THE POTENTIAL IMPACT OF A MID-AMERICA EARTHQUAKE ON THE RAILROAD NETWORK

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THESIS

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CHAPTER 1
INTRODUCTION

1.1 Introduction

In the winter of 1811-12, more than 2000 earthquakes occurred over a five-month interval. These earthquakes were centered near the confluence of the Mississippi and Ohio rivers near the town of New Madrid, Missouri. Between six and nine of these earthquakes registered a moment magnitude of 7.0 or greater (Johnston and Shedlock, 1992). The earthquakes were felt from Mexico to Canada, and as far away as Washington D.C. (Knox and Stewart, 1995). The largest earthquake in the New Madrid series occurred on February 7, 1812. This event registered a moment magnitude of over 8.5. It caused the Mississippi River to flow backwards, created numerous waterfalls and initiated widespread liquefaction throughout the area.

Analysis of historical and geological data shows that the New Madrid earthquakes described above were not isolated events. Strong earthquakes in the central Mississippi Valley have occurred repeatedly. The New Madrid region continues to have the greatest level of seismicity in the U.S. east of the Rocky Mountains (Knox and Stewart, 1995). Over 4,600 low magnitude earthquakes have been recorded in the Mid-America region since 1974 (Figure 1.1).

When the next major Mid-America earthquake occurs, the potential losses in the region are expected to be substantial because many of the structures in the region were not designed to be seismically resistant, there is poor soil foundation material in the area, and the size of the affected area is large (CUSEC, 1996). These factors, coupled with the knowledge gained from past earthquakes, indicate that transportation systems in the Mid-America region are vulnerable to damage in a large earthquake.

A major component of the national transportation system is the railroad network (Figure 1.2). In 1999, there were approximately 2,107 route miles of active trackage in the Mid-

America region exposed to potentially damaging peak ground accelerations. This represents 2.2% of the total route miles in the U.S. Many busy rail lines traverse the Mid-America region, including several with traffic levels greater than 60 MGT (Figure 1.3). In 1999, the total gross ton-mileage for rail lines in the region was 96.2 billion, which represents 3.2% of the total gross ton-miles in the U.S.

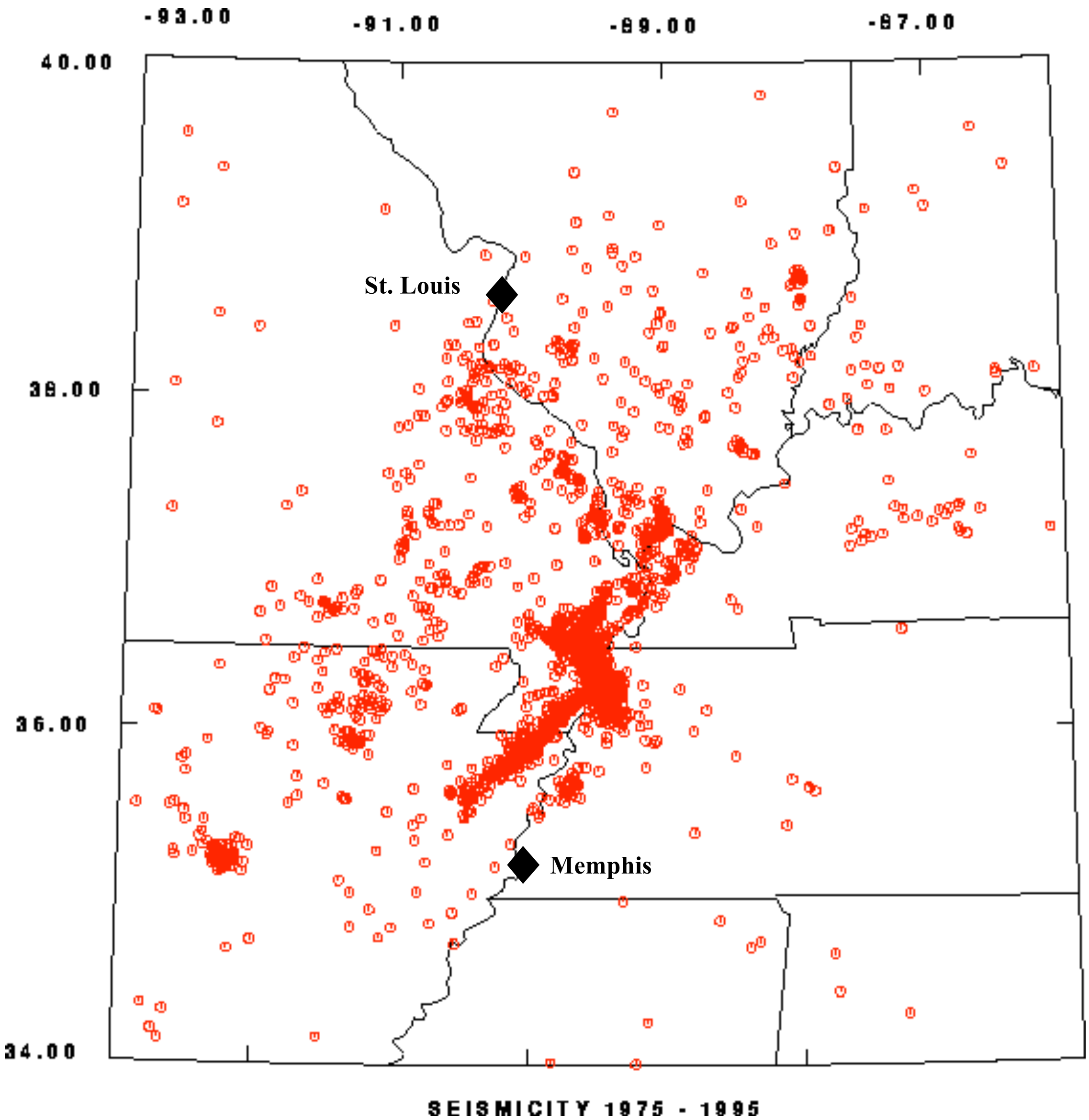

Figure 1.1. Earthquake Epicenters in the Central United States since 1974 (USGS, 1997)

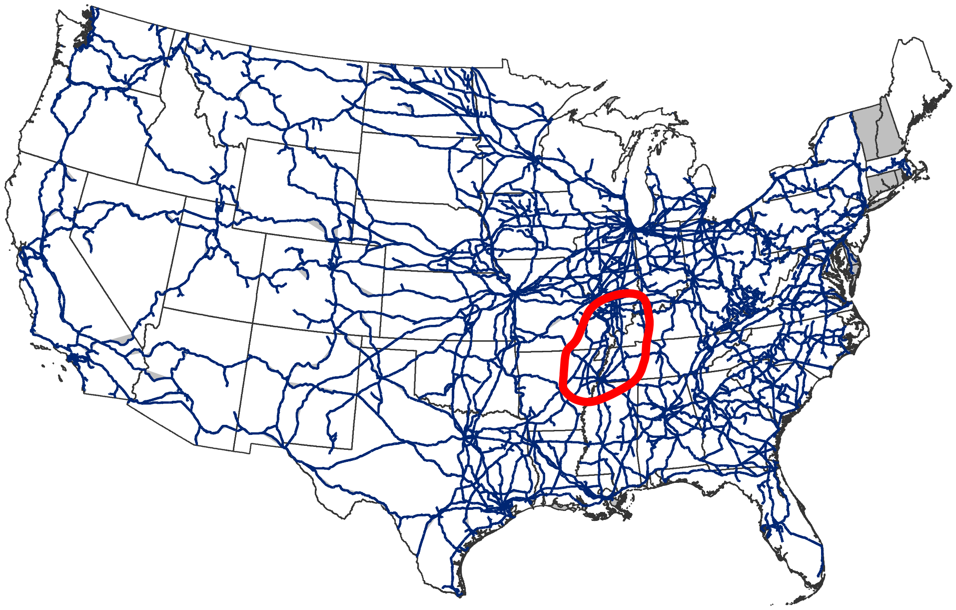

Figure 1.2. The United States Rail Network Including Rail Lines with One MGT or Greater (damaging ground motion area outlined in red)

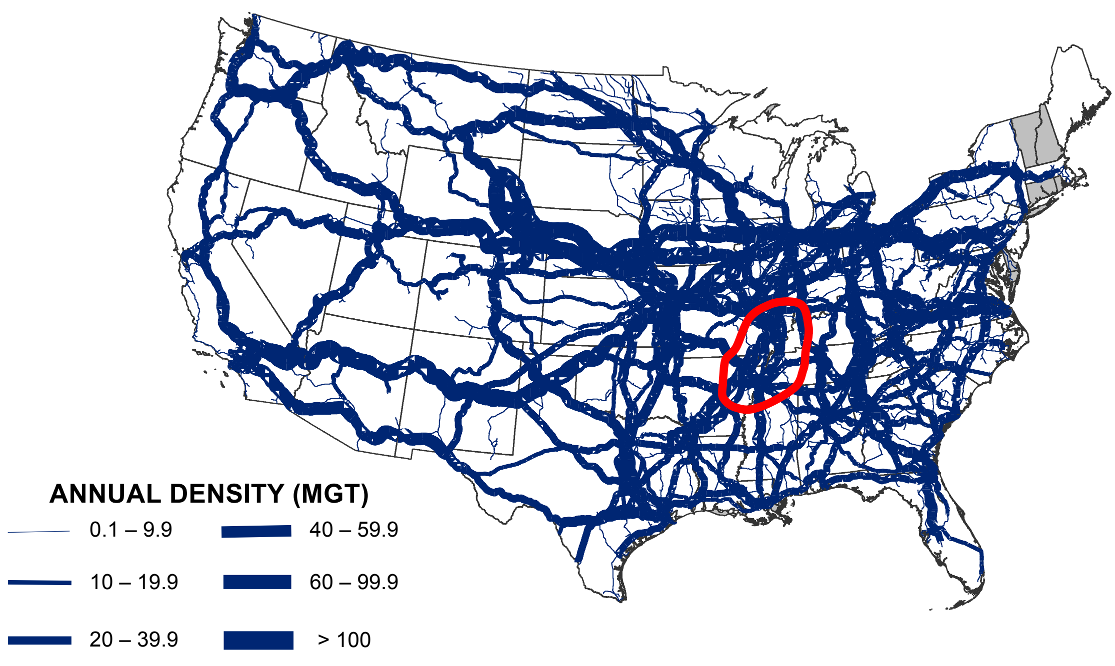

Figure 1.3. United States Railroad Network by Density (United States DOT 2001)

The scope of this project is to quantify the potential impact of a major earthquake in the Mid-America region on the nation's railroad infrastructure and traffic. The primary investigation is centered on the freight railroads because of the major east-west and north-south rail lines in the region. The region also includes two major freight railroad gateways, St. Louis and Memphis. These gateways serve as large classification facilities for sorting and interchanging rail traffic moving between eastern and western railroads.

Two major waterways, the Mississippi and Ohio Rivers, also traverse the Mid-America region. Much of the nation's rail traffic traveling through the central United States must cross one or both of these waterways. Eight major bridges in the region, five over the Mississippi River and three over the Ohio River, carry approximately 245 million tons of freight per year (USDOT, 2001), which accounts for 11.4% of the freight tons originated in the United States annually. Disruption to the rail network in the Mid-America region would require much traffic to be detoured, thereby creating additional capacity demands on rail lines outside of the region. The purpose of the analyses presented in this thesis is to:

- Quantify the railroad infrastructure potentially exposed to high seismic ground motion and liquefaction given a large earthquake in the central US.
- Identify and evaluate the major Mississippi and Ohio River railroad bridge crossings likely to be exposed to substantial ground motion.
- Examine the cost effectiveness of upgrading critical railway infrastructure features to improve resistance to seismic damage.
- Investigate the railroad traffic flows in the region and possible rerouting scenarios given a major earthquake in the central United States.

CHAPTER 2
REVIEW OF PREVIOUS EARTHQUAKE EVENTS INVOLVING RAILROADS

2.1 Introduction

Railway infrastructure has generally performed satisfactorily in previous earthquakes (Byers et al., 1994). Nevertheless, railroad bridges and roadbed are both susceptible to seismic damage if ground motion is severe. The first recorded damage to U.S. railroad bridges due to an earthquake was in the late 1800's (Pauschke, 1990). International earthquake events have also damaged railroad infrastructure. Among the earthquakes for which damage to railroads has been reported in detail are: the Charleston Earthquake of 1886, the California Earthquake of 1906, the Santa Barbara Earthquake of 1925, the Alaska Earthquake of 1964, the Loma Prieta Earthquake of 1989, the Costa Rica Earthquake of 1991, the Hanshin Earthquake of 1995 and the Izmit, Turkey Earthquake of 1999.

The US earthquakes listed above caused damage to bridge decks, piers, abutments, foundations and supporting soils (Pauschke, 1990). Early-built railroad bridges in the United States were constructed prior to the development of seismic construction standards. Railway roadbeds are also at risk in an earthquake. Many rail lines traveling through the central US are built on steep embankments, near major river floodplains. Both of these factors increase the risk for landslides and liquefaction, thus increasing the probability of damage to track structure.

2.2 Basic Measurement of Earthquake Severity

The severity of an earthquake can be described in terms of magnitude and intensity (USGS, 1989). Earthquake magnitude is the measure of the seismic energy released at the epicenter of the earthquake. It is represented by a single, instrumentally determined value (USGS, 1989). The Richter scale is the preferred method for describing the magnitude of an earthquake. Richter scale values are based on the amplitude of ground waves at the epicenter of an earthquake. Each whole number increase in magnitude represents a ten-fold increase in the amplitude of the ground waves (USGS, 1989). A Richter scale value of 6.3 or higher is generally considered to be a strong earthquake (USGS, 1989).

Intensity of an earthquake is based on the severity of the shaking experienced by people, infrastructure or natural features (USGS, 1989). Intensity differs from place to place and is dependent upon the distance from the location of the epicenter, local geology and other factors. Various scales exist to evaluate the intensity of earthquakes. The Modified Mercalli Intensity (MMI) Scale is the scale commonly used to record earthquake intensity in the United States (USGS, 1989). The intensity values are based on observed effects during the earthquake. A rough conversion scale between Richter scale and MMI scale is:

$$R = 1 + (2/3)*MMI \quad \text{(Beavers, 2002)} \tag{2.1}$$

where: R = the Richter magnitude

MMI = the Modified Mercalli Intensity value at the epicenter

Peak ground acceleration (PGA), another measurement of intensity, describes the maximum ground motion experienced based on distance from the epicenter of the earthquake. PGA values are typically recorded as a percentage of gravity (g), where 1g = 32.2 ft/s^2. A conversion between PGA and MMI is shown in Table 2.1.

