the 1996 USGS maps. The impact of using such a low spectral acceleration for verification of serviceability will be probably negligible.

CHAPTER 4 METHODOLOGY FOR EVALUATION OF SEISMIC VULNERABILITY OF THE INDIANA EMERGENCY ROUTES

4.1 Route Alternatives

U.S. Congress and the Federal Administration through the FHWA have defined priority and critical routes in different instances. One such example is the National Truck Network established in 1982 by the Surface Transportation Assistance Act in order to designate a national network of highways that would allow the passage of trucks of specified minimum dimensions and weight. Figure 4-1 shows the National Truck Network in southwestern Indiana.

Fig. 4-1 – National Truck Network in southwestern Indiana

Corridor Network connectors linking major military installations and other defense-related facilities to the Strategic corridors. The fifth component are important arterial highways that serve interstate and interregional travel and that provide connections to major ports, airports, public transportation facilities, and other intermodal facilities.

Fig. 4-3 – Congressional High Priority Corridors

The National Highway System and its role in determining geometric design of highways is discussed in Section 40-1.03 (01) of Chapter 40 "Basic Design Controls" of the Indiana Department of Transportation's Design Manual.

Figure 4-4 shows the highways designated in Indiana as part of the National Highway System. It includes in the north-south direction: SR-69 from I-64 to Mount Vernon in the Kentucky border (Ohio River), US 41 from Terre Haute to Evansville and across the Ohio River to Kentucky, I-164 from I-64 to Evansville, SR-57 from the intersection with SR-67/US-231 to I-64, US-231 from the intersection with SR-67 to Kentucky border

One more recent highway system definition is the National Highway System. The National Highway System Designation Act of 1995 selected 264,000 km of roadways as the National Highway System (NHS) including 4,750 km in Indiana. Figure 4-2 shows the whole National Highway System.

Fig. 4-2 – National Highway System

NHS consists of five parts. The first component is the Interstate Highway System, which accounts for almost 30 percent of NHS. The second component includes 21 congressionally designated high-priority corridors (see Fig. 4-3), three of which include highways in Indiana as part of them. The third component is the non-interstate portion of the Strategic Highway Corridor Network identified by the Department of Defense in cooperation with the Department of Transportation. These corridors and the interstate highways are critical strategic links. The fourth component is major Strategic Highway

(Ohio River) south of Rockport, and SR-67 from Indianapolis to the intersection with SR-57. In the east-west direction includes US-50 from the Ohio border to Vincennes in the Illinois border (Wabash River), I-64 for the Ohio border to Illinois border (Wabash River), SR-66 from Rockport on the Kentucky border (Ohio River) to Evansville, and SR-62 from Evansville to Mount Vernon on the Kentucky border (Ohio River).

Fig. 4-4 – Highways in Indiana designated as part of the National Highway System

From the methodological point of view it is evident that the routes in southwestern Indiana contained in both the National Truck Network and the National Highway Network had to be studied as candidates for the Earthquake Emergency Routes of Indiana.

Part I of this report presents different alternatives for Earthquake Emergency Routes for the State of Indiana based on trade off of three parameters, namely total travel time, population covered, and number of square feet of bridge deck (as a parameter associated with eventual retrofit cost).

The Indiana Department of Transportation provides maintenance within the State to Interstate Highways, U.S. Routes, and Indiana State Highways. County roads and city roads are excluded unless special agreements are in effect. The grid of routes presented in Fig. 4-5 corresponds to routes that belong to the National Truck Network, the National Highway System, and are within the domain of the Indiana Department of Transportation, and is presented as the preferred choice of Earthquake Emergency Routes of Indiana.

Fig. 4-5 – Selected grid of Earthquake Emergency Routes in the Indot Vincennes District

The grid of routes presented in Fig. 4-5 comprises the following:

- US 41 from Terre Haute South to Evansville and the Kentucky border (Ohio River).
- SR 67 from Freedom to SR 57 junction and SR 57 from SR 67 to I-64.
- I-164 from I-64 to US 41.
- US 231 from SR 54 at Bloomfield south to Rockport and to Kentucky border (Ohio River).
- SR 37 from Bloomington south to SR 237 and SR 237 to Cannelton and the Kentucky border (Ohio River).
- SR 154 from Illinois border (Wabash River) to US 41, and SR 54 from US 41 east to SR 45.
- US 50 from Illinois border (Wabash River) at Vincennes east to SR 446 east of Bedford.
- SR 64 from Illinois border (Wabash River) to Princeton to US 41.
- I-64 from Illinois border (Wabash River) to SR 66 at Carefree.
- SR 57 from US 41 past Evansville Airport to I-164.
- SR 62 from Illinois border (Wabash River) thru Evansville to US 231
- SR 66 from US 41 east to Rockport.

4.2 Available Information

The Indiana Department of Transportation has a very well organized data base where information for the purposes of present project was obtained. The information used was the following:

- *Indot Inspection and Maintenance Database* – This database was modernized just before the initiation of the project. It consists in a MS Access database that contains records of all bridges being maintained by Indot. It follows Federal Highway Administration guidelines [FHWA, 1996] and contains additional information beyond the FHWA requirements. Only the portion related to bridges

in the Vincennes Indot District was used containing 827 bridges, distributed by Counties as shown in Table 4-1. See Fig. 4-6 for their location.

Table 4-1 – Distribution by County of Vincennes District bridges contained in Indot Inspection and Maintenance Database

County	No. Vincennes District Bridges in Database
Crawford	35
Daviess	22
Dubois	50
Gibson	66
Greene	41
Knox	42
Lawrence	36
Martin	51
Monroe	1
Orange	27
Owen	60
Perry	62
Pike	63
Posey	65
Spencer	60
Sullivan	40
Vanderburgh	121
Vigo	84
Warrick	72
Washington	1
Total	827

- *Bridge Drawings* – The bridge drawings at the Indot Vincennes District offices were consulted for bridges located on the selected Earthquake Emergency Routes. The drawings contain the original design plans, those corresponding to changes and modifications, and a description of the soil boring performed at the time of design of the bridge reporting SPT values in many cases.
- *GIS Information* – Geographical Information System (GIS) related to the State of Indiana, and other information was supplied by Indot. This information can also be downloaded from the following Indiana University webpage: <http://igs.indiana.edu/arcims/southwest/download.html>

Fig. 4-6 – Vincennes District bridges contained in Indot Inspection and Maintenance Database (those whose drawings were used in this research are marked as blue filled dots)

4.3 GIS Implementation

A two tier approach was developed for managing the information described in the previous section. The first stage consisted in developing tools for assessing the vulnerability condition of the studied Earthquake Emergency Routes. The second stage consisted in implementing the results in a dynamic database with Geographical Information Systems (GIS) capabilities.

For the first stage a database was developed as a search-engine to allow the static and dynamic evaluation of the condition of the selected critical routes prior to and following an earthquake disaster. Several specialized software programs were developed to act as

independent units — all of them interacting with the database as needed — for evaluating the different sources of vulnerability along the Earthquake Emergency Routes.

Three modules were developed, as shown in Fig. 4-7. The first module corresponds to a *Data Management Module*. This module handles the information related to the routes, and bridges. The information included is the one described in the previous section obtained from Indot Inspection and Maintenance Database.