PGA (%g)	< 0.17	0.17 - 1.4	1.4 - 3.9	3.9 - 9.2	9.2 - 18	18 - 34	34 - 65	65 - 124	>124
MMI	I	II - III	IV	V	VI	VII	VIII	IX	X+

Table 2.1. Approximate Conversion Scale between PGA and MMI (TriNet, 2001)

2.3 Accounts of Damage to Railroad Infrastructure in Individual Earthquakes

2.3.1 The California Earthquake of 1906

During the California Earthquake of 1906, the Southern Pacific Railroad's Pajaro Bridge suffered damage. The Pajaro Bridge crosses the Pajaro River near the city of Watsonville, California. The bridge consisted of four 120-foot intermediate deck Pratt trusses and two 50-foot end deck plate girder spans (Jordan, 1907). The fault producing the California Earthquake of 1906 crossed the bridge on the west end. The concrete piers and abutments were shifted laterally due to the ground motion. The greatest amount of longitudinal movement occurred near the west end, with a maximum increase of 3.5 feet distance between piers. The plate girder span resting on the abutment near the west end shifted off its bridge seat, but did not fall during the earthquake (Jordan, 1907). The bridge superstructure was also pushed slightly out of alignment.

2.3.2 The Santa Barbara Earthquake of June 29, 1925

The Santa Barbara Earthquake occurred early on the morning of June 29, 1925, with an estimated magnitude of 6.3 (Byers, 1996). The earthquake disrupted the Southern Pacific Railroad's Oakland – Los Angeles Coastline Route. It caused damage from milepost 331 near Gaviota, to milepost 598 near Ventura (Kirkbride, 1927). The earthquake caused damage to the track roadbed, various bridges and other railroad infrastructure including engine facilities and yard offices. The ground motion generated by the earthquake caused some damage to a steel viaduct over Dos Pueblos Canyon. The structure is 660 feet long and has a maximum height of 65 feet at the center of the bridge (Figure 2.1). The steel viaduct vibrated in such a distressing manner that an eyewitness thought the entire structure was going to collapse (Kirkbride, 1927). After the ground motion subsided, the structure returned to its original position on its bridge seats. Damage to the viaduct included shearing of the bed-plate bolts on the bridge seats and cracking of individual masonry piers (Kirkbride, 1927). The cracks were repaired using fresh mortar and 12-in thick concrete jackets. Damage to the track structure was due to the heavy amount of settlement of the larger fills in the area. The subsidence of fills ranged from 8 to 36 inches. Some deep cuts also broke down causing more damage to the roadbed. The earthquake also demolished building structures owned by the railroad.

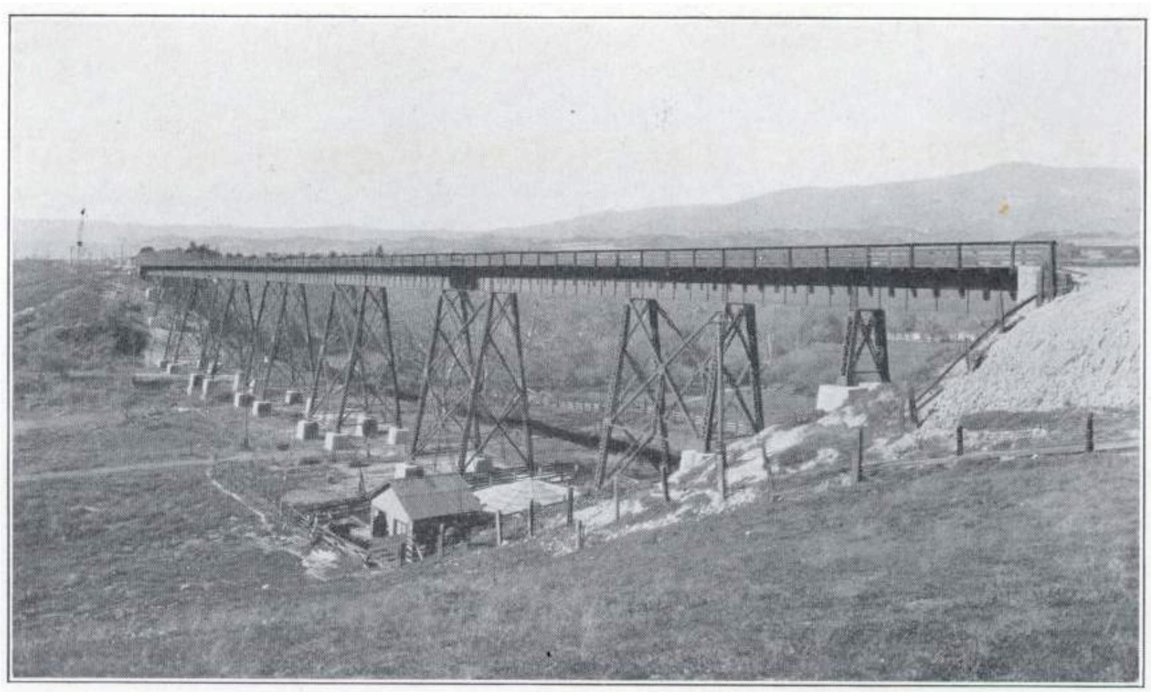

Figure 2.1. Dos Pueblos Viaduct on the Southern Pacific Railroad

2.3.3 The Alaska Earthquake of 1964

The Alaska earthquake occurred on March 27, 1964. The magnitude of the earthquake was 8.3 on the Richter scale. The Alaska Railroad, the primary overland freight and transportation link between Anchorage and Fairbanks and the ports of Seward and Whittier, sustained major damage due to the earthquake. Over 225 miles of Alaska Railroad line had severe to moderate damage associated with it. Infrastructure severely damaged or destroyed included bridges, shops, docks, communication facilities and rolling stock (Sturman, 1973). The main causes of damage were landslides, embankment failures, tsunami action and movement of soils that distorted or destroyed bridges. Reconstruction of the federally-owned railroad took over two years to complete at a cost of approximately $22 million (Sturman, 1973). In inflation-adjusted dollars, this is equivalent to $128 million using the Consumer Price Index (United States Department of Labor, 2002).

Approximately 110 bridges on the Alaska Railroad suffered damage due to the earthquake. 71 of the 110 seismically damaged railroad bridges required repairs prior to train operations (Sturman, 1973). The three general types of bridges found on the Alaska Railroad include steel-truss structures for large-stream and river crossings, and timber and plate-girder structures for small-stream crossings. The most severely damaged bridges were on the rail line connecting Seward (mile 0) and Portage (mile 64). A consistent feature of the damage to the bridges between Seward and Portage was shortening or longitudinal compression that dislodged superstructures from substructures (Sturman, 1973). The total repair cost for all bridges on the railroad totaled approximately $1.6 million. For further details on the damage to these bridges, see Pauschke, 1990 and McCulloch and Bonilla, 1967.

Railroad track structure damage occurred from Seward (mile 0) to Pittman (mile 167). The main cause of the roadbed damage was the submarine slide at Seward, subsidence of the embankments along the 50 miles of mainline track near the Portage area (Figure 2.2), and isolated areas of bending and kinking (Figure 2.3 and 2.4) (Sturman, 1973). The earthquake caused major distortion of the track structure both vertically and laterally. Substantial repairs were needed for embankments, grades and the track structure. Embankments were rebuilt using riprap to support steep slopes, sturdy fill material was used to rebuild the grade at two major slide locations, and the straightening and relaying of the track structure took place in various sections of the railway. The total cost for these activities was approximately $6.82 million. Figures 2.2-2.4 show track structure damage and embankment failures resulting from the earthquake.

Figure 2.2. Earthquake-triggered landslide on the Alaska Railroad mainline near Potter Hill (Abston and McGregor, 1995)

Figure 2.3. Buckled rails due to lateral movement of the embankment over a crushed culvert (Abston and McGregor, 1995)

Figure 2.4. Rails that buckled due to channel movement of the riverbanks during the 1964 Alaska earthquake (Abston and McGregor, 1995)

2.3.4 The Loma Prieta Earthquake of October 17, 1989

An earthquake with an estimated magnitude of 7.1 on the Richter Scale struck the greater San Francisco area on October 17, 1989. Railroad damage from the earthquake was minor compared to highway and building damage in the area surrounding the epicenter. Damage caused by the ground motion of the earthquake was documented on four Southern Pacific (SP) Railroad bridges and one Santa Fe (AT&SF) Railroad bridge (Kealey, 1990).

Bridge No. 119.67 – Santa Cruz (SP)

This bridge was constructed in 1903 and consists of a deck plate girder span 62 feet in length, and two 120-foot long through riveted truss spans (Kealey, 1990). This bridge was located approximately thirteen miles from the epicenter. Peak ground accelerations as high as 0.65 g were recorded at the epicenter of the earthquake (EQE Engineering, 1989). Earthquake damage was primarily confined to the pier between the deck plate girder and one of the through truss spans. Relative to the pier concrete, the capstone shifted eastward five inches. The pier also rotated to the west. To repair the pier, shifted stones were secured and grouted. Further monitoring of the shifted pier was recommended to determine if further stabilization was necessary (Kealey, 1990).

<u>Bridge No. 101.26 – Watsonville (SP)</u>

This bridge was constructed in 1906. It consists of five 60-foot through plate girder spans with short timber trestles at each end and was approximately twelve miles from the epicenter (Kealey, 1990). The piers are a pair of wrought iron casings filled with concrete, atop timber piling. The ground surrounding the bridge was primarily soft clay. Parts of the timber trestle subsided, causing the deck and track to drop four to six inches (Kealey, 1990). The other earthquake damage was at the end pier where the concrete beam was twisted and tilted. Temporary pile bents were placed to expedite the opening of the bridge to traffic.

<u>Bridge No. 113.46 – Castroville (SP)</u>

This bridge was constructed in 1904. The bridge consists of five 140-foot long through truss spans, and is located approximately 25 miles from the epicenter (Kealey, 1990). The main damage to the bridge was shifting of the piers, which caused lateral displacement and track misalignment. Pier two shifted five inches to the east, and pier three shifted to the left approximately six to nine inches (Kealey, 1990). To repair the bridge, the track was shifted to the original alignment and the trusses were also realigned.

<u>Bridge No. 33.31 – Martinez (SP)</u>

This bridge was constructed in 1929. It is a vertical lift bridge (double tracked) totaling 5,603 feet in length, and is located approximately 65 miles from the epicenter of the earthquake. The bridge was in the down position and suffered only minor damage during the earthquake. The primary damage was severe bending of the counterweight guide rails (Kealey, 1990).

Bridge 11.65 – Oakland (AT&SF)

This bridge was built in 1937. It consists of six skewed steel spans on a twelve-degree curve with timber trestle approaches on each end (Kealey, 1990). The six skewed spans consist of one 36.5-foot wide flange beam span, and deck girder spans measuring 73, 65, 56, 52, and 50 feet in length. The skewed spans are supported on seven concrete piers. Damage to this bridge included fine cracks and some spalling at the base of each independent column section, although no reinforcement was exposed (Kealey, 1990). Timber braces on the east approach were also broken during the earthquake.

The Southern Pacific Railroad estimated the cost to be approximately $100,000 to repair the above structures for continued service (Kealey, 1990). The amount of railroad damage was much less than that experienced by highway bridges in the same area. This is most likely due to the high live load design of railroad bridges, which provides a large reserve capacity over the dead load design (Kealey, 1990).

2.3.5 Costa Rica Earthquake of April 22, 1991

An earthquake of magnitude 7.5 occurred in Costa Rica on April 22, 1991. This earthquake initiated liquefaction-induced lateral spreading in the area causing extensive damage to bridges and track structure in the railroad system. Several segments of roadbed were misaligned due to the lateral spreading caused by the earthquake. The greatest amount of damage occurred at major river crossings. Bridge damage included bridge decks being propelled over abutments, piers shifting toward the river, and ground fills that subsided as much as two meters (6.56 feet) (Rollins et al., 1992). Below is a summary of three railroad bridges that sustained major damage in the earthquake.

Rio Matina Railroad Bridge

This bridge measures 400 meters (1,312 feet) in length. It consists of five simply supported steel plate girder sections on each side of the main channel, resting atop concrete piers. The main river channel crossing consists of three truss sections resting on concrete caissons. The sediments of the floodplain experienced liquefaction during the earthquake on both sides of the river. The subsequent lateral spreading caused lateral displacements of the bridge piers and caissons. These displacements caused several plate girder sections to drop off their supports and fall to the ground. The trusses moved off their seats, but did not tip or fall off the caissons. Temporary repair of the structure using timber shoring to place the girders to their approximate original alignment and elevation allowed trains to use the bridge, but at reduced speeds (Rollins et al., 1992).

Rio Bananito Railway Bridge

This railway bridge is a single-truss structure that measures 50 meters (164 feet) in length. During the earthquake, ground displacements due to liquefaction and lateral spreading thrust the supporting caissons out from under the seating plates on both ends of the truss bridge (Rollins et al., 1992). This movement of the bridge allowed the truss to tip eastward by about 15 degrees. The lateral displacement of the ground beneath the south and north ends of the bridge ranged from 1 meter to 2.5 meters. The retaining walls in the abutments were also shifted 1.4 meters (4.6 feet) and 2.8 meters (9.2 feet), on the northeast and southeast ends of the bridge, respectively.

Rio Estrella Railway Bridge

This bridge, constructed of simply-supported plate girder spans resting on steel piers in the floodplain and steel trusses supported on caissons over the river, sustained extensive damage during the earthquake. The ground motion tilted the piers and caused most of the plate girder sections to fall onto the floodplain (Rollins et al., 1992). The piers shifted between 0.15 meters (0.49 feet) and 0.8 meters (2.62 feet). Clockwise rotation of the piers also occurred as much as a few degrees.

2.3.6 The Hanshin Earthquake of January 17, 1995 (Japan)

The Great Hanshin Earthquake of January 17, 1995 caused major damage to the railway system within Kobe City and the surrounding areas. The four railroads affected by this earthquake were the privately-owned Hankyu and Hanshin Railways, and the Japanese-owned Shinkansen and JR West Railways. The railway infrastructure items with major damage were tunnels, elevated framed concrete viaducts, and bridges. The elevated viaducts created the greatest problems due to major sections collapsing from the ground motions produced by the earthquake. Shear failure in the concrete reinforced columns occurred on 144 out of 1,263 thirty-meter long sections (Billings, 1995). Due to the urgency to reopen the lines to traffic, most of the superstructure was reused with newly constructed columns. The Japanese railway companies surveyed their lines following the earthquake to find the most vulnerable sections with inadequate shear capacity (Billings, 1995). Time needed to reopen various affected lines ranged from 2.5 – 5 months. The repair cost of the four damaged railway lines totaled approximately $1.5 billion (Billings, 1995). Losses, representing the loss of revenue to the railways due to the earthquake, came to approximately $417.5 million (Table 2.2).