Fig. 4-7 – Organization of the GIS implementation

The *Pattern Recognition Module* uses the information contained in the Data Management Module to process and obtain vulnerability assessment of bridge structures and geotechnical aspects of the bridges and routes and was used to feed information to the Scenario Management Module.

The *Scenario Management Module* is implemented on the extraordinary capabilities of HAZUS software developed by the Federal Management Agency [FEMA, 2005] that is capable of accepting the feed-back of the results obtained from the Pattern Recognition Module. HAZUS is capable of obtaining vulnerability assessments for any case in particular for mitigation purposes, allows simulations for different earthquake occurrence scenarios, and will allow in the future to be used when online capabilities are implemented within Indot for monitoring response to an actual earthquake occurrence.

4.4 Vulnerability Assessment Approach

The assessment of seismic vulnerability for a significant number of structures requires the use of identification of patterns of behavior of these structures to the envisioned earthquake ground motion. The approach used in comparable previous cases has been to define groups of structures having similar properties or configuration for which common behavior patterns can be defined. These behavior patterns have been referred in the literature as fragility curves relating probability of having a particular type of damage to a ground motion descriptor such as peak ground acceleration.

For implementation in this project several shortcomings were identified in using this approach:

- The inventory of bridges under study covers from bridges recently built to bridges built in the 1910's and 1920's. Figure 4-8 shows the year of construction of the Indot Vincennes District bridges. Establishing general fragility curves giving probability that a certain type of damage occurs for such a diverse inventory lead to many types of fragility curves and their use in establishing actual vulnerability may be questionable.
- Many of the bridges have been rebuilt (except bridges built after 1990) at different moment. Figure 4-9 shows the percentage of the Vincennes District bridges that have been rebuilt, and the average year when the rebuilding took place. The influence of such a significant percentage of bridges having been rebuilt makes the fragility curve type of approach even more difficult to apply.
- Associating the bridge vulnerability of a set value of seismic risk assessment as presented in a specific map, for example the one contained in Current 17th Edition of the Standard AASHTO Specifications, will require a new assessment if the map contained in the requirements changes. The adoption of new more modern specifications will surely take place in the near future, but the nature of the changes that will be introduced before adoption are unknown presently. The adoption of the proposed Draft LRFD Seismic Requirements has been voted

down by several states who have questioned the complexity and eventual unneeded extra cost that their adoption will bring.

Fig. 4-8 – Number of bridges built per decade (Vincennes District)

Fig. 4-9 – Percentage of bridges rebuilt according to decade of construction and average year when rebuilding was made (Vincennes District bridges)

Based on these limitations a different approach was implemented in this study. The approach used is based in selecting a number of bridges to study their seismic behavior under varying earthquake ground motion severity thus making the study independent of current requirements and making the results valid under mapped values having different return period or acceleration (peak ground or spectral) values. Using the results from the study of these bridges a procedure to extrapolate the results to the rest of the bridges in the Vincennes District was devised, requiring as information only that contained in the Indot Inspection and Maintenance Database. This establishes two levels of confidence on the vulnerability assessment. The first one is general and covers all bridges in the Inspection and Maintenance Database. The second one is more detailed because is based on the information contained in the bridge drawings. The advantage of this approach is that it permits to gage the vulnerability of the whole bridge inventory. It permits devising policies of general nature with respect to the selection, or variation in the future, of the Earthquake Emergency Routes. It also permits a more detailed assessment by supplying additional information to the GIS system data base, in case the detailed data is not already in the system. A detailed explanation of the procedure is given in Chapter 6.

4.5 Mitigation and Simulation

Vulnerability mitigation policies that cover the bridge inventory in southwestern Indiana are possible using the HAZUS GIS system. The assessment of the impact of a variation of the seismic risk maps or the simulation of occurrence of an earthquake with specific location and magnitude can be easily achieved by matching the new values to the already computed curves relating the bridge expected damage to the level of acceleration. If a bridge is retrofitted, the changes introduced to the bridge substructure or superstructure can be introduced to the database for future assessments or simulations.

CHAPTER 5 GEOTECHNICAL ASSOCIATED SOURCES OF VULNERABILITY

5.1 Introduction

This chapter describes the methodology implemented for processing the geotechnical information available for the selected bridges and to be used, in the future, as the database is expanded by adding more information from drawings or from soil explorations. The Inspection and Maintenance Database does not contain geotechnical information; therefore, the approximate vulnerability assessment does not include evaluation of the geotechnical sources of vulnerability.

Not all the studied bridge drawings contained soil exploration boring information. In the majority of cases studied the boring logs included in the drawings contained the soil description for each layer and reported the Standard Penetration Test (SPT) values at appropriate intervals. In rock sites many of the boring logs report the recovery percentage appropriately. In some instances the boring logs only report the soil classification for each layer without any SPT values. Textural description of the soil is the accepted practice for defining the soil classification in the field by the boring team to be reported in the boring log. In general, especially for older borings, the description is based on the U.S. Department of Agriculture textural classification chart [USDA, 1951] presented in Fig. 5-1. Percentages of sand, silt, and clay are established visually based only in the particle size. The soil classification description used corresponds to the one in the zone of the figure where the three percentage values meet. No laboratory procedures are involved in assigning the soil layer description. The AASHTO Soil Classification System or the Unified Soil Classification (ASTM D2487 — “Standard Classification of Soils for Engineering Purposes — Unified Soil Classification System”) require laboratory tests and are routinely performed on the soil samples and included later in the soils report.

Fig. 5-1 – USDA Textural Soil Classification Chart [USDA, 1951] routinely used for soil classification by soil exploration boring teams

The software implementation uses the soil exploration boring reports contained in the selected bridge drawings to study the geotechnical sources of vulnerability for each of the bridge sites. Other sites can be studied by adding information to the database in the future.

Two evaluations are performed using the boring information: a soil profile amplification assessment and a liquefaction potential assessment. These evaluations are made for varying peak ground accelerations in rock to be consistent with the adopted approach in this research project of obtaining the vulnerability for different levels of ground motion at the site and not to depend on a unique description of ground motion as contained in a particular set of seismic risk maps. Soil profile amplification and liquefaction potential are acceleration dependent; therefore, the values obtained cover different seismic risk levels.

The possibilities of evaluation are presented in the main screen of the software developed for this purpose, as shown in Fig. 5-2.

Fig. 5-2 – Main screen for the liquefaction evaluation module developed

This module permits to enter, save, and retrieve boring information for any bridge site borings, and to process it to obtain the soil profile amplification potential, evaluate the liquefaction susceptibility, and to save to results for further analysis.

5.2 Soil profile amplification

The amplification potential of the soil profile is needed for the liquefaction evaluation and for the bridge vulnerability assessment; in the former case for estimating the ground motion acceleration at the surface and in the later case for defining the ground motion description for evaluation of the bridge. The methodology used is based on the procedure contained in the 2000 NEHRP Provisions [NEHRP, 2000] as described in Section 2.3.1 of this report.