2.3.7 The Izmit (Kocaeli), Turkey Earthquake of August 17, 1999

The Turkish State Railway system operates passenger and freight trains using both electric and diesel power. The rail system is owned and operated by the government as a branch of the highway system (Tang, 2000). The railroad infrastructure damaged due to the earthquake (moment magnitude = 7.4) included two major items: track and roadbed that crossed the fault line, and buildings owned by the railroad. The passenger car shop at Adapazari, used for producing and maintaining the passenger car fleet, was demolished by the earthquake. Track damage in the area was located close to the fault. The major track and roadbed problems were buckling, loss of track geometry

Route	Date of Reopening		US $ Millions		Critical Repair item for Reopening
	Date	Days from Earthquake	Repair Cost	Losses	
Shinkansen	4/8/95	81	835	417.5	Viaduct reconstruction
J R Trunk Line	4/1/95	74			Viaduct/station reconstruction
Hankyu Railway	6/12/95	146	292.2	-	Viaduct reconstruction
Hanshin Railway	6/26/95	160	417.5	-	Line reconstruction

Table 2.2. Summary of cost and repair times for railways following the Hanshin Earthquake of 1995 (Billings, 1995)

and loss of strength in the supporting soils. Some local areas of track suffered from subsidence and/or liquefaction of the sub-grade (Tang, 2000). Overall the track structure was solid and required only minor repairs. None of the 181 bridges (over four meters in length) within eighty kilometers of the earthquake epicenter were damaged (Tang, 2000). Following restoration of the electrical system and repair of the track damage, the system faced only minor speed restrictions. Normal operations were expected 1.5 months after the earthquake. The estimated cost of repair was approximately $23 million (Tang, 2000), most of which was reconstruction of railroad buildings.

2.4 Summary of Railroad Damage

Damage to railroad infrastructure over the past century has varied considerably; however, the accounts described above illustrate the potential for railroad infrastructure damage in a major Mid-America earthquake.

Although some instances of major bridge damage occurred, railroad bridge performance in earthquakes has generally been superior to highway bridges (Byers, 1996). It is thought that this is due to the following factors: conservatively sized foundations, bearing support conditions, large factors of safety in the design of many components, and most railroad bridges have simply supported spans (Byers et al., 1994). The strong bond between the bridge structure and adjacent roadbed provided by the track is another factor that uniquely contributes to the lateral and longitudinal resistance of railway bridges. The Transportation Technology Center, Inc. (TTCI) performed tests for the Association of American Railroads (AAR) to determine the lateral and longitudinal resistance provided by the track structure in an earthquake. The tests showed that the lateral resistance for an open deck, deck plate girder bridge with five 62-foot spans exceeds some of the most severe requirements used in seismic design (Uppal, Joy, and Otter, 2000), and longitudinal resistance is greater than approximately three times the dead weight of the span. In another test, Uppal, Otter and Doe (2001) found that properly anchored rail and bridge decks can provide significant resistance to seismic ground motion. These tests show that track structure offers additional restraint and a means to transfer seismic loads to the roadbed, thus reducing the seismic load carried by the substructure of the bridge.

Data for twenty earthquakes with magnitudes greater than 6.0 that caused railroad disruptions are presented in Table 2.3 (including several with detailed descriptions earlier in this report.) Soil movement from liquefaction and/or lateral spreading and intense shaking were the most common causes of railroad damage in these earthquakes (Byers, 1996).

Despite the generally good performance of railroad bridges exposed to seismic ground motion, there are instances of serious damage as documented above. Because of the seismic force of the New Madrid fault, the possibility of a high magnitude earthquake in the Mid-America region is not remote. There is a 5% probability of an earthquake with an MMI value of VII in the next fifty years. This, combined with the number of important rail lines and the concentration of rail freight traffic on the limited number of major river-crossing bridges in the region, suggested that an analysis of the potential impact of a Mid-America earthquake on the regional and national rail transportation system was appropriate.

Table 2.3, Byers Table

In the following sections of this thesis, I will estimate the exposure of various aspects of railroad infrastructure in the Mid-America region, develop a simple model for determining the cost effectiveness of upgrading railway bridges to enhance seismic resistance, and present a preliminary study of the rail freight traffic flow in the region and some examples of possible rerouting scenarios of rail traffic given the loss of one or more bridges due to a major earthquake in the central United States.

CHAPTER 3

EXPOSURE AND RISK ASSESSMENT OF MAJOR MID-AMERICA RAILROAD INFRASTRUCTURE

3.1 Introduction to Risk and Data Acquisition

The Mid-America region has a vital position in the United States railroad network. Five of the seven major Class 1 North American railroads have important rail lines or trackage rights in the area, including Burlington Northern Santa Fe (BNSF), CSX Transportation (CSXT), Canadian National-Illinois Central (CN-IC), Union Pacific (UP) and Norfolk Southern (NS). The region is one of the major transfer points for rail traffic interchange between eastern and western railroads. Two of the five major east-west U.S. "gateways" where this interchange occurs, St. Louis and Memphis, are in the affected region. Cincinnati, another important interchange point, is also in the region. Amtrak, the nation's intercity passenger railroad, has one daily train (City of New Orleans, Train #58 and #59 between Chicago and New Orleans via Memphis) in each direction through the affected region. The southern end of the Chicago to St. Louis intercity passenger rail corridor is also within the area likely to be affected by a Mid-America earthquake. This line is currently being upgraded for high-speed rail service. If the rail infrastructure in the Mid-America region were seriously disrupted, the entire United States rail network would be affected.

A large portion of the Mid-America region is potentially vulnerable to a major earthquake due to the New Madrid fault (Table 3.1). The seven-state Mid-America region includes Arkansas, Illinois, Indiana, Kentucky, Mississippi, Missouri, and Tennessee. The seven-state region has a total land area of approximately 345,610 square miles. The United States Geological Survey (USGS) has developed a geographic information systems (GIS) database containing probable peak ground acceleration contours with a two, five and ten percent likelihood of occurrence in the next fifty years. The contour values are expressed as a percentage of one gravity (g), where 1g = 100%. For the seven-state region, 2% of the area has a 5% chance of experiencing ground motions ranging from 20 – 40% g, and 13% of the region has a 2% chance of experiencing 20 – 40% g peak ground accelerations. Three percent of the seven-state region has a 2% chance of experiencing PGA values exceeding 40% g.

	Probability of Major Earthquake in the Next 50 Years		
Ground Motion (g's)	10%	5%	2%
0.04 - 0.19	130,662 (38%)	253,000 (73%)	345,610 (100%)
0.2 - 0.4	0	8,022 (2%)	43,367 (13%)
> 0.4	0	0	9,647 (3%)

Table 3.1. Area of the Seven-State Mid-America Region (in square miles) by Earthquake Probability and Ground Motion Category (percentage of total area in parenthesis)

A GIS analysis was performed to quantify the exposure of railroad infrastructure to large seismic ground motions and liquefaction effects in the region. The infrastructure study concentrates on railroad route mileage and major railroad bridges in the region. The analysis provides a quantitative estimate for the extent that a major earthquake could affect each of the major railroads.

The GIS data were obtained from the following sources: the United States Department of Transportation (USDOT), the United States Geological Survey, a major railroad in the region, and the Illinois Geological Survey. The Federal Railroad Administration (FRA) manages the GIS data provided by the USDOT. The database includes a GIS shapefile of the railroad network in the continental United States. It also contains attributes for each railroad line on the network. The key attributes in the database include: railroad owners, railroads with trackage rights, traffic density of the rail line (measured in annual MGT), railroad classification code, type of traffic control system, and the traffic density of the rail line from the previous year (in annual MGT).

To assess the probability of a major earthquake, the USGS has developed hazard maps for the Mid-America earthquake region (Figure 3.1). The data include a GIS shapefile of PGA contours with a ten percent, five percent, and two percent likelihood of occurrence in the next fifty years. These hazard maps provide an estimate of possible future seismic ground motions in the area.

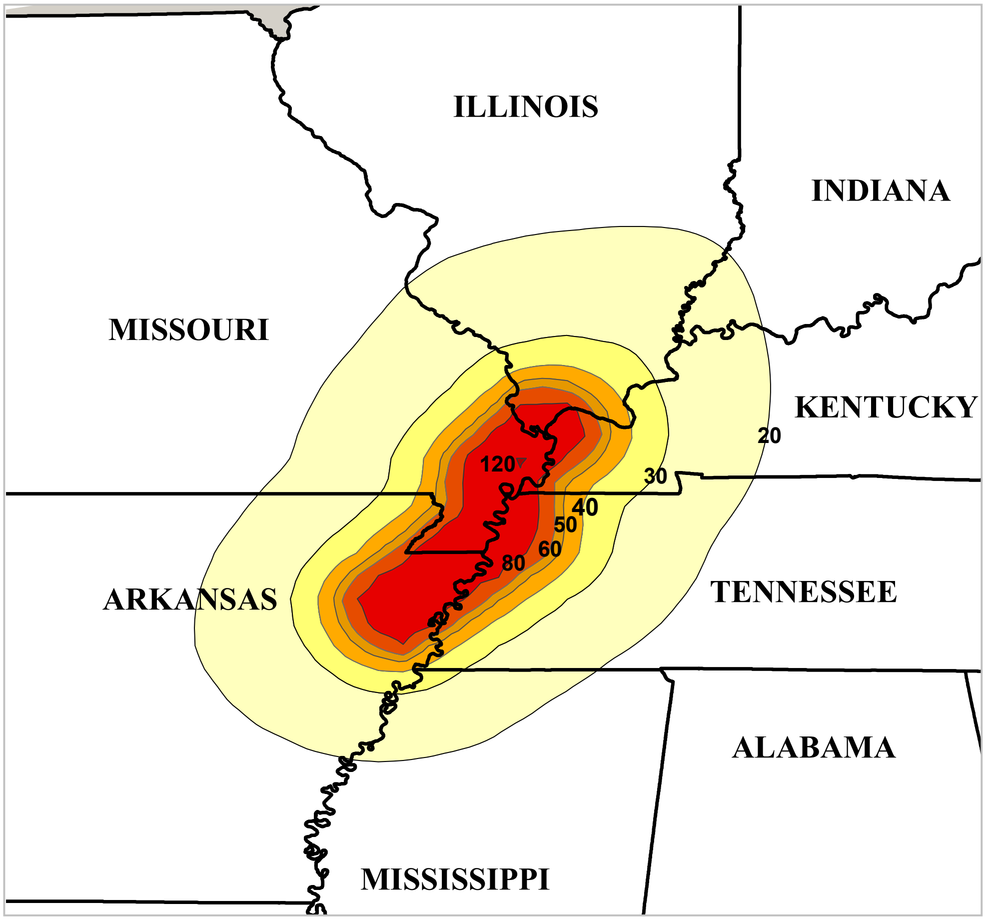

Figure 3.1. PGA Contours of the Mid-America Region with a 2% Probability of Occurrence in the Next Fifty Years

One of the major railroads in the region provided their GIS database. The database contains a shapefile of their railroad lines and information on railway bridges and miles of track within the Mid-America region. A list of categories in the GIS database is provided in Appendix A (Table A-1). The attributes of interest include: the location of bridges, the length of the bridge spans and the construction materials of the spans (steel, concrete, timber, or a combination of these).

Another factor that could affect railroad infrastructure in the Mid-America region is soil liquefaction. Liquefaction is a phenomenon in which the strength and stiffness of a soil are reduced by rapid shaking due to ground motion. Liquefaction occurs in saturated soils, in which the pores between soil particles are filled with water. The vibration in an earthquake causes the pore water pressure to increase. As the pore water pressure increases, the friction between soil particles becomes negligible. This causes the saturated soil to lose its strength, thus reducing its capacity to act as a foundation. Soils in the Mid-America region are often sufficiently saturated that soil liquefaction is a concern.

I obtained soil liquefaction GIS data from the Illinois Geological Survey (State of Illinois, 1995) to determine the extent of railroad infrastructure potentially exposed to liquefaction in the Mid-America region. The GIS data characterize soil liquefaction potential as either high or low, depending upon soil characteristics (Figure 3.2).

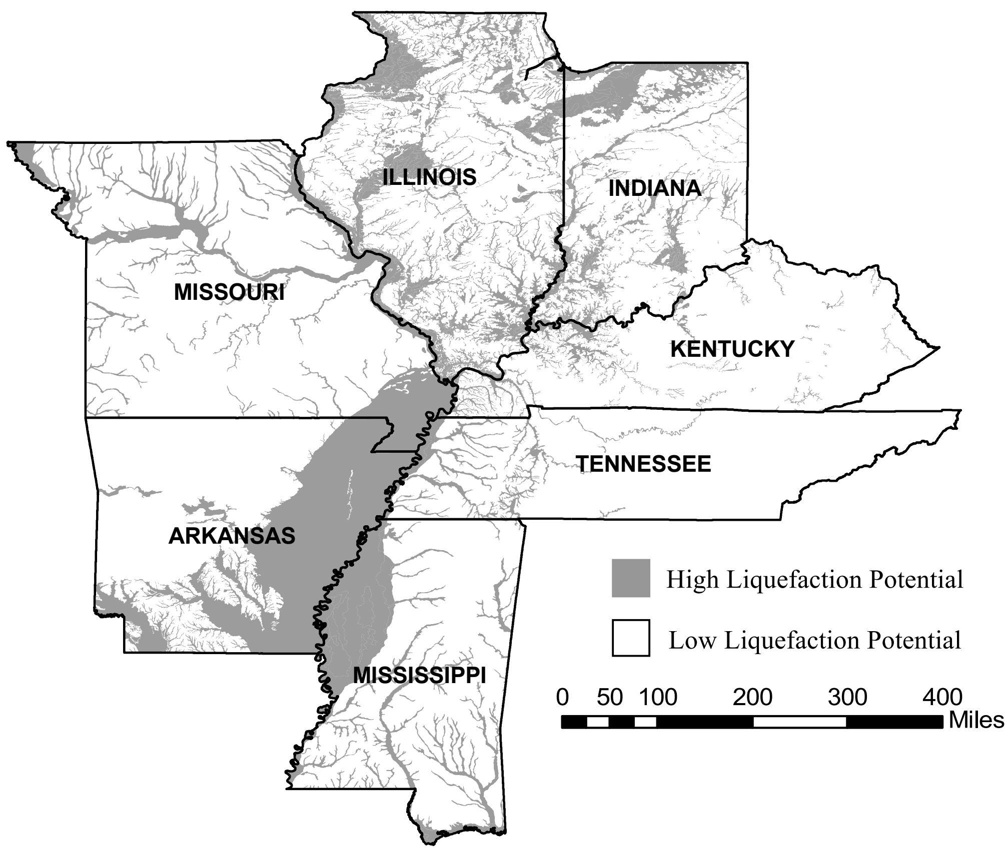

Figure 3.2. Soil Liquefaction Potential for the Seven-State Mid-America Region

3.2 Major Railroad Route Mileage at Risk

A GIS analysis was performed to determine the major railroads that would be affected by a Mid-America earthquake. Using GIS software, a map displaying the major rail lines in the Mid-America region was created (Figure 3.3). Major rail lines are defined as those carrying greater than or equal to twenty MGT annually. Twenty MGT is roughly equivalent to a total of eight average-sized freight trains per day (Appendix B).