The first step is to define the site class (A to F) as presented in Table 2-11. The classification is made using average shear wave velocity ($\bar{v}_s$), average standard penetration resistance ($\bar{N}$), average standard penetration resistance for the cohesionless

soils only ($\bar{N}_{ch}$), and average undrained shear strength in case of cohesive soils ($\bar{s}_u$). The software implementation is capable of using the more appropriate parameter from the soil exploration and laboratory reports. For the selected bridges because only Standard Penetration Test are reported the amplification potential is evaluated from the average standard penetration resistance ($\bar{N}$) computed for the upper 100 ft (30 m) of the site profile using Eq. (5-1). Profiles containing distinctly different layers are subdivided into those layers by a number that ranges from 1 to n .

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (5-1)$$

where d_i is the thickness of any layer between 0 and 100 ft (30 m) and N is the Standard Penetration Resistance (SPT) not to exceed 100 blows/ft as directly measured in the field without corrections.

Once the site is classified as being Class A to E (Class F requires a site specific evaluation) the curves presented in Figures 2-24 and 2-25 are used to obtain the amplification parameters F_a and F_v for the acceleration value under study. Spectral acceleration (S_s or S_l) values are used directly to obtain F_a and F_v . If peak acceleration in rock is used (A) the value is multiplied by 2.5 to be used in Fig. 2-24 to obtain F_a and is used directly in Fig. 2-35 to obtain F_v .

The values obtained were compared with results using the program SHAKE as described for several sites in Indiana in [Bobet et al., 2001] obtaining good correlations of the amplification levels obtained.

Figure 5-3 shows the results obtained for a specific case. The figure reports that in this case (Bridge No. 41-42-04638) the bridge drawings included four borings. It also indicates that the procedure classified the soil profile in all borings as being Type E. The statistics shown in the screen indicate the number of borings that report a certain soil profile type. As opposed to buildings, where usually a single soil profile type is present, bridges by being extended structures may have different soil profile types under the abutments or at the bridge piers. This is specially true in alluvial plain sites, where many

of Indiana bridges are sited. Figure 5-3 also shows in graphic form the relationship between the horizontal acceleration in rock and the horizontal acceleration in surface for peak ground acceleration, or spectral ordinates (S_s and S_1).

Fig. 5-3 – Typical results for soil profile classification and amplification potential

In total 182 bridge sites located on the defined Earthquake Emergency Routes were evaluated. This includes all the bridges located on the Emergency Routes whose drawings contained soil boring records shown as part of them. The information was used directly to define the soil profile type for each boring using the procedure described. When reporting the soil profile type that causes the maximum amplification the results shown in Fig. 5-4 are obtained indicating that for 159 bridges out of 182 (87%) the soil profile is classified as Type E (see Table 2-11 for description of the soil profiles). Soil profile Type D is obtained in 16 bridges (9%), soil profile types (B and C) just carry 7 bridges (4%), and none are classified in soil profile Type A. These results are hardly surprising for the State of Indiana where very few rock type sites are found.

Notwithstanding, this situations has important implications from the point of view that mild earthquakes may produce moderate to high peak ground accelerations on surface due to the soil amplification caused by the intervening soil responding essentially in a elastic manner without the possibility of a decrease caused by the damping inherent in the nonlinear response of the soil ever occurring.

Fig. 5-4 – Worse soil profile type at bridge site for 182 bridges on the Earthquake Emergency Routes

5.3 Liquefaction

The liquefaction potential is evaluated using the procedure described in Section 2.3.2 of this report. The software implementation accepts information from field test in the form of cone penetration test (CPT), standard penetration test (SPT), or shear-wave velocity (V_s) measurements. The application of the procedure using the boring information available was restricted to evaluations using Standard Penetration Test values, but as more information is included in the database it is desirable to use more reliable field information such as CPT or shear wave velocity measurements for evaluations in the future.

For the selected bridge evaluation corrections, as described in Section 2.3.2, for fines content and earthquake magnitude were implemented. The corrections to SPT listed in Table 2-12 were implemented but not used (except overburden pressure and rod length corrections that were used in all cases) due to lack of information on the boring equipment and procedures as reported in the bridge drawings. As the database is expanded in the future the software implementation permits the use of these corrections if reported in the boring log.

Fines content in a soil can have a significant influence in the liquefaction potential of a soil layer. The procedure to account for the presence of fines in the evaluation of the liquefaction potential is to adjust the value of the corrected blow count $(N_1)_{60}$ using Eq. (2-22) to an equivalent value for clean sand, $(N_1)_{60cs}$. The effect of the adjustment is an increase of the value of $(N_1)_{60}$ effectively used. It is considered [Youd and Idriss, 2001] that clean granular soils having a value of $(N_1)_{60}$ greater than 30 are too dense to liquefy and are classed as non-liquefiable. For most soil classification procedures, as shown in Fig. 5-5, fines are considered particles that can pass through a No. 200 sieve corresponding to a 0.08 mm size particle.

Fig. 5-5 – Soil particle size according to several soil classification procedures [Winkerkorn and Fang, 1975]

Boring reports as presented in bridge drawings do not include the results from the sieve analysis. In order to adjust the fine content of the soils as reported in the boring logs the percentage of silt and clay was determined from the soil classification reported in the borings using the chart presented in Fig. 5-1 or alternatively for some soil descriptions not contained there using the values suggested by the Federal Aviation Administration (FAA) classification as shown in Fig. 5-6. The implemented software permits editing the value of the fines content for any soil layer when needed.

The influence of the fines content in the evaluation of the liquefaction potential cannot be underestimated and is recommendable that in the future, as more sites are included in the database, that fine content be reported from sieve analysis of the soil.

Fig. 5-6 – FAA textural classification of soils [Winkerkorn and Fang, 1975]

When evaluating liquefaction potential the expected number of cycles needed for the pore pressure increase development has been traditionally associated with the causing earthquake magnitude. The developers of the liquefaction evaluation procedure used the moment magnitude scale — M_w . Magnitude for the 1811-1812 New Madrid events has been estimated using different techniques, reporting values that range from a lower value

of 6 to an upper value of 9. The magnitude scale in which these values have been reported is uncertain and in some cases is not reported.

The preference for moment magnitude is based on being the most consistent scale along a wide magnitude range, but it is the scale that requires more instrumental information to define its assigned value. Recent studies made by the USGS [Frankel et al., 2002] for the 2002 seismic risk maps update have set a maximum characteristic magnitude for the central and eastern U.S. for purposes of seismic risk assessment. The scale in which the characteristic magnitude is expressed is moment magnitude and has direct use in liquefaction potential evaluation.

Figure 5-7 shows the maximum characteristic moment magnitude assessed for the central and eastern U.S. A value of $M_w = 6.5$ is given for all the state of Indiana, except the Wabash River Valley that is assigned a value $M_w = 7.5$. JTRP Project SPR 2812 on Analysis of Seismic Hazard Assessment for Indiana being currently conducted by Purdue Professors Haase and Nowack will surely give more insight on the values of M_w to be used and implemented in future liquefaction potential assessments. The software developed permits the variation of the magnitude to use.

Fig. 5-7 – Maximum magnitude for the Central and Eastern U.S. [Frankel et al., 2002]

It is interesting to note that if the proposed Draft AASHTO LRFD seismic requirements are adopted in Indiana, for the seven Indiana cities evaluated for comparison purposes in section 3.4.6 of this report any new bridge design would require liquefaction evaluation for the bridge site. Table 2-13 indicates the cases in which no liquefaction evaluation is needed for Seismic Detailing Requirements (SDR) 3 to 6 that require in all cases magnitudes lower than 6.4.

Typical results screens from the implemented software are shown in Figures 5-8 to 5-10. In Fig. 5-8 the description of the boring is given reporting the soil layers and the Standard Penetration Test (SPT) values and the water table level.

Fig. 5-8 – Example description of boring log as presented in the implemented software

In Fig 5-9 the safety factor for liquefaction is given. The software first searches for the lowest surface ground acceleration that will produce liquefaction — in this case it is 15% of the acceleration of gravity g — and reports the safety factor along the soil profile. In this case liquefaction occurs in a sand layer that reports a SPT value of 17 blows per foot at elevation 410. The peak ground acceleration in rock that produces the acceleration in surface leading to liquefaction corresponds in this case to 6% g . The soil profile reported by this boring is classified as being Type E and for a peak rock acceleration of 6% g the amplification factor, F_a , has a value of 2.5 thus converting the 6% g acceleration in rock into 15% g acceleration in surface.

Fig. 5-9 – Example description of liquefaction safety factor for the lowest ground surface acceleration inducing liquefaction. In this case 15% g leads to a safety factor of 0.9 meaning that liquefaction would occur

Different peak ground accelerations in rock can be evaluated. Figure 5-10 shows the results in the liquefaction potential as described by the liquefaction safety factor for peak ground acceleration in rock of 4%g leading to a ground surface acceleration of 10%g the safety factor against liquefaction increases to a value of 1.4 thus reporting no liquefaction. In this manner the safety factor is obtained for different acceleration scenarios.

Fig. 5-10 – Example description of liquefaction safety factor for a different ground surface acceleration from the one shown in Fig. 5-9 not inducing liquefaction. In this case the lowest safety factor for 10% g surface acceleration is 1.4

The factor of safety for liquefaction for a range of ground surface acceleration is stored in the database. This permits to handle different seismic risk scenarios without having to reprocess the boring information in each instance.

Bridge 050-42-06963 reported in the previous figures (Fig. 5-8 to Fig. 5-10) has description for four borings included in the drawings. Borings 1 and 3 are classified as being Soil Profile Type D while borings 2 and 4 are classified as Soil Profile Type E. The peak ground acceleration in rock required to produce liquefaction obtained were 6% g for borings No. 1, 2, and 4 and 4% g for boring No. 3.

The situation just described is present in many of the 182 bridges studied along the Earthquake Emergency Routes. The evaluation was performed for two values of M_w ($M_w = 6.5$ and $M_w = 7.5$). Table 5-1 presents a summary of the minimum peak ground acceleration in rock required to produce liquefaction at bridge sites studied. The information is presented for the two moment magnitude studied.

Table 5-1 – Minimum peak ground acceleration in rock required to produce liquefaction on the 182 bridges sites studied along the Earthquake Emergency Routes

PGA rock (g)	No. of Bridges			
	$M_w=7.5$	%	$M_w=6.5$	%
0.02	2	1.1%		0.0%
0.03	27	14.8%	3	1.6%
0.04	23	12.6%	18	9.9%
0.05	10	5.5%	24	13.2%
0.06	3	1.6%	10	5.5%
0.07	2	1.1%	7	3.8%
0.08	1	0.5%	2	1.1%
0.09	4	2.2%	2	1.1%
0.10	1	0.5%	1	0.5%
0.11	2	1.1%		0.0%
0.12	1	0.5%	1	0.5%
0.13	1	0.5%	2	1.1%
0.14		0.0%	2	1.1%
0.15		0.0%	1	0.5%
0.16		0.0%		0.0%
0.17	1	0.5%		0.0%
0.18	1	0.5%	2	1.1%
0.28		0.0%	1	0.5%
0.42		0.0%	1	0.5%
0.48		0.0%	1	0.5%
No liquefaction	103	56.6%	104	57.1%
	182		182	

With the minimum peak ground acceleration in rock needed to produce liquefaction at the bridge site it is possible to have a general picture of the vulnerability caused by liquefaction for the bridges along the Earthquake Emergency Routes. This information is presented in Fig. 5-11. In this figure the percentage of bridge sites that may be affected by liquefaction when the peak ground acceleration in rock reaches a certain value may be obtained. At peak ground accelerations in rock of the order of 10% g the curve flattens, meaning that the number of bridges sites susceptible of liquefaction would remain essentially the same even for higher accelerations. At this level of acceleration in rock approximately 40% of the bridges located along the Earthquake Emergency Routes would have reported liquefaction. This acceleration is of the same order of magnitude of those contained in current *Division I-A of AASHTO Standard Specifications*.

Fig. 5-11 – Percentage of bridges along the Earthquake Emergency Routes that will not have liquefaction at the site for a given peak ground acceleration in rock computed using the minimum acceleration to produce liquefaction at the site

Liquefaction potential was computed at all borings for each of the studied bridge sites. Only in few cases the minimum acceleration needed to produce liquefaction is the

same for all borings. If instead of reporting the minimum peak ground acceleration in rock the average acceleration for those borings reporting liquefaction is computed, the vulnerability situation changes. Figure 5-12 shows the information computed for the average peak ground acceleration in rock to produce liquefaction at the site. The curves shown flatten at a higher acceleration (15% g) but show a similar trend. Using the average peak ground acceleration in rock at the bridge site, the number of vulnerable bridges due to liquefaction at the level of acceleration of the current AASHTO Standard Specifications reduces to approximately 35% of the bridges on the Earthquake Emergency Routes.

Fig. 5-12 – Percentage of bridges along the Earthquake Emergency Routes that will not have liquefaction at the site for a given peak ground acceleration in rock computed using the average acceleration to produce liquefaction at the site

The primary concern when interpreting the data just presented is related to the difference in peak ground acceleration in rock values for the same bridge site. If the difference between the minimum accelerations to produce liquefaction and the mean acceleration for the same bridge site is relatively large, it indicates significant variations

in the soil profile recorded in the soil exploration borings for the same bridge site. The ratio of mean to minimum peak ground acceleration producing liquefaction at the same bridge site has values that range from 1 to 6 with an average of the order of 1.4 for the bridge sites studied, indicating significant variations in the soil profiles.

To emphasize the need for engineering judgment in interpreting the results presented, the following example brings out features found in many of the bridges located in the Earthquake Emergency Routes. Bridge 50-42-04625 on the Wabash River carries US Route 50 across the river to Illinois. The bridge drawings contain records of 24 soil borings made in 1960 all of them containing appropriate information (soil description and standard penetration test – SPT – logs) to evaluate the soil amplification potential and the liquefaction potential at each boring. The evaluation of the Soil Profile Type indicates that the profile at one boring would be Type B, at five borings Type D, and Type E at the remaining 18 borings. The general classification of the bridge site based on the softer profile would assign the bridge site as being Type E. The liquefaction evaluation for the borings report possibility of liquefaction in 11 out of the 24 borings for magnitude $M_w = 7.5$. Table 5-2 presents the peak ground acceleration in rock required for producing liquefaction at each of the borings reporting liquefaction. In the same table the minimum and maximum peak ground acceleration obtained is reported as well as the mean and standard deviation for the acceleration.