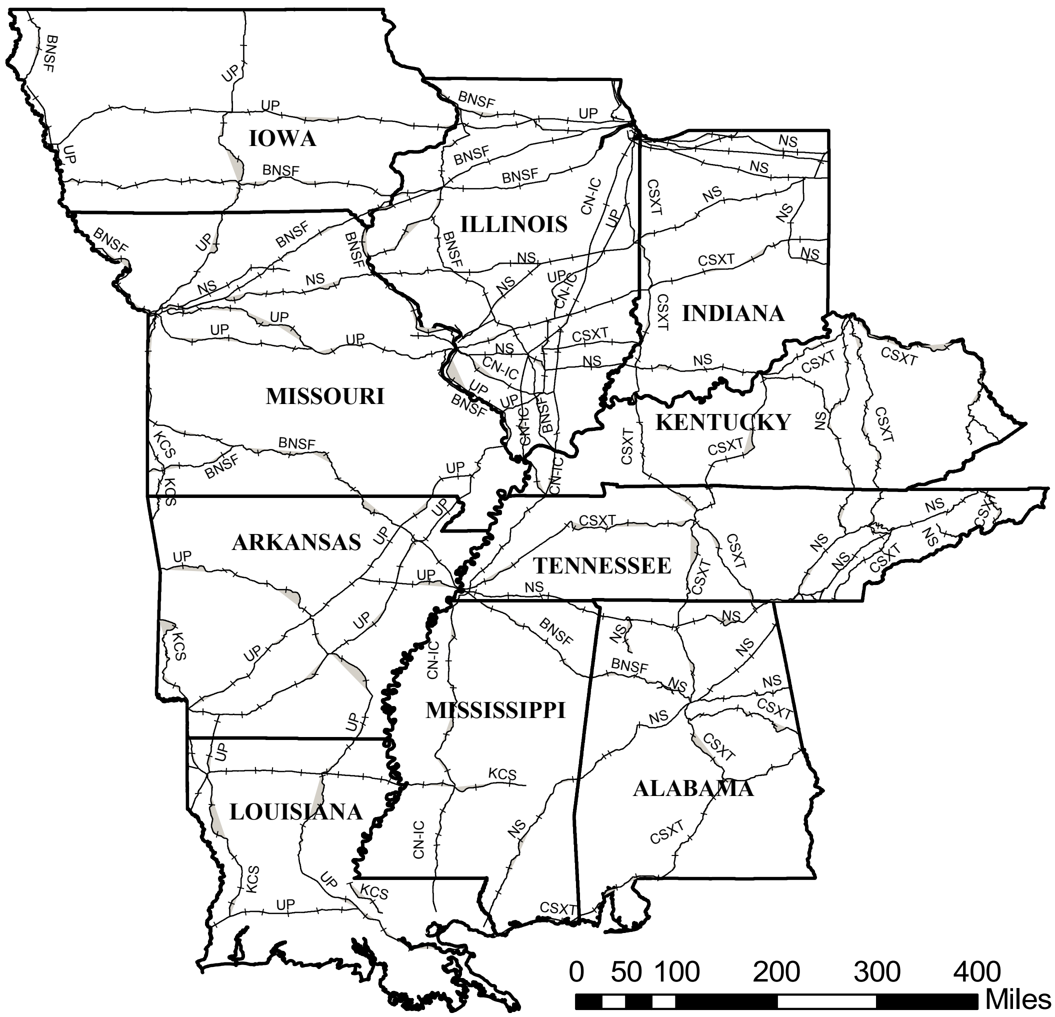

Figure 3.3. Railroads in the Mid-America Region with an Annual Density Greater than or Equal to 20 MGT

The railroad network analysis concentrates on the route mileage potentially exposed to peak ground accelerations and soil liquefaction in the Mid-America region. Railroad bridge engineers have developed estimates of threshold criteria for railroad damage in earthquakes as follows (Byers, 2001):

- Less than 0.2 g PGA ~ slight damage
- 0.2 – 0.4 g PGA ~ moderate damage
- Greater than 0.4 g PGA ~ severe damage

These threshold PGA values may be lower if soil liquefaction occurs. Using GIS software, I performed an overlay of the major railroad lines and the PGA contours for the Mid-America region (Figures 3.4 – 3.6). The route mileage exposed to various probabilities of PGA levels and high liquefaction potential soil zones were also determined, and are presented in Tables 3.2 – 3.4.

The contour values in Figure 3.4 represent the probable peak ground accelerations for a 10% probability earthquake with a fifty-year return period. There are approximately 4,566 route miles within the 4 – 9% g PGA range. CSXT and NS have the largest number of route miles affected, but their percentage in high liquefaction potential zones is negligible. The high liquefaction potential areas are associated with the major river valleys. UP has multiple rail lines that follow the Mississippi River valley. Of their 949 route miles in the affected region, 68% are in areas of high liquefaction potential. CN-IC and BNSF also have extended portions of their rail lines within the Mississippi and Ohio River valleys, so their route mileage potentially exposed to high liquefaction is also substantial (20% and 36%, respectively).

The contour values in Figure 3.5 represent the probable peak ground accelerations for a 5% probability earthquake with a fifty-year return period. The contour values are higher than for the 10% probability earthquake due to the increased intensity of the lower probability earthquake. There are approximately 6,668 route miles in the affected region with peak ground accelerations ranging from 5 – 25% g. CSXT and NS both have a large amount of route miles in the affected region, but no route miles in the higher PGA range

(20 – 25% g), and their percentage in high liquefaction potential areas is minimal. UP has 1,588 route miles in the affected region and 77 route miles potentially exposed to damaging PGA values. Of their 1,588 affected route miles, 85% are in areas of high liquefaction potential. CN-IC has 852 route miles in the affected region and 156 route miles potentially exposed to damaging PGA values. Of their 852 affected route miles, 85% are in areas of high liquefaction potential. BNSF has 827 miles in the affected region and 65 route miles potentially exposed to damaging PGA values. Of their 827 affected route miles, 34% are in areas of high liquefaction potential.

The contour values in Figure 3.6 represent the probable peak ground accelerations for a 2% probability earthquake with a fifty-year return period. The contour values are noticeably higher compared to the 10% and 5% probability earthquakes, again due to the increased intensity associated with the lower probability earthquake. There are approximately 2,107 route miles in the region ranging from 20 – 100% g. CSXT and NS, respectively, have 332 and 212 route miles in the affected region, all of which are within the moderate damage range (20 – 40% g). UP has 692 route miles in the affected region and 192 route miles in the severe damage range (> 40% g). Of their 692 route miles in the affected region, 85% are in areas of high liquefaction potential. CN-IC has 573 route miles in the affected region and 177 route miles in the severe damage category. Of their 573 route miles in the affected region, 17% are in areas of high liquefaction potential. BNSF has 299 route miles in the affected region, of which 79 are in the severe damage range, and 33% are in areas of high liquefaction potential.

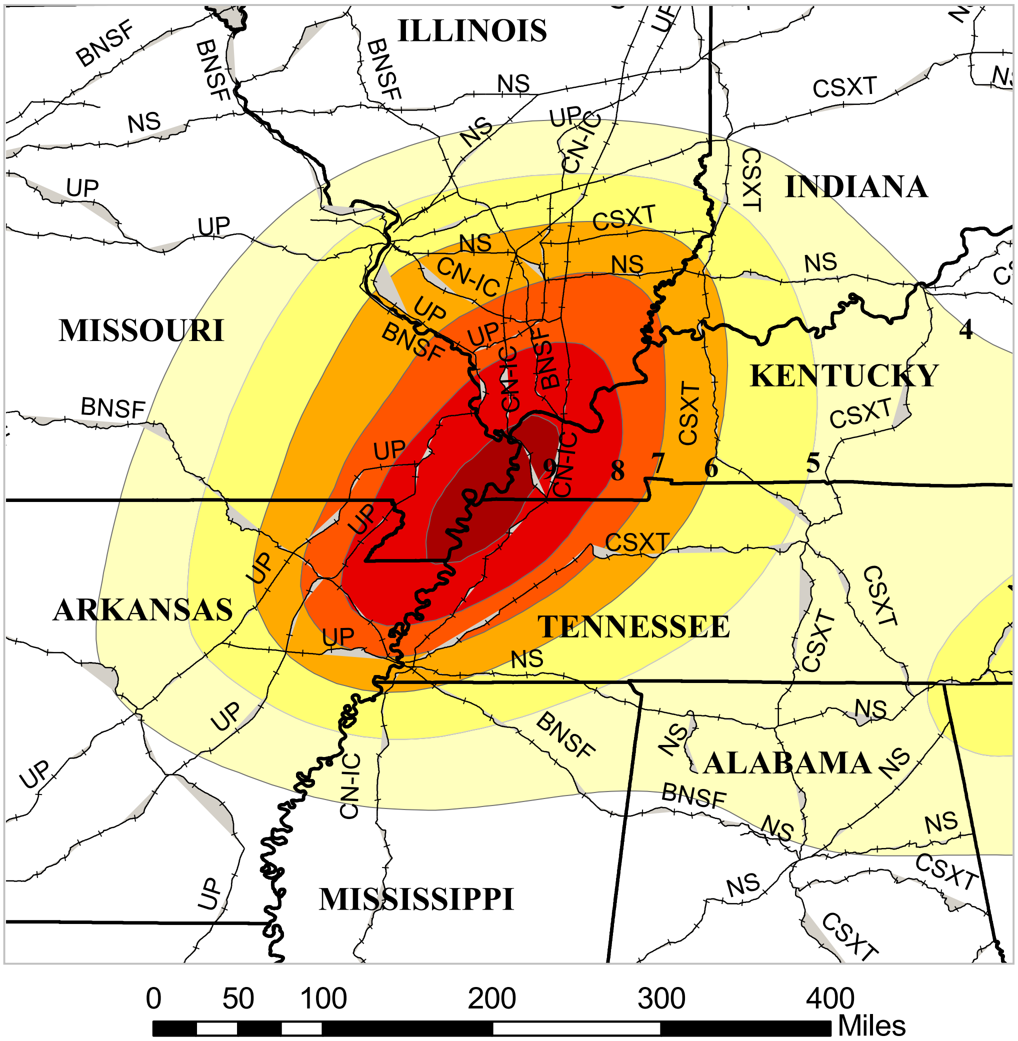

Figure 3.4. Mid-America Rail Lines and PGA Contours with a 10% Probability of Occurrence in the Next 50 Years

	Railroad (route miles)					
PGA	BNSF	CN-IC	CSXT	NS	UP	Total
4 – 9% g	565	676	1,400	977	949	4,566
Percentage in High Liquefaction Area	36%	20%	< 10%	< 10%	68%	25%

Table 3.2. Railroad Route Mileage Potentially Exposed to PGA Values with a 10% Probability of Occurrence in the Next 50 Years

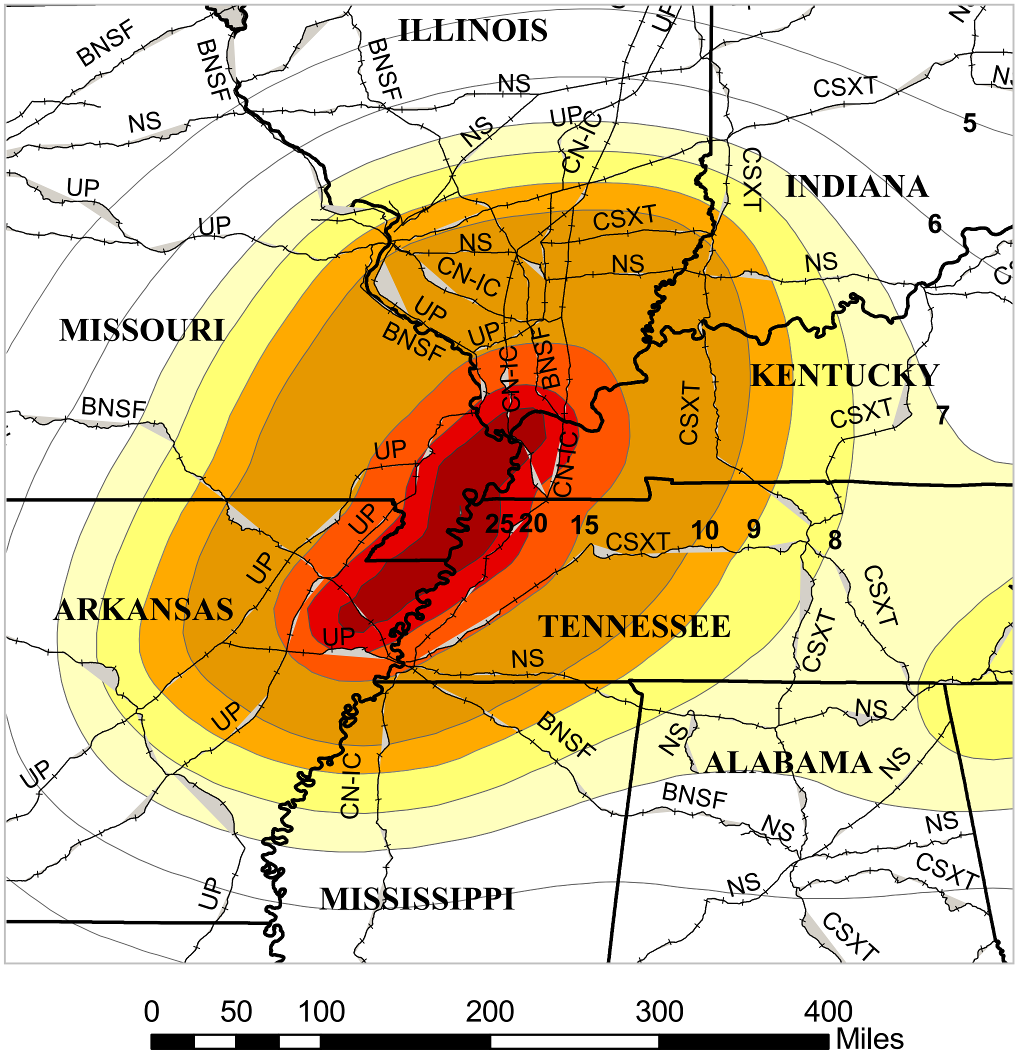

Figure 3.5. Mid-America Rail Lines and PGA Contours with a 5% Probability of Occurrence in the Next 50 Years

	Railroad (route miles)					
PGA	BNSF	CN-IC	CSXT	NS	UP	Total
5 - 9% g	517	271	1,622	1,205	920	4,535
10 - 19% g	245	425	356	220	591	1,836
20 - 25% g	65	156	0	0	77	298
Total Miles	827	852	1,978	1,425	1,588	6,668
Percentage in High Liquefaction Area	34%	18%	< 10%	< 10%	85%	36%

Table 3.3. Railroad Route Mileage Potentially Exposed to PGA Values with a 5% Probability of Occurrence in the Next 50 Years