In order to interpret the liquefaction potential computation for the studied bridges besides the information presented, two other variables were studied:

- Percentage of borings reporting liquefaction of the total number of borings.
- Ratio of the range of peak ground acceleration in rock producing liquefaction to the mean peak ground acceleration.

For Bridge 50-42-04625 the percentage of borings reporting liquefaction was 58% and the ratio of the range of peak ground acceleration in rock producing liquefaction to the mean peak ground acceleration was $(0.36-0.05)/0.10 = 3.1$. These values were computed for all bridges reporting liquefaction and based on the values obtained a recommendation was formulated dividing the bridges studied into four groups designated as: bridges having no liquefaction, bridges with a high probability of liquefaction, bridges

with an appreciable probability of liquefaction, and bridges having liquefaction but were a wide variability of the results is present that require additional soil exploration and a more in depth liquefaction study performed by geotechnical consultants.

Table 5-2 – Minimum peak ground acceleration in rock required to produce liquefaction on the 11 borings that report liquefaction for Bridge 50-42-04625

Boring No.	Peak Ground Acceleration in Rock for liquefaction
14	8% g
15	8% g
16	9% g
17	7% g
18	6% g
19	6% g
20	8% g
21	36% g
22	5% g
23	7% g
24	7% g
Min. PGA_{rock}	5% g
Max. PGA_{rock}	36% g
Mean PGA_{rock}	10% g
Standard Deviation	9% g

Figure 5-13 presents the percentage of bridges studied that fall into the recommendation categories for the two moment magnitude studied.

Other considerations that are beyond present study are related to issues related to variation of the soil profile as used in the evaluation that may have occurred during the years since the borings were performed. Other important consideration to take into account is related to the water table at the site. Liquefaction can only occur in relatively loose granular soils located under the water table. The potential for liquefaction studied and reported used the water table level reported in the borings. The possibility of variations of the water table level due to seasonal effect with respect to the moment the borings were performed can affect the results presented. This is specially important for

those cases not reporting liquefaction due to a depressed water table in cases where loose granular soils exist at the site.

Fig. 5-13 – Recommendation for bridges located along the Earthquake Emergency Routes

Using HAZUS-MH [FEMA, 2005] — a GIS-based software tool developed by the Federal Emergency Management Agency (FEMA) — Indot bridge inventory was assessed for the liquefaction potential that was evaluated using the actual soil borings included in the bridge drawings, as described before. The Indot bridge data base was integrated within HAZUS-MH and the GIS functions were activated. The liquefaction potential was evaluated using HAZUS-MH own liquefaction potential evaluation routines. This was performed for the $M_w = 6.5$ earthquake scenario obtaining comparable result.

5.4 Soil spread

For a soil spread situation to occur during earthquake liquefaction has to occur. The soil spread scenarios are difficult to state and requires analysis and studies beyond what

can be implemented in application software of the type implemented in this project. In a previous JTRP project [Bobet et al., 2001] soil spread scenarios and consequences were studied for several sites in southwestern Indiana indicating the possibility of their occurrence. The study also indicated that the Indiana Department of Transportation practice of using steel H piles and steel encased concrete (SEC) piles reduces significantly the potential damage to piles during earthquakes. This study recommends mitigation studies and implementations that should be adopted for all bridge structures located along the Indiana Emergency Routes. A warning should be issued when liquefaction may occur and the bridge is supported on footings. The same type of warning is given when possibility of liquefaction is identified below the tip of piles, situation that seldom occurs.

5.5 Embankment stability

The absence of information on embankment slope geometry and description of mechanical properties of the fill material both in the Inspection and Maintenance Database and the selected bridge drawings precludes the inclusion of test cases for the Earthquake Emergency Routes. Notwithstanding, test cases studied for fill material commonly used in Indiana indicate that for embankments with slopes of 1 vertical to 2 horizontal there is a low probability of instability in the embankment itself for ground accelerations up to the order of 15% g and embankment heights lower than 30 ft (10 m). The possibility of liquefaction or other type of ground failure under the embankment must be studied independently along the routes. Information on soil profile under the embankment and properties of the fill material probably would require soil exploration for that purpose along the Emergency Routes.

CHAPTER 6 BRIDGE VULNERABILITY

6.1 Introduction

This chapter describes the methodology implemented for assessing the bridge seismic vulnerability by processing information available for all bridges located in the Vincennes District of the Indiana Department of Transportation and for selected bridges whose drawings were used. The information contained in the Indot Inspection and Maintenance Database was used directly for obtaining an approximate evaluation of the bridge seismic vulnerability. The drawings of selected bridges were used for obtaining a more reliable vulnerability assessment of each selected bridge. The results of the analysis of the bridges whose drawings were used to calibrate the approximate procedures based on the Inspection and Maintenance Database information.

6.2 Information from the Inspection and Maintenance Database

Indot Inspection and Maintenance Database is implemented in MS Access and contains information descriptive of the bridge that is relevant from the inspection and maintenance management point of view. Only part of the information contained in the database is related to structural information and although general in nature, lends itself to assign the bridge within broad seismic behavior categories.

The information from the database that was used directly in the approximate vulnerability assessment can be grouped in the following general categories: for database indexing purposes, relative to location, road under or over the bridge, date of construction and repair, general bridge geometry, superstructure characteristics including deck information, substructure characteristics, and information on the approaches and abutments. Table 6-1 presents the database keys that were used for processing the approximate vulnerability assessment.

Table 6-1 – Basic information from Indot Inspection and Maintenance Database
that was included in the analysis

Group	Information
Database	Number of Records
	Total No. of Bridge Records
	Record No.
File Data	National Bridge Inventory No.
	INDOT Bridge No.
	District Code
	County Code
	Sub-district No.
	City Town Code
	Location
Road Data	Facility Carried
	Feature Intersected
	Road Number Over
	Road Number Under
Age Data	Year Built
	Structure Designation
	Year Reconstructed
	Sufficiency Rating
Geometry Data	Number of Intermediate Piers
	Approach Structure Type
	Deck Width
	Skew Angle
	Structure Length
	Minimum Vertical Clearance Over
	Minimum Vertical Clearance Under
	Total Horizontal Clearance Over
Superstructure Data	Largest Vertical Distance
	Structure Type
	Length Maximum Span
	Number of Main Spans
	Deck Structure Type
	Deck Thickness
	Bearing Diaphragm
	Bridge Joint Type Interior
	Bridge Joint Type (N/E)
	Bridge Joint Type (S/W)
	Cross Bracing
	Hinge Pin Connection
	Intermediate Diaphragms
	Main Structure Type
	Number of Approach Spans
	Number of Beams
	Number of Floor Beams
	Number of Girders
	Number of Stringers
	Redundancy Code
Substructure Data	Abutment Type (W/S)
	Abutment Type (E/N)
	Approach Material
	Approach Construction

6.3 Approximate vulnerability assessment methodology

The approach to an approximate evaluation of vulnerability in many instances has been based just on the type of bridge. This methodological approach depends on general correlations for each bridge type, usually called fragility relationships, between probability of reaching a certain performance level (total collapse, partial collapse, repairable damage, or other) and expected ground acceleration at the bridge site. Results from this type of analysis are applicable, within the uncertainties inherent to this type of methodology, at a national or state level. As the size of the sample becomes smaller the results obtained are less reliable especially if diverse bridge types of varying age are present in the sample. In this research project it was considered that the information contained in the Inspection and Maintenance Database is appropriate to determine, in an approximate but more reliable manner, the relevant parameters that are needed to establish a vulnerability level. The relevant parameters considered are:

- Mass of the bridge
- Stiffness of the substructure
- Toughness available for adequate nonlinear response
- Strength of the lateral load resisting elements

The Inspection and Maintenance Database contains information from which the values of the relevant parameters can be estimated as follows:

The mass of the bridge is associated mainly with the mass of the superstructure and the deck is generally the main contributor. The Inspection and Maintenance Database lists the deck material, and deck width and thickness; thus permitting to obtain a good approximation of its mass. Allowances for wearing surface, barriers and, guard rails are also included. The number of girders, beams, and stringers; and the structural material is listed in the database. Their mass can be estimated as a function of a span dependent depth that is characteristic of each superstructure type. The contribution to mass of the diaphragms and bracing elements can also be estimated from the number and type as listed in the database. Having an estimative of the superstructure mass and geometry it is possible to establish minimum cross-sectional dimensions required to support the gravity effects for the bents based on the adjacent span lengths and structural type. Vertical

clearance indicates the height of the bents. Results from these estimates were compared with mass computed for the selected bridges whose drawings were used in the formal vulnerability assessment and with comparison of projects whose information was available in the literature.

The lateral stiffness of the bridge depends on the substructure type whose minimum dimensions were established in the computation of mass. The Inspection and Maintenance Database does not describe if the bridge is supported on frame or wall type bents, thus the estimate of the lateral stiffness is approached from the low side. Adjustments are made to this estimate for live load allowance especially in short span bridges.

Using the estimative of mass and lateral stiffness fundamental vibration periods for the transverse and longitudinal directions are computed taking into account the expansion joint and hinge pin connection descriptions contained in the database. Lateral force demand, longitudinal and transverse, is established for varying values of ground acceleration. Expected lateral deformation of the superstructure and base shear demand on the substructure is then estimated for each acceleration level.

The level of toughness required for adequate nonlinear response of the substructure elements is associated directly with the date of construction of the bridge. Lateral load strength is associated directly with the estimated substructure dimensions.

Bridge vulnerability is then established for each acceleration level based on excessive lateral deformations or lack of strength to resist the imposed base shear. For each acceleration level the bridge is red-tagged meaning vulnerable, yellow-tagged meaning marginal vulnerability, and green-tagged meaning not vulnerable.

These computations are made for all bridges in the Vincennes district inventory. For selected bridges the results were compared to the results obtained using the actual dimensions and reinforcement from the bridge drawings.

6.4 Formal vulnerability assessment methodology

The implemented software permits to include information from the bridge drawings in the vulnerability assessment database. This was performed for selected bridges as previously mentioned. The information covered is divided as shown in Table 6-2.

Table 6-2 – Basic information from bridge drawings needed for formal vulnerability assessment and included in the software implementation database

Group	Information	Detail
Database	File management	Same Information from Inspection and Maintenance Database
Bridge Information	General Bridge Information	Number of spans, bents, element types, units (US customary or metric SI)
		Number of vibration modes to use
		Use of rigid zones in bents
	Deck and Span Information	Deck width, thickness, additional mass,
		Span Information
Structural Elements	Geotechnical Information	Deck expansion joints type and location
		Same as described in section 5.2
		Cross-section dimensions
	Bent Column Information	Material information
		Modeling parameters (cracking, effective area, nonlinear characteristics)
		Cross-section dimensions
	Bent Beam Information	Material information
		Modeling parameters (cracking, effective area, nonlinear characteristics)
		Cross-section dimensions
	Bearings Information	Type
		Dimensions
	Restrainers	Type
		Dimensions
		Material properties
	Piles	Pile types
		Piles dimensional properties
		Piles material properties
	Footings	Footing Type
		Footing dimensional properties
		Footing material Properties
		Piles geometrical distribution
	Abutments	Abutment Type
		Abutment dimensional properties
		Abutment material Properties
		Piles geometrical distribution
Bents	Bent Types	Geometry
	Bent Location	Element type
Ground Motion	Spectrum	Location in bridge
	Accelerogram	From Bridge Specs
	Spectrum computed from accelerogram	From accelerogram database
	Increasing ground motion definition	Computation of spectrum from ground motion and smoothing possibilities
		Parameters to obtain performance patterns by variation of ground motion intensity

The information gathering process for each bridge is initiated including the information contained in the Inspection and Maintenance Database. The rest of the data must be provided from information contained in the bridge drawings.

Using this information several tasks were performed once a process is initiated. Some of these tasks are performed only once in the process — routine tasks — while others are performed repeatedly with each level of intensity of ground motion — ground motion intensity-dependent tasks—.

6.4.1 Routine tasks

Those processes that are not dependent on the ground motion intensity were performed first. They comprised the following tasks:

- Computation of element section properties.
- Computation of mass and stiffness properties for the superstructure and substructure elements.
- Computation of transverse and longitudinal vibration periods and modal shapes.
- Definition of strength in flexure and shear for all elements. For bent girders this was performed at sections where maximum stresses are expected under seismic effects. For bent piers and walls flexure-axial force interaction diagrams were computed. Reinforcement anchorage strength at joints was evaluated.
- Collapse mechanisms were evaluated for all bents defining the maximum tributary base shear strength. Distinction was made between flexural and shear collapse mechanisms.
- The lateral-load strength of bearings was evaluated. For bearings vulnerable to overturning or sliding under lateral load, the maximum transmitted shear before overturning or sliding was evaluated. Maximum available seat lengths were defined at non-fixed joints.
- For abutments and foundation elements, stiffness and deformation properties were evaluated. Strength for shear and overturning were evaluated.

From the above computed information a path of least strength was defined. For example bent piers were evaluated for the possibility of reaching shear strength before flexural strength thus initiating a brittle failure mode. At pier-foundation or wall-foundation joints the ability to transmit the forces developed when the pier reaches its maximum strength was evaluated, thus defining a weak link in the seismic load path. The maximum seismic ground-motion induced-forces that the structural system is capable of resisting were defined from the load path evaluation. These forces indicate the level of seismic base shear that may induce severe damage or collapse of the bridge.

6.4.2 Ground motion intensity-dependent tasks

The processes that depend on the ground motion intensity were performed for increasing levels of intensity of the ground motion. The ground motion intensity was defined in rock as peak ground acceleration, A_g . For a set value of peak ground acceleration in rock the following tasks (see Fig. 6-1) were performed in the order presented:

1. From the geotechnical information the soil profile amplification was determined using the procedure described in Section 5.2. This permitted the computation of spectral ordinates that are comparable to those defined through parameters S_S and S_I as described in the *Draft AASHTO LRFD Seismic Provisions*. This fully described the response spectrum for which the rest of the vulnerability assessment for the set ground motion was made.
2. The liquefaction potential was evaluated for the set acceleration value and reported. No further analysis on collateral potential of damage to the bridge caused by liquefaction was performed as explained in Section 5.4.
3. Structural element internal forces were obtained using a linear modal spectral response procedure based on the vibration periods and modal shapes computed in the routine tasks and the response spectrum from Step 1. Strength demands were compared with the strength capacity levels obtained in the routine tasks. A similar comparison was performed for the displacement demand against the maximum allowable displacement obtained in the routine tasks. The bridge was deemed not vulnerable for the set intensity of seismic ground motion and green-tagged and no

further analysis was performed if: a) the strength demand in all the structural elements was not exceeded, b) the displacement demand was within the established tolerable limit, and c) displacements at the supports did not exceed the limits established previously in the routine tasks,. If any of the strength or displacements limits were exceeded tasks 4 to 7 were performed.