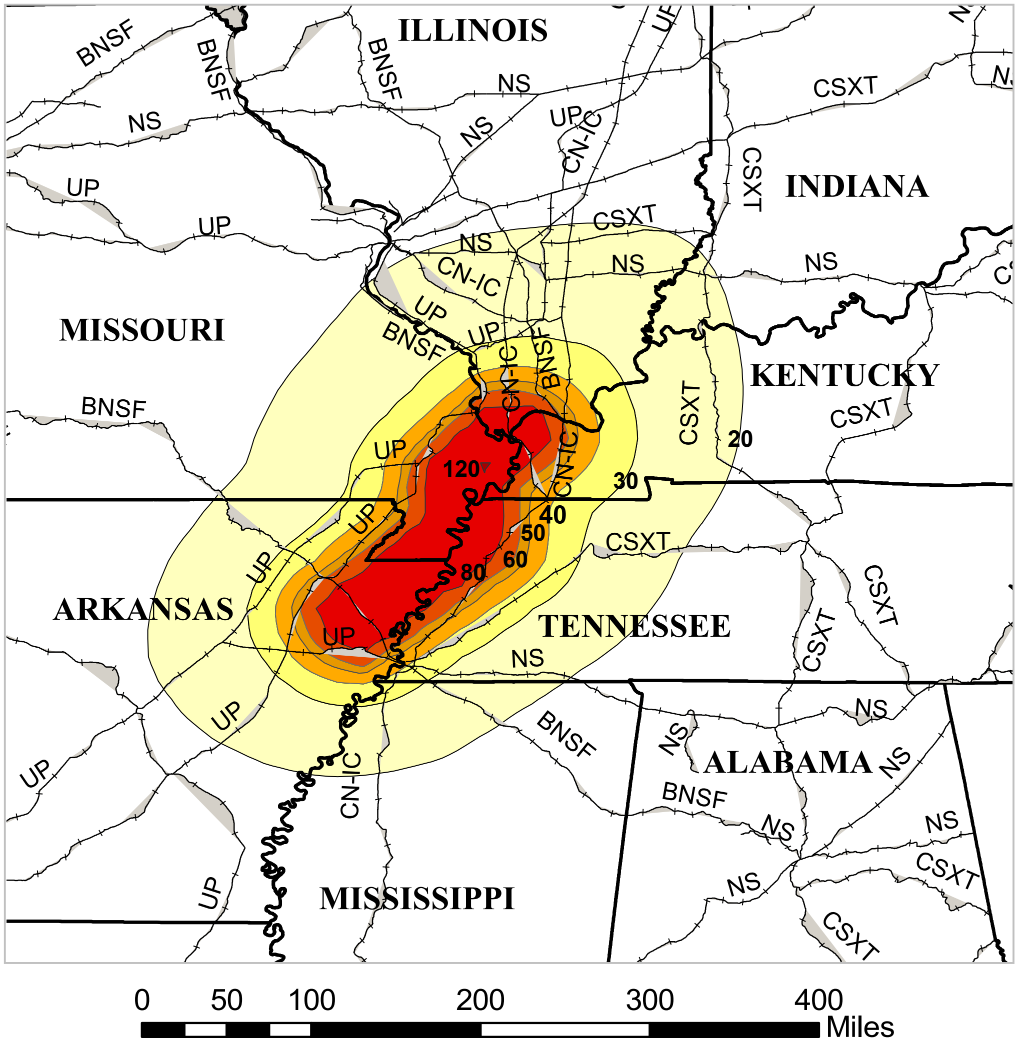

Figure 3.6. Mid-America Rail Lines and PGA Contours with a 2% Probability of Occurrence in the Next 50 Years

	Railroad (route miles)					
PGA	BNSF	CN-IC	CSXT	NS	UP	Total
20 - 40% g	220	396	332	212	501	1,661
41 - 70% g	42	133	0	0	183	358
71 - 100% g	37	44	0	0	8	89
Total Miles	299	573	332	212	692	2,107
Percentage in High Liquefaction Area	33%	17%	< 10%	< 10%	85%	40%

Table 3.4. Railroad Route Mileage Potentially Exposed to PGA Values with a 2% Probability of Occurrence in the Next 50 Years

3.3 Summary of Route Mileage Exposure

A total of approximately 298 railroad route miles are in areas with a 5% probability of experiencing PGA values sufficient to cause moderate damage. Thirty six percent of these 298 route miles are in areas of high soil liquefaction potential. There are approximately 2,107 route miles in areas with a 2% probability of experiencing PGA values sufficient to cause moderate to severe damage. Forty percent of these are in areas of high soil liquefaction potential.

3.4 Major Railroad Bridges at Risk

Bridges are the principal railroad infrastructure feature of interest in the region. There is a general agreement in the railroad bridge engineering community that railroad bridges generally suffer less damage in earthquakes than highway bridges (Byers, 1996), but it is unclear what percentage of railroad bridges might survive a major earthquake with minimal damage. Hwang (2000) has developed fragility curves for highway bridges that estimate the probability of bridge failure as a function of ground motion. Unfortunately no similar curves have been developed for railroad bridges.

The railroads are primarily concerned with damage to bridges of substantial length due to significant repair time and extensive detour lengths (AREMA 9, 2001). The Mid-America earthquake region has two major waterways, the Mississippi River and the Ohio River, and there are only a limited number of railroad crossings. Eight major railroad river crossing bridges, carrying moderate to high traffic densities, are in locations that may experience substantial peak ground accelerations (Table 3.5). Aerial photographs of these bridges are presented in Appendix D.

To determine the significance of these bridges to the nation's rail freight traffic, the following equation was used to calculate the percentage of freight tons that cross these bridges annually:

$$P = (D*(RT/G)) / RC \qquad (3.1)$$

where: P = percentage of traffic crossing over the major Mid-America bridges

D = Total density over major Mid-America bridges (in gross tons)

RT = Revenue ton-miles

G = Gross ton-miles

RC = Revenue tons carried

For the eight bridges of interest, the following values were used to calculate the amount of freight that crosses these bridges annually:

D = 509 MGT (USDOT, 2001) G = $2.982*10^{12}$ ton-miles (AAR, 2000)

RT = $1.433*10^{12}$ ton-miles (AAR, 2000) RC = $2.155*10^{9}$ tons (AAR, 2000)

These eight bridges carry approximately 245 million tons of freight per year, which accounts for 11.4% of the total rail freight originated in the United States annually.

Railroad	River	Location	Density (MGT)	2% PGA
CN-IC	Ohio	Cairo, IL	40	0.9
Union Pacific	Mississippi	Thebes, IL	95	0.8
BNSF (CN-IC)	Ohio	Metropolis, IL	50	0.6
BNSF	Mississippi	Memphis, TN	65	0.4
Union Pacific	Mississippi	Memphis, MO	99	0.2
CSXT	Ohio	Henderson, KY	60	0.25
TRRA	Mississippi	St. Louis, MO	50	0.2
TRRA	Mississippi	St. Louis, MO	50	0.25

Table 3.5. Major Railroad River Crossing Bridges in the Mid-America Region

The bridges in Table 3.5 are arranged in descending PGA order. The first three bridges, located at Cairo, Thebes and Metropolis, are all within the region that has a 2% chance of experiencing severe PGA values (substantially above 0.4 g) in the next 50 years. The CN-IC Railroad owns the Cairo bridge and has trackage rights on the Metropolis bridge, which is owned by the BNSF. UP owns the third bridge, located at Thebes. There are five bridges in areas that may experience moderate PGA values (0.2 – 0.4 g). Two of these bridges are located in Memphis, Tennessee, one owned by UP and the other by the BNSF Railway. CSXT owns a bridge over the Ohio in the moderate ground motion area, located in Henderson, Kentucky. The Terminal Railroad Association (TRRA) of St. Louis, which is jointly owned by the Class 1 railroads in the region, owns two bridges, both in St. Louis. All eight bridges carry moderate to high traffic densities and are in areas of high liquefaction potential, thus increasing the probability of severe damage given a major earthquake.

3.5 Analysis of One Major Railroad: Exposure to Soil Liquefaction

A major railroad in the region provided their GIS database. I used this database to develop summary statistics for this railroad's bridges and route mileage within the entire seven-state Mid-America region in areas of high and low liquefaction potential. Using GIS software, I performed an overlay of the network of this railroad and the liquefaction potential of the soils in the Mid-America region. The GIS data were divided into two infrastructure types: track and bridges. The data were sorted by liquefaction category to determine the number of bridges and the route mileage of track potentially exposed to liquefaction in the region. The GIS data showed that 40% of this railroad's route miles (approximately 1,250 miles) are in areas of high liquefaction potential (Figure 3.7).

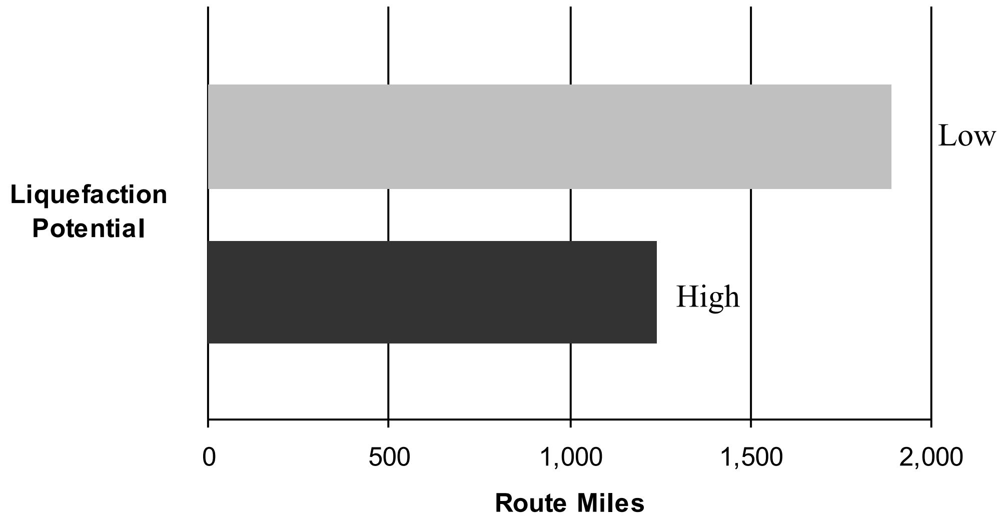

Figure 3.7. Route Miles per Liquefaction Zone for One Major Railroad in the Seven-State Mid-America Region

It was also determined that 48% of this railroad's bridges (approximately 889 bridges) are in areas of high liquefaction potential (Figure 3.8).

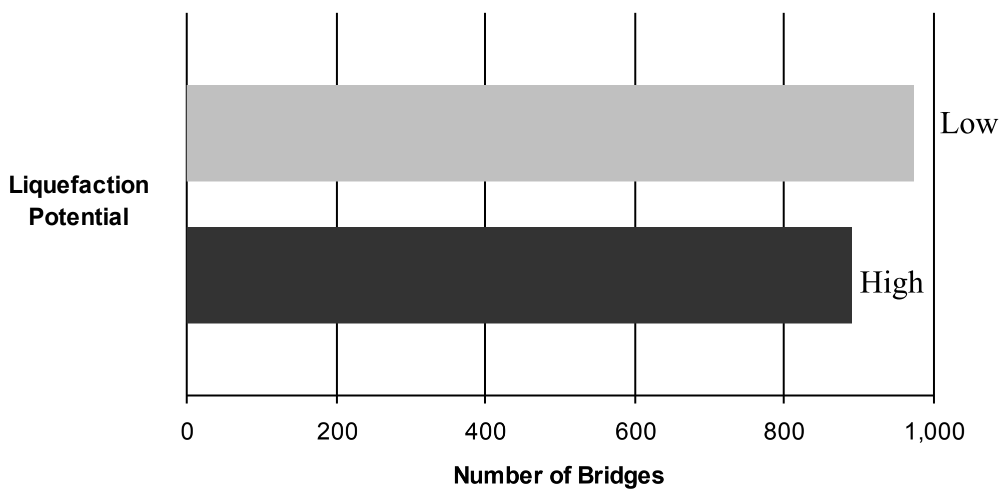

Figure 3.8. Bridges per Liquefaction Zone for One Major Railroad in the Seven-State Mid-America Region

The relative local ground motion between adjacent piers of a bridge may vary. Each individual pier may experience differing ground motions due to the local characteristics of its soil foundation. Thus, longer bridges can be expected to have a higher likelihood of failure than shorter bridges for two reasons:

1) Longer bridge spans are more likely to experience greater relative displacement and misalignment in an earthquake because of localized differences in soil attributes.
2) Longer bridges have a greater chance of misalignment or failure due to the exposure of an increased number of spans as compared with shorter bridges.

The GIS data provided for the railroad bridges were further broken down by length and material type to investigate the relationship between these parameters and liquefaction potential. The distribution of bridge lengths for this railroad is presented in Figure 3.9. Approximately half of their bridges are less than 80 feet in length (note the x-axis is a log scale) and 90% are less than 290 feet. Only 1% of their bridges are greater than 1,660 feet, with the longest bridge being 5,015 feet in length.

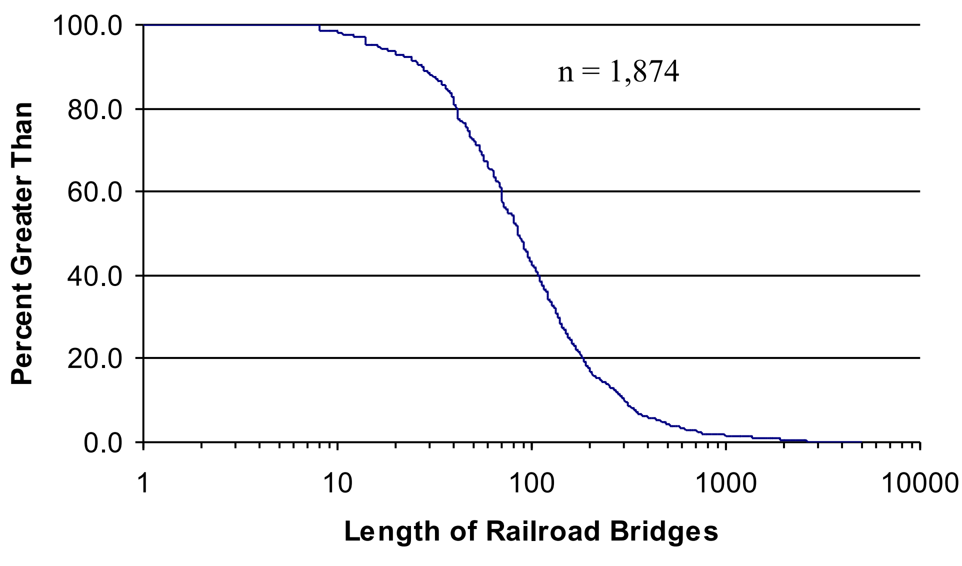

Figure 3.9. Distribution of Bridge Length for One Major Railroad in the Seven-State Mid-America Region

Soil liquefaction is a phenomenon of saturated soils, such as those commonly found in river valleys like the Mississippi and Ohio. River valleys also tend to be areas where longer bridges are required, suggesting that longer bridges may be more likely to occur in areas with high soil liquefaction potential. To test this hypothesis, I compared the distribution of bridge length to high and low soil liquefaction potential (Figure 3.10). The results of a chi-squared analysis showed significant heterogeneity in the expected direction ($\chi^2 = 15.2$, $p<0.01$, df = 5). The shortest bridges were more likely to be found in areas with low liquefaction, and the longest bridges (> 500 feet) were more than twice as likely to be located in areas with high liquefaction potential. This finding, that the longest bridges (and thus most costly and difficult to replace) are most likely to be exposed to soil liquefaction, underscores the importance of understanding more about soil liquefaction and bridge performance in earthquakes.

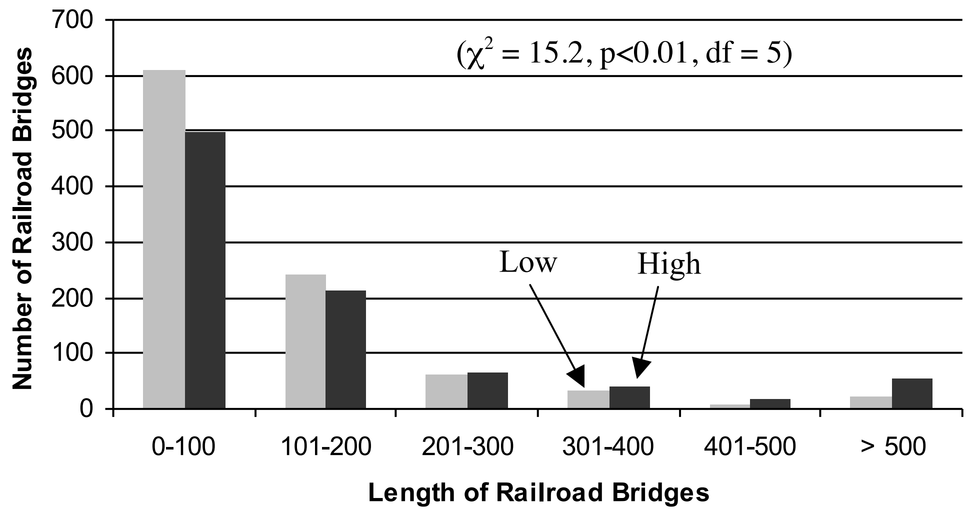

Figure 3.10. Bridges in the Seven-State Mid-America Region for One Major Railroad by Length

The railroad bridges in the Mid-America earthquake region are constructed primarily of steel, timber, concrete or a combination of these. A chi-squared analysis was conducted to determine the relationship between bridge construction material and exposure to liquefaction potential. The analysis showed no significant relationship between bridge construction material and exposure to liquefaction ($\chi^2 = 2.90$, $p>0.5$, df = 5) (Figure 3.11). For this major railroad, approximately 75% of the bridges are constructed of steel, concrete, or a combination of these.

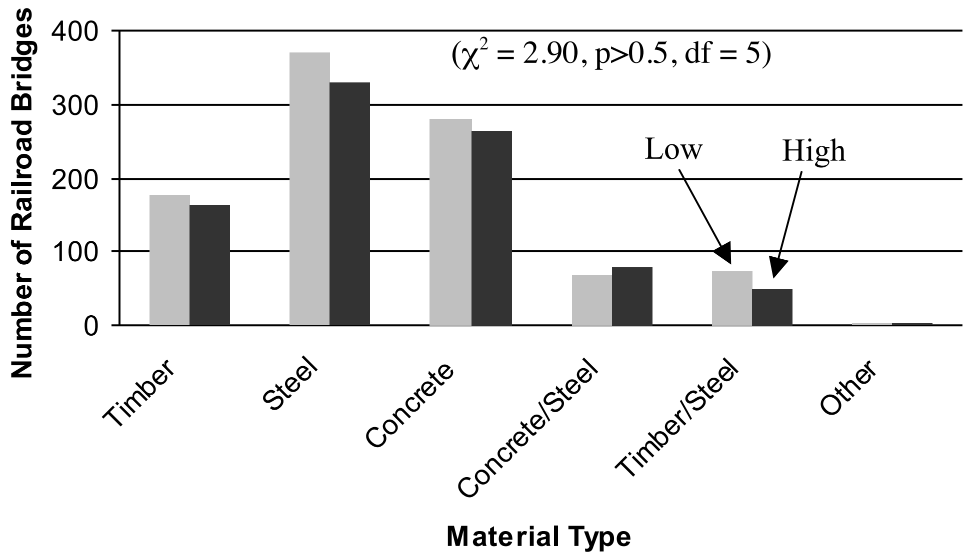

Figure 3.11. Bridges in the Seven-State Mid-America Region for One Major Railroad by Material Type

3.6 Railroad Bridge Estimates for the Mid-America Region

An objective of this study was to develop estimates of railroad bridge exposure to liquefaction and high seismic ground motion for all the railroads in the Mid-America region. However, detailed GIS data on infrastructure characteristics were not available for the other railroads in the region. Therefore, I developed a method to estimate these figures by using the data for one railroad to extrapolate for the others. The construction material type and the length of the bridges were estimated in a similar way. The major railroad used to extrapolate these estimates has 299 route miles in areas potentially exposed to damaging PGA values (> 20% g), which represents 14.1% of the total route mileage in damaging PGA areas. Within the seven-state Mid-America region, this railroad has approximately 0.59 bridges per route mile (or 1 bridge per 1.7 miles). In areas potentially exposed to damaging peak ground accelerations for a 2% probability earthquake with a fifty-year return period, there are approximately 0.61 bridges per route mile (or1 bridge per 1.6 miles). I used the formula below to estimate the number of bridges in areas potentially exposed to large seismic ground motions:

$$\hat{H} = (N'/R')*R \tag{3.2}$$

where: $\hat{H}$ = Estimated number of bridges in damaging PGA areas

N' = Number of railroad bridges in damaging PGA areas for one railroad

R' = Route mileage in damaging PGA areas for one railroad

R = Total route miles in damaging PGA areas for all railroads

I used the USDOT Bureau of Transportation Statistics (BTS) GIS data containing information about the other railroads to determine their route mileage (R). I applied the rate of bridges per mile for one major railroad in areas capable of producing damaging peak ground accelerations with a 2% probability in 50 years (0.61 bridges/mile) to estimate the total number of railroad bridges in high seismic ground motion areas. The number of railroad bridges in areas capable of producing damaging ground motions is estimated to be 2,082. Of these, 59% (approximately 1,237) are in areas with high liquefaction potential (Figure 3.12).

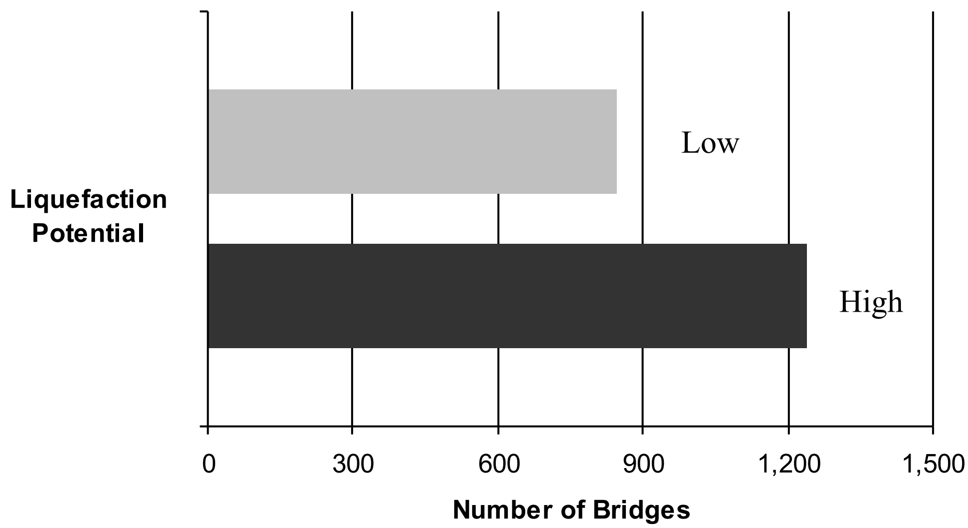

Figure 3.12. Estimated Number of Bridges in High Seismic Ground Motion Areas per Liquefaction Zone

I used a similar approach to develop estimates for bridge length and construction material type. Of the estimated 2,082 bridges, 16% (approximately 337) are greater than 200 feet in length (Figure 3.13), and 77% (approximately 1,482) are constructed of concrete, steel, or a combination of these (Figure 3.14).

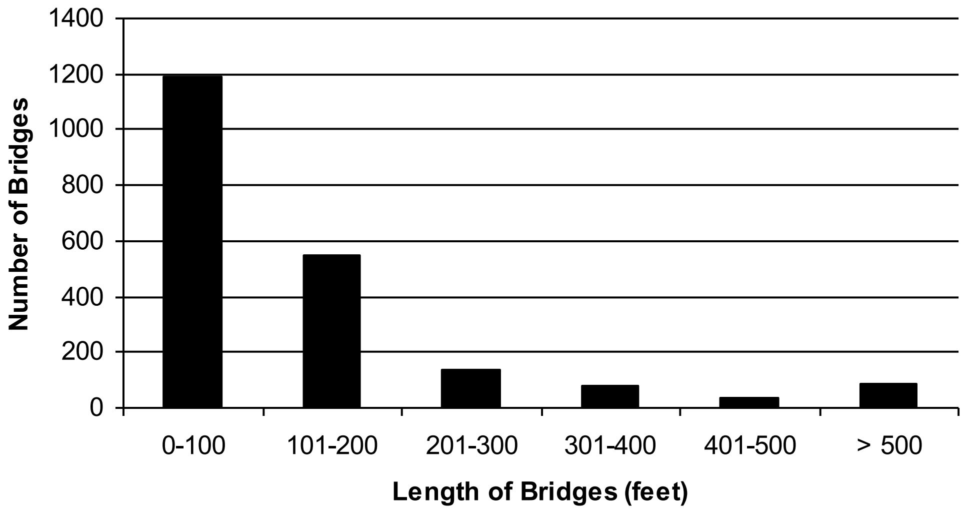

Figure 3.13. Estimated Number of Railroad Bridges in High Seismic Ground Motion Areas for all the Railroads in the Region

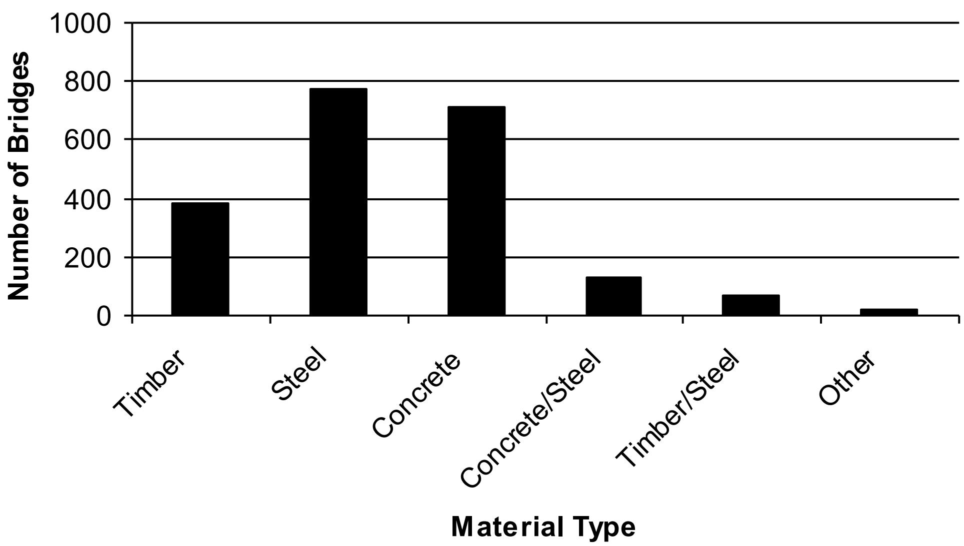

Figure 3.14. Estimated Number of Railroad Bridges in High Seismic Ground Motion Areas for all the Railroads in the Region by Material Type

In summary, the bridge data for one major railroad were used to estimate the total number of bridges for all the railroads in the region, in areas of high seismic ground motion (2% probability earthquake in the next 50 years). For the one railroad, the total number of bridges in the seven-state region was 1,861. This yielded a rate of 0.59 bridges per mile.

The total length of bridge for this major railroad in the seven-state region equals 277,023 feet (52.47 miles), producing an average bridge length of 149 feet. In areas capable of experiencing high seismic ground motion (20% g or greater), the total length of bridge for this major railroad equals 50,099 feet (9.49 miles), yielding an average bridge length of 147.4 feet. Assuming that bridge length and bridges per mile for this railroad are representative of the other major railroads in the region, these statistics can be used to extrapolate the figures in Table 3.6. The total number of railroad bridges estimated to be in areas with the potential of experiencing damaging ground motions is 2,082. Multiplying 2,082 by the average bridge length for one major railroad (147.4 feet), I estimated the total length of bridge for all the railroads in the region to be 306,784 feet (58.10 miles).

	Number of Bridges	Total Length of Bridges	
		Feet	Miles
One Major Railroad	340	50,099	9.49
All Mid-America Railroads	2,082	306,784	58.10

Table 3.6. Number of Bridges and Total Length of Bridge in Areas Capable of Experiencing Damaging Ground Motions

CHAPTER 4
COST EFFECTIVENESS MODEL FOR UPGRADING RAILWAY BRIDGES

4.1 Cost Analysis

There are approximately 2,082 railroad bridges in areas with the potential to experience damaging peak ground accelerations (20% g or greater) in the Mid-America region. The total length of these bridges is 306,784 feet (58.10 miles). The large number of bridges at risk indicates the possibility for a railroad network disruption exists. There are two options available to the railroads for handling the possibility of a major earthquake in the Mid-America region. One option is to defer preventive action, and repair whatever damage actually occurs in the event of a major earthquake. The second option is to retrofit some or all of their bridges to reduce the probability and magnitude of bridge failure, thus improving their resistance to seismic activity.

We can calculate the replacement cost for all of the bridges in the region. This is, of course, an extreme and unrealistic scenario. Even in a severe earthquake, not all of the bridges in the region will fail. However, this scenario does provide an upper bound for the direct financial impact of bridge replacement cost. The following equation is used to determine the replacement cost:

$$\text{Total Replacement Cost} = TL*C \tag{4.1}$$

where: TL = Total length of all bridges
C = Replacement cost per track-foot

Railroad engineers estimate that the average replacement cost of bridges is $2,100/track-foot (AREMA 9, 2001). Based on this replacement figure, the total cost to replace all 2,082 bridges would be $644,246,400. Although the worst-case scenario would be considerably less than this, it is difficult to quantify what it actually might be. The estimated PGA maps and other data provided by the USGS allow probabilistic estimates about the epicenter, magnitude and consequent local ground motions that the bridges in the region may experience. Even with these values for a particular location, the lack of railroad bridge fragility curves precludes quantification of the number and location of railroad bridges that might fail. Given the generally good performance of railroad bridges in the past and the size of the region, it is likely that only a small fraction would actually fail, so adopting a "wait and see" strategy for most railroad bridges in the region may generally be a rational approach.