4. If the shear strength level or the evaluation of reinforcement anchorage indicated possibility of a bond failure, the bridge was deemed vulnerable. The bridge was yellow-tagged or red-tagged depending on the level of expected damage and the spread of damage to the different elements.
5. If the possibility of bearing element failure by overturning or excessive displacement at movable supports was detected, the bridge was deemed vulnerable. The bridge was yellow-tagged or red-tagged based on the amount of expected deformation and the magnitude of potential horizontal or vertical permanent displacements of the superstructure.
6. The bridge was updated to red-tag if compound yellow-tagging was obtained both from the strength and the displacement checks. No further analysis was performed and the vulnerability level was reported for the set value of ground acceleration if steps 4, 5, or 6 yellow-tagged or red-tagged the bridge.
7. If the flexural strength for any element was exceeded in the evaluation performed in step 3, a nonlinear response evaluation was made. A substitute-structure procedure [Shibata and Sozen, 1976] was performed by introducing damage ratios, μ , greater than unity to the elements where flexural strength was reached. A series of analyses were carried out for increasing values of the damage ratios to the elements until the base shear strength of the bridge as computed in the routine tasks was reached. The expected lateral displacement for the set value of ground motion intensity was the one obtained when the bridge reached the base shear strength. This procedure gives comparable results to those obtained by performing a push-over analysis. The stability of the structure was judged for the displacements obtained using displacement-based procedures. Based on these

results the vulnerability of the bridge was assessed as yellow-tagged or red-tagged.

Fig. 6-1 – Flow diagram for the formal vulnerability assessment implemented

CHAPTER 7 UPGRADE NEEDS OF BRIDGES ON THE EMERGENCY ROUTES

7.1 Introduction

The approach of developing and implementing a tool for studying the alternatives for defining the Indiana Earthquake Emergency Routes and the upgrade needs of the transportation structures along these routes responds to the importance of two variables linked to the emergency routes selected, and the definition of the earthquake ground motion intensity for which the vulnerability of the current inventory of bridges should be gauged. The effect on the upgrade needs of these two variables is substantial. Variations of the road segments to be included within the emergency routes affect the travel times and the number of bridges that may need upgrading. Once a set of routes were defined, the envisioned earthquake ground motion intensity to be used in defining the vulnerability of the transportation structures affects directly the upgrade needs for these structures.

7.2 Bridges studied

Table 7-1 shows the distribution by bridge type of the inventory of bridges in the Indot Vincennes District. Currently there are 827 bridges that are located in the Interstate System, in US Routes, and in State Roads. The distribution by location in the different counties was presented in Table 4-1. A total of 230 of these bridges are located in the selected Earthquake Emergency Routes of Indiana, corresponding to 28% of the inventory. Table 7-1 also shows the distribution by bridge type of the 69 bridges that were studied in detail using the information contained in drawings. The sample was defined trying to maintain the bridge type percentages similar.

Table 7-1 – Distribution by bridge type in the Vincennes Indot District of the total bridge inventory, in the suggested emergency routes, and in the bridges studied in detail

Bridge Type	Bridge Type Description	Amount in Vincennes District	% by bridge type	Located on EQ Emerg. Routes	% by bridge type
BT	Bailey Truss	1	0.1%		
CCTB	Continuous Concrete T - Beam	1	0.1%	1	0.4%
CPCBB	Continuous Prestressed Concrete Box Beam	23	2.8%	4	1.7%
CPCIB	Continuous Prestressed Concrete I - Beam	69	8.3%	22	9.6%
CPCTB	Continuous Prestressed Concrete T - Beam	5	0.6%	2	0.9%
CRCB	Continuous Reinforced Concrete Box	1	0.1%		
CRCG	Continuous Reinforced Concrete Girder	27	3.3%	15	6.5%
CRCRF	Continuous Reinforced Concrete Rigid Frame	1	0.1%	1	0.4%
CRCS	Continuous Reinforced Concrete Slab	162	19.6%	53	23.0%
CRPG	Continuous Riveted Plate Girder	3	0.4%	2	0.9%
CSB	Continuous Steel Beam	89	10.8%	46	20.0%
CSG	Continuous Steel Girder	16	1.9%	10	4.3%
CSTA	Continuous Steel Tied Arch - Truss	2	0.2%		
KCSB	Composite Continuous Steel Beam	51	6.2%	31	13.5%
KCSBG	Composite Continuous Steel Box Girder	1	0.1%		
KCSG	Composite Continuous Steel Girder	56	6.8%	33	14.3%
KSB	Composite Steel Beam	10	1.2%		
KSG	Composite Steel Girder	8	1.0%		
MPA	Metal Pipe Arch	1	0.1%		
MPAUF	Multi-Plate Arch - Underfill	23	2.8%		
PCAUF	Precast Concrete Arch - Underfill	3	0.4%		
PCB	Precast Concrete Beam	10	1.2%	2	0.9%
PCBB	Prestressed Concrete Box Beam	61	7.4%	2	0.9%
PCIB	Prestressed Concrete I - Beam	31	3.7%	1	0.4%
PCS	Precast Concrete Slab	1	0.1%		
PCSUF	Precast Concrete Slab - Underfill	3	0.4%		
PTRCS	Post-Tensioned Reinforced Concrete Slab	1	0.1%		
RCA	Reinforced Concrete Arch	32	3.9%		
RCAOS	Reinforced Concrete Arch - Open Spandrel	1	0.1%		
RCAUF	Reinforced Concrete Arch - Underfill	17	2.1%		
RCBUF	Reinforced Concrete Box - Underfill	16	1.9%		
RCG	Reinforced Concrete Girder	51	6.2%	2	0.9%
RCS	Reinforced Concrete Slab	21	2.5%	1	0.4%
RCSUF	Reinforced Concrete Slab - Underfill	2	0.2%		
RPG	Riveted Plate Girder	2	0.2%		
SB	Steel Beam	6	0.7%	1	0.4%
SPT	Steel Pony Truss	6	0.7%		
STT	Steel Thru Truss	9	1.1%		
UCA	Unreinforced Concrete Arch	1	0.1%		
WSTG	Welded Steel Thru Girder	3	0.4%	1	0.4%
Sum		827	100.0%	230	100.0%

Figure 7-1 shows the distribution by bridge type on the Earthquake Emergency Routes. The most abundant type of bridge (23%) corresponds to Continuous Reinforced Concrete Slab bridge type both in the whole district and in the selected emergency routes. Continuous Steel Beam bridges (20%) and Composite Continuous Steel Girder (14.3%) correspond to the next largest groups.