Alternatively, railroads could retrofit some or all of their bridges in the region to reduce their probability of failure in an earthquake. Some options for retrofitting include: replacing bearings and anchor bolts with a more seismically resistant design, adding restraining devices to bridge spans, encasing piers, or increasing the size and stability of bridge footings (Foutch, 2002). To estimate the cost of retrofitting railway bridges in the region, I assumed the preferred retrofit method is to replace bridge bearings with more seismically resistant designs. The cost to retrofit bearings on a typical 100-foot simple span is estimated to be \$15,000 per bearing (Dooley, 2002). The following assumptions were made to determine the cost to retrofit all railroad bridges capable of experiencing damaging ground motions in the region:

- Bridge spans are simply supported.
- Each span is connected with 4 bearings (two at each end).
- Each pier supports 4 bearings.
- Each abutment supports 2 bearings.
- Average span length is equal to approximately 75 feet.

As a rough guideline, bridge spans less than 100 feet in length are constructed as girder spans. Bridge spans greater than 100 feet are usually constructed as trusses. I estimated the average bridge length in the Mid-America region to be 147 feet in length. Assuming a majority of the bridge spans in the region are girder spans, I divided the average bridge length by two and used an average span length of approximately 75 feet to determine the total retrofit cost. The following expression was used to estimate the cost associated with retrofitting all the railway bridges in areas capable of experiencing damaging peak ground accelerations:

$$\text{Total Retrofit Cost} = S*B*BC \qquad (4.2)$$

where: S = Total number of bridge spans

B = Number of bearings per span

BC = Retrofit cost per bearing

The total cost to retrofit all railroad bridges in areas capable of experiencing damaging ground motions is estimated at $245,427,200. Although lower than the replacement cost figure, it is still substantial, and the difficulties in predicting which bridges might actually fail complicate the decision to retrofit. The associated cost, the generally satisfactory performance of railway bridges in past earthquakes and the uncertainty about which bridges would actually benefit all argue against the retrofit approach for small to medium-size bridges. Furthermore, railroads are generally capable of replacing short spans and small bridges quite rapidly (AREMA 9, 2001 & Johnson, 2002). This ability for rapid replacement, combined with the large cost to retrofit railway bridges, makes railroads less concerned with short to medium-sized bridges.

However, replacing large trusses and lengthy deck truss spans characteristic of the major river-crossing bridges is another matter. Their repair would require long lead times for design, manufacturing, fabrication and delivery. The possibility of damage to multiple bridges could also mean a shortage of personnel, and potentially limit access needed for concurrent reconstruction of large bridges in the region. Therefore, my analysis of the railway bridges in the region concentrates on the major river crossings. To examine the large river-crossing bridges in the region, I developed a model to determine the cost-effectiveness of upgrading these railway bridges.

4.2 Model Introduction

There are 85 railway bridges greater than 500 feet in length that are estimated to be exposed to substantial peak ground accelerations in the Mid-America region (Table C-7). The following analysis concentrates on eight major railroad river-crossing bridges in the Mid-America region (Table 3.5) because of their importance to the rail network. The analysis could also be applied to scenarios involving other large railway bridges in the region.

The model uses a net present value (NPV) analysis to compare the cost of retrofitting with the risk cost associated with not retrofitting in current dollars. Input parameter values are inserted into the model and it produces an expected net present monetary value associated with retrofitting the bridge and not retrofitting the bridge. The difference in monetary value between retrofitting and not retrofitting the bridge represents the risk associated with the decision. This risk value can be compared versus the cost of retrofitting to assess upgrade possibilities. The model can be used as an aid in determining the cost-effectiveness of upgrading railway bridges.

There are significant parameters that railroads need to consider in determining to retrofit or not retrofit major railway bridges. If the bridge is retrofitted, the railroad has the immediate cost to retrofit the bridge. The railroad gains a bridge with greater resistance to high seismic ground motion, but this provides no benefit until an earthquake occurs causing ground motion sufficient to damage the bridge if it were not retrofitted. Thus the benefit is in the reduced probability of failure and magnitude of damage to the bridge. If the bridge is damaged, it is assumed less time would be needed to repair it. Thus, there would be a lower cost of detouring traffic over alternative routes. If the bridge is not retrofitted, the railroad has no immediate cost. However, given a major earthquake, it is assumed there is a higher probability of bridge failure and/or damage to the bridge. More time would be needed to repair the damaged structure, thus leading to higher detour costs. The higher detour costs result from having to use a longer route on the same railroad or the extra cost associated with detouring trains over another railroad.

4.3 Model Development

Determining whether to upgrade major railway bridges is a decision problem for the owner. Is it cost-effective to retrofit the bridge given current probability estimates? To develop the model, a decision tree with two alternatives was created. The decision tree is a visual aid for the decision process (Figure 4.1). It consists of decision nodes and chance nodes. The decision node (square) in the model provides two alternatives, to retrofit or not retrofit. After an alternative is chosen, the possible outcomes stemming from that alternative are represented by chance nodes (circular).

Chance nodes A and B account for the exceedance probabilities of various ground motions. An example is the probability of exceeding 0.4 g (40% g) ground motion, which equals 0.03 (P(Exd 0.4 g) = 0.03). Chance nodes one through eight account for the failure probability of the bridge given that it was retrofitted or not retrofitted. An example is the probability of an F_1 type bridge failure given the bridge is retrofitted is 0.53 ($P(F_1 \mid R) = 0.53$). The outcomes from each chance node are mutually exclusive and collectively exhaustive (Ang and Tang, 1990), so the probabilities associated with each branch stemming from the chance nodes must add up to one.

Two types of bridge failure are defined for the model. An F_1 type failure is defined as a bridge having major damage. For an F_1 type failure, the time to repair and cost to repair is substantial. An F_2 type failure is defined as a bridge having minor to medium damage. For an F_2 type failure, the time to repair is minimal depending on the damaged bridge components. A bridge having no measurable damage is denoted by $\overline{F}$.

In decision analysis, the objective is to make the "best" decision (Ang and Tang, 1990). For this model, the utility values are stated in terms of the respective probabilities for each alternative. Since each alternative can be expressed in monetary value, the model uses the maximum expected monetary value (EMV) criterion, discounted to determine the net present value of the risk associated with retrofitting and not retrofitting. The net present value approach discounts future net benefits and costs to their present value (Sassone and Schaffer, 1978). The NPV approach is used in the model because it reduces the stream of future costs associated with retrofitting and not retrofitting into a single comparable cost. The difference between the NPV values of risk for retrofitting and not retrofitting is then compared with the cost of retrofitting the bridge to determine the

Figure 4.1

optimal alternative. The optimum alternative is the one whose expected monetary value is

$$d(a_{opt}) = \max_i \left\{ \sum_j p_{ij} d_{ij} \right\} \qquad \text{(Ang and Tang, 1990)} \qquad (4.3)$$

where $d(a_{opt})$ = optimal alternative

d_{ij} = monetary value of the jth consequence associated with alternative i

p_{ij} = the corresponding probability

This approach allows the decision maker to systematically weigh the value of each outcome by the corresponding probability (Ang and Tang, 1990).

4.4 Model Parameters and Output

The model allows the user to input values for parameters related to the specific bridge in question. Model input variables include:

- Earthquake probabilities (USGS, 1998)
- Probability of Bridge Failure (estimated based on ground motion)
- Repair Cost given Failure Type ($ per track foot) (AREMA 9, 2001)
- Retrofit Cost ($ per bearing) (Dooley, 2002; Byers, 2002)
- Detour Length (miles)
- Time to Repair the Bridge (days)
- Bridge Length (feet)
- Detour Cost (including variable, fixed and lost revenue costs)
- Annual Tonnage (MGT) (USDOT, 2001)
- Number of main spans and approach spans

The model uses the probability of bridge failure in the decision process. Due to the lack of fragility curves for railroad bridges, the model uses the assumptions below for bridge failure probability. It must be stressed that these are hypothetical values, solely for the purpose of testing and illustrating the model and its output.

- The probability of bridge failure without retrofitting is estimated based on probable ground motions within the region.
- The seismic resistance gained by retrofitting follows the subsequent scale:
  - ♦PGA > 0.8 g ~ 25% reduction in failure probability
  - ♦0.2 g > PGA > 0.8 g ~ 50% reduction in failure probability
  - ♦0.05 g > PGA > 0.2 g ~ 75% reduction in failure probability

The model takes the input values for each of the input parameters and produces an expected monetary value of risk based on the maximum EMV criterion for retrofitting or not retrofitting the bridge. The monetary value of risk is compared with the cost of retrofit to determine the optimum alternative in the decision process. The model output may be used as an aid to decide if retrofitting major railroad bridges is a cost-effective alternative.

4.5 Mathematical Equation for Cost-Effectiveness Model

The mathematical expression for the cost-effectiveness model described is as follows:

B > FC ~ cost-effective to retrofit railway bridges

where: B = Risk cost

FC = Retrofit cost

$$B = K_1 - K_2 \tag{4.4}$$

where:

$$K_1 = \sum_{k=1}^{4} \sum_{i=0}^{50} D_1 \times d_i \times S_k \tag{4.5}$$

$$K_2 = \sum_{k=1}^{4} \sum_{i=0}^{50} D_2 \times d_i \times S_k \tag{4.6}$$

where: d_i = discount factor from year 0 to year 50

S_k = Exceedance probability per year for kth scenario

(when k = 1, S_1 = P(Exd. 0.8 g)/50; k = 2, S_2 = P(Exd. 0.4 g)/50;

k = 3, S_3 = P(Exd. 0.2g)/50; k = 4, S_4 = P(No earthquake)

$$D_1 = P(F_1R)*\{(DC|_{F1}*T_{F1}) + RC_{F1})\} + P(F_2R)*\{(DC|_{F2}*T_{F2}) + RC_{F2})\} + P(F_0R)*\{(DC|_{F0}*T_{F0}) + RC_{F0})\}$$

$$D_2 = P(F_1R|_0)*\{(DC_{F1}*T_{F1}) + RC_{F1})\} + P(F_2R\ |_0)*\{(DC_{F2}*T_{F2}) + RC_{F2})\} + P(F_0R|_0)*\{(DC_{F0}*T_{F0}) + RC_{F0})\}$$

where: $F_0 = \overline{F}$ = No measurable damage

F_1 = Bridge failure with major damage

F_2 = Bridge failure with minor to medium damage

R = Retrofit

$R_0 = \overline{R}$ = No retrofit

DC_j = Detour cost per jth failure type (j = 0, F_0 failure type, etc.)

T_j = Time to repair per jth failure type

RC_j = Bridge repair cost per jth failure type

4.6 Model Example in the Mid-America Region

The Mid-America region has eight major bridges of interest in six different locations (Table 3.5, Figures D-1 – D-7). Some of these bridges are close to one another. If one of the major bridges in the region is out of service, the railroad that owns the bridge must find an alternative for moving freight traffic over alternate routes. If a major earthquake were to occur, a variety of network disruption scenarios are possible. One example of a network disruption in the Mid-America region is for the Canadian National-Illinois Central railroad. The CN-IC's main line from Chicago to New Orleans splits into two sections at Edgewood, Illinois and rejoins again at Fulton, Kentucky. Both main lines cross the Ohio River, the CN-IC Railroad owns one of the bridges, at Cairo, Illinois, and has trackage rights on the other, at Metropolis, Illinois (Figure 4.2). A major earthquake could put one or both of these bridges out of service for an extended period of time. If one of the bridges was lost, and assuming there were adequate capacity, the CN-IC railroad could detour its traffic over its other line with little disruption or extra cost. In this case, there would be no substantial detour cost for shifting the traffic to the line still in service, since it is owned by the CN-IC railroad (Figure 4.3).

However, if both of the bridges CN-IC railroad uses were put out of service by a major earthquake, the CN-IC railroad would need to detour its traffic over another railroad. Line capacity permitting, the shortest detour would likely be the best option. In this example, the shortest detour for the CN-IC is over the Union Pacific and Burlington Northern Santa Fe. The CN-IC has an interchange with the UP at Tamaroa, Illinois. This connection allows the CN-IC to cross the Mississippi River on UP trackage at Thebes, Illinois, connect with the BNSF at Rockview, Missouri and return to their own tracks at Memphis, Tennessee (Figure 4.4). In this case, the detour mileage over the UP and BNSF is 250 miles.

Due to the location of the two CN-IC bridges and the UP bridge, all in areas with a high probability of experiencing severe PGA values (> 40% g), a third scenario exists. For this scenario, the Union Pacific bridge at Thebes, Illinois also suffers damage in a major earthquake, along with the Metropolis and Cairo Bridges, and all three bridges would be out of service for an extended period of time (Figure 4.5). In this situation, the shortest route for the CN-IC is to use their line from Gilman, Illinois to Springfield, Illinois, and then use trackage rights on Union Pacific from Springfield to St. Louis, MO. After reaching St. Louis, CN-IC traffic would use the BNSF line to Memphis, Tennessee, and then return to their mainline. In this case, the detour mileage over the BNSF is 278 miles.