Fig. 7-1 – Bridge type distribution on the Earthquake Emergency Routes

7.3 Retrofit Needs

The current inventory of bridges in the Vincennes Indot District was studied for varying degrees of earthquake ground motion intensity using an approximate vulnerability assessment methodology. The same approach of varying the intensity of the ground motion was performed for the selected bridges that were studied in detail.

As expected, bridge vulnerability and liquefaction potential in some bridge sites increased with the ground acceleration level. The importance of the definition of the

ground motion intensity for which the vulnerability level is assessed is explained by the number of bridges that would be red-tagged — 7% of the total Vincennes District bridge inventory — when the envisioned design ground motion contained in current 17th Edition of the AASHTO Standard Specifications are used as compared with a 15% when the ground motion description contained in the Draft AASHTO LRFD Update.

The vulnerability assessment indicates that the most frequent cause of concern is associated with unseating of the superstructure at supports (65% of bridges red-tagged) followed by shear failure of bent vertical elements (25% of bridges red-tagged). Other sources of vulnerability studied account for the rest of the red-tagged bridges. This figure is consistent for the whole inventory and for the selected bridges studies in detail. Bridge age correlates directly with vulnerability in the entire sample.

Overturning of the widely used movable expansion support of the type shown in Fig. 7-2 was found to be associated with many of the cases of unseating detected. In some instances peak ground accelerations in surface as low as 10%g would cause overturning.

Fig. 7-2 – Typical Expansion Rocker Bearing

Liquefaction as a source of vulnerability was present in the sample studied using the bridge drawings that reported soil borings. 182 bridges out of 230 reported a geotechnical exploration that could be used for evaluating liquefaction with 43% of them reporting sand or sandy soil and a water table level above the material susceptible of liquefaction. Lowest peak ground acceleration in rock that would produce liquefaction was below 5% g in several sites. The average peak ground acceleration in rock to produce liquefaction for the 79 sites where potential for liquefaction was detected was found to be approximately 5% g for the $M_w = 7.5$ scenario and 8% g for the $M_w = 6.5$ scenario.

The HAZUS-MH implementation produced comparable results indicating the approach of using the FEMA methodology is warranted.

7.4 Improvement of the approximate vulnerability assessment

The approximate seismic vulnerability assessment based on the information contained in Indot Inspection and Maintenance Database can be improved in the future by including additional information in the database. This additional information already exists in each set of bridge drawings. Although the drawings are available in electronic form they were scanned from hard copies and this format is not amenable for automatic retrieval of the information thus requiring participation of an experienced engineer for collecting and adding the information to the vulnerability assessment database.

There is a natural tradeoff between the effort of adding and maintaining this additional information and the degree of resolution sought in the approximate assessment procedure. The possibility of making a more detailed vulnerability analysis as implemented for selected bridges constitutes the upper limit of the additional information to be included. This may be unnecessary for a properly calibrated approximate procedure. The amount of additional information must be selected in a manner that maximizes the quality of the result and minimizes the data acquisition effort.

Based on this, the minimum suggested set of additional information is the following:

- Description of bent type and element dimension for all supports of the bridge.
- Reinforcement at selected sections of the bent elements focusing mainly on transverse reinforcement.

- Additional information on bearings more from the point of view of vulnerability to seismic effects.
- Additional information on foundation types and soil profile.

This additional information may be collected when the routine programmed inspections of the bridge are performed.

The improvement in dependability on the approximate vulnerability analysis with just this additional information may well justify the information gathering effort.

CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Different alternatives for Earthquake Emergency Routes for the Vincennes District of the Indiana Department of Transportation (INDOT), located in the southwestern corner of the State, were studied from the transportation point of view (Part I of this report) and the implications of seismic vulnerability of the transportation structures located in these roads were studied (Part II of this report).

The Indot bridge inventory data and soils information were incorporated in a software tool based on geographical information systems technology (GIS) developed by FEMA (HAZUS-MH [FEMA, 2005]) for evaluation of the Earthquake Emergency Routes network of Indiana and for establishing policies for mitigation of earthquake hazard on these routes. This tool may be used to perform simulations for different earthquake scenarios, and to help in the response to an actual emergency caused by the occurrence of a strong earthquake in the region.

The software tool is capable of performing a seismic vulnerability approximate assessment of the bridge inventory of the Vincennes Indot District using solely the information contained in Indot Inspection and Maintenance Database. Using information from the bridge drawings the software tool developed allows performing a detailed seismic vulnerability assessment of selected bridges. The vulnerability assessment performed for the selected bridges was used to calibrate the approximate analysis procedure based only on Indot Inspection and Maintenance Database and to evaluate the vulnerability of the selected bridges for different levels of earthquake ground motion intensity.

The reported research will help in the study and adoption of emergency response policies for different earthquake occurrence scenarios. These policies may be updated in the future as mitigation programs are implemented and their results incorporated in the bridge vulnerability assessment database.

The reported comments on the current state of seismic risk evaluation for the southwestern region of Indiana will help the State of Indiana in making decisions related to the adoption of proposed bridge seismic design provisions or proposed seismic risk maps by permitting an evaluation of the impact of the new provisions or seismic risk maps.

8.2 Recommendations

It is suggested that the findings and developments of this research project may be applied by the Indiana Department of Transportation in the following ways:

1. Indot should consider formally adopting the Earthquake Emergency Routes for the State of Indiana from the routes studied, or variations of them. It is clear that the final set of Earthquake Emergency Routes adopted and maintained will be affected by issues and policies whose scope is outside those studied and reported in this research project. Policy decisions, budgetary constraints, and decisions related to new road projects, such as the projected south western continuation of I-69, will affect the formal adoption of the selected routes and will surely impose variations in the future. The methodology developed in this research project will provide valuable information to assist Indot the decision-making process presently and in the future.

2. The study and adoption of mitigation strategies to implement in the future by Indot to update and maintain the Emergency Routes of Indiana can be made possible by performing simulations for different earthquake scenarios including earthquake ground motion defined in current AASHTO Standard Specifications, proposed Draft AASHTO LRFD Seismic Provisions, or other appropriate bridge specification. These simulations will provide corresponding bridge retrofit scope for the selected routes or alternative definitions.
3. It is recommended that Indot expands the information contained in the Inspection and Maintenance Database to include information currently available only in the bridge drawings (see section 7.5). This will improve the vulnerability assessment of the bridges refining the mitigation policies to adopt in the future. This expansion of the database can be extended to other counties and districts not covered in this research project.
4. It is recommendable that Indot adopt a program for assessing the liquefaction, soil spread, and embankment failure potential along the routes at locations different from just bridge sites by providing a consistent evaluation methodology based on the implementation developed in this research project. This implementation will require a soil exploration program to provide geotechnical properties of sites where no information exists or was performed many years ago with different objectives than liquefaction evaluation. In the reported research liquefaction evaluations were made using solely boring information contained in selected bridge drawings.
5. It is recommended that FEMA HAZUS-MH software be adopted as the methodology for vulnerability assessment, mitigation decisions through the study of appropriate earthquake occurrence scenarios, and emergency response programs tuned to the study of these scenarios. Along with this

recommendation a key feature is the training of Indot personnel in the use of FEMA HAZUS-MH software. Engaging of the services of the Polis Center of Indiana University, an authorized HAZUS-MH earthquake and flood service provider, for this purpose should be studied by Indot.

CHAPTER 9 REFERENCES

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