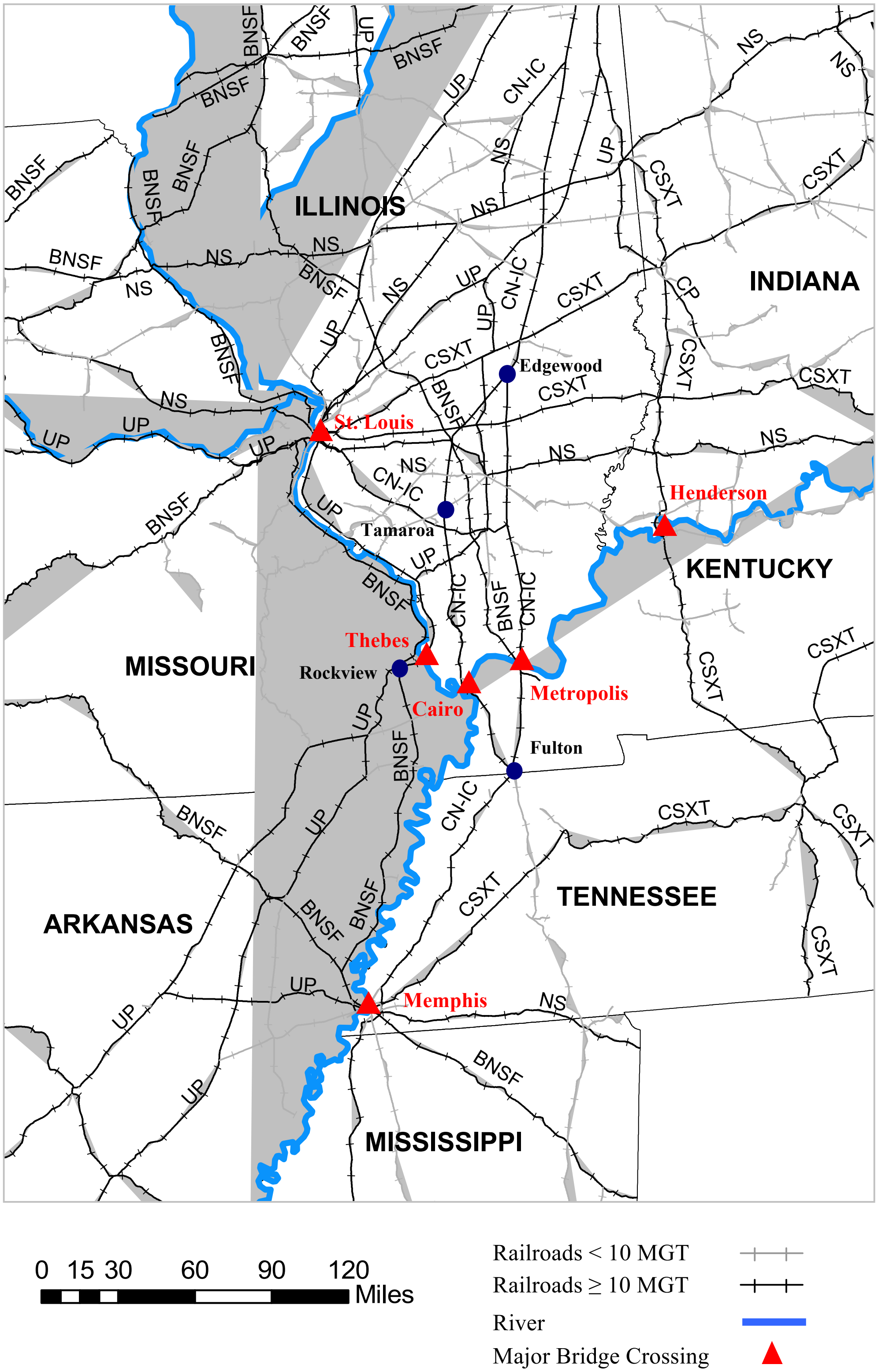

Figure 4.2. Map of Railroad Lines in the Mid-America Earthquake Region

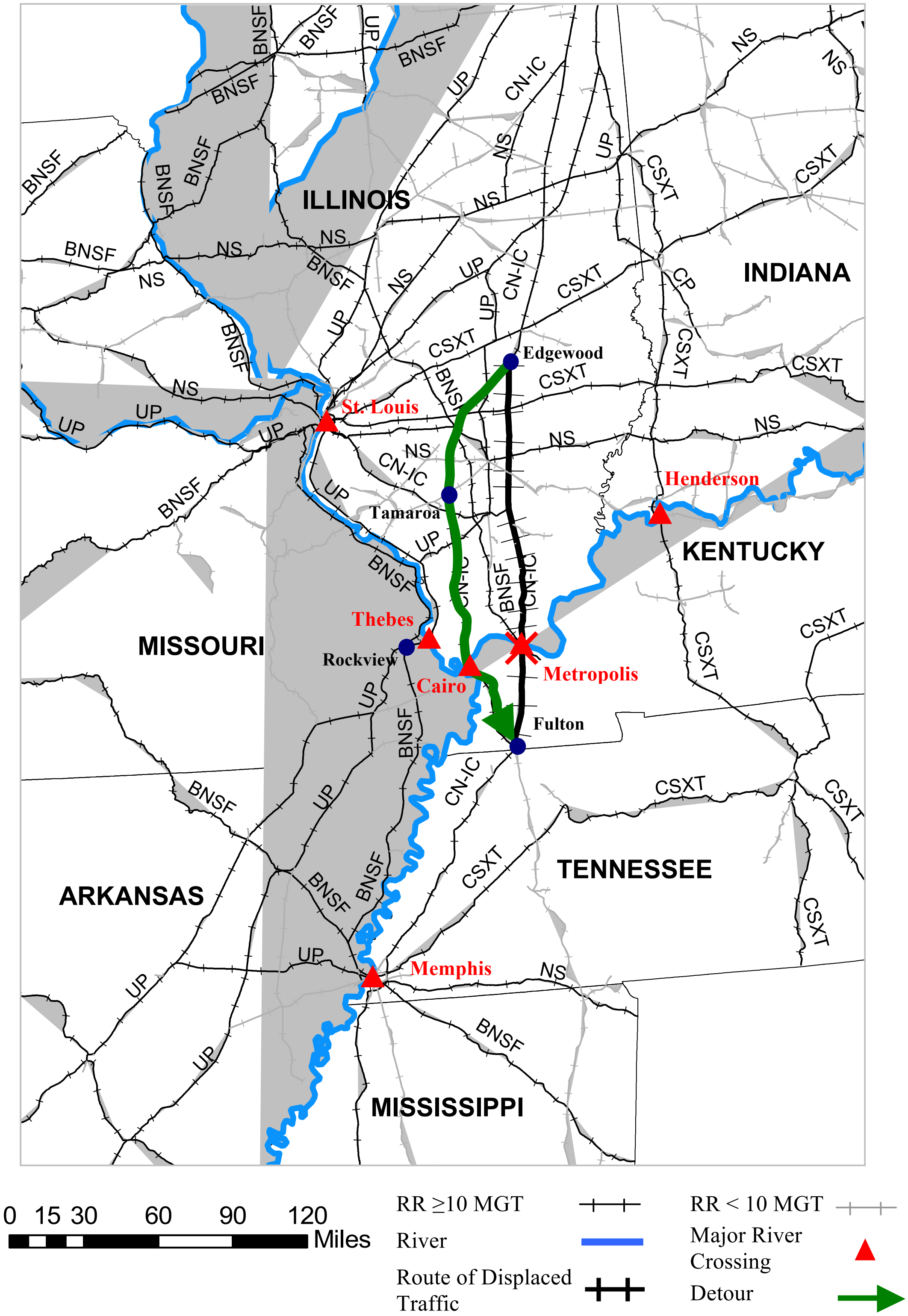

Figure 4.3. Example Detour Scenario for CN-IC Railroad with the Metropolis Bridge Out of Service

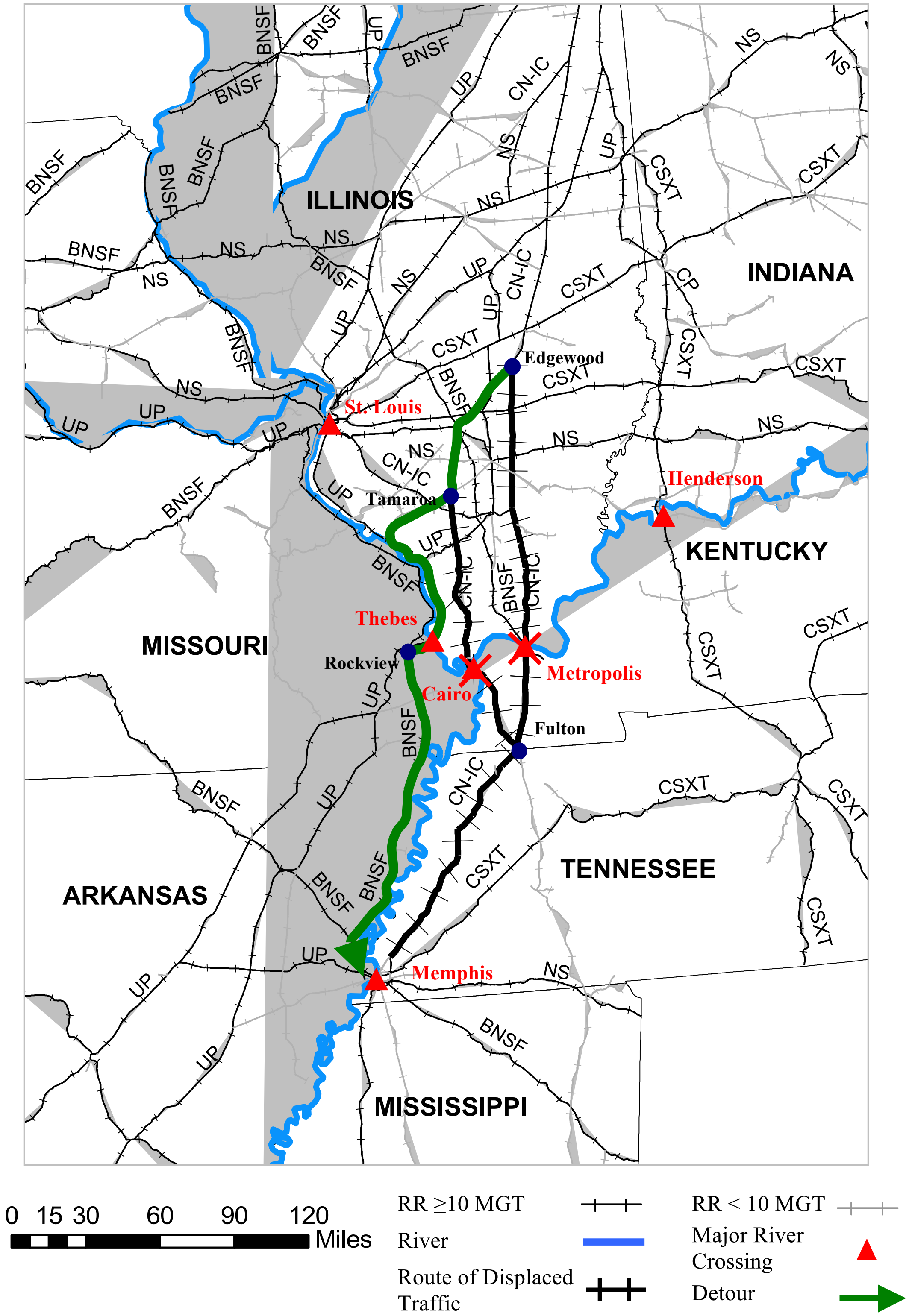

Figure 4.4. Example Detour Scenario for CN-IC Railroad with both the Metropolis Bridge and the Cairo Bridge Out of Service

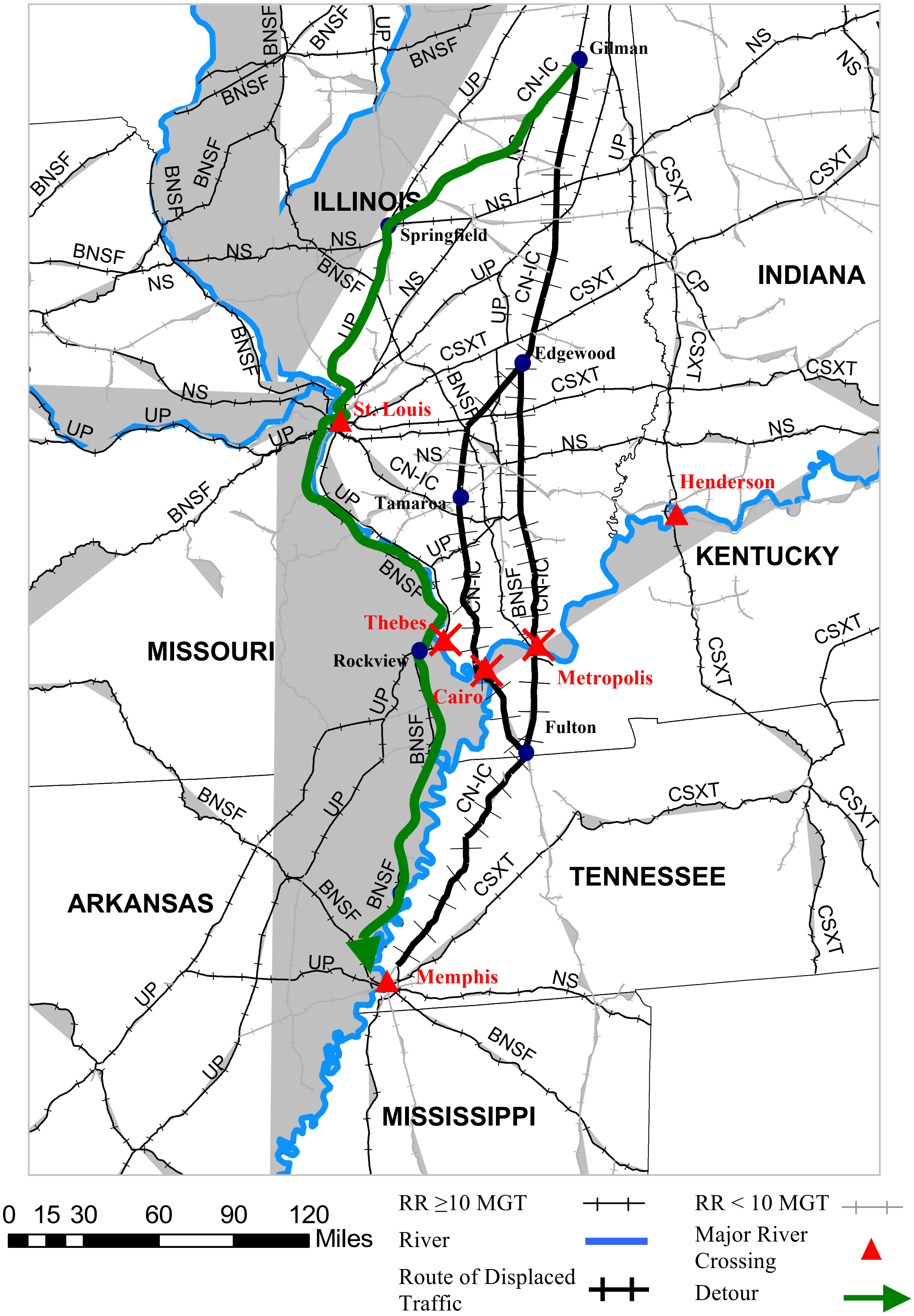

Figure 4.5. Example Detour Scenario for CN-IC Railroad with the Metropolis Bridge, the Cairo Bridge and the Thebes Bridges All Out of Service

The scenario described above assumes adequate capacity being available to handle the increase in traffic over the alternate lines. Some of the factors affecting the capacity of rail lines include (Salzman, 2002):

- Track speed
- Traffic type (intermodal, coal, auto parts, etc.)
- Traffic control system
- Distance between sidings (single track)
- Percent grade and maintenance cycles
- Level of service provided

In fact, substantial excess capacity may not exist on some of the routes that would have to be used. A more thorough capacity analysis would need to be completed to determine if the hosting railroad could manage the increase in traffic.

For the above example, input parameters were inserted into the model to determine the EMV risk cost associated with the possible scenario explained above. For CN-IC, presumably the best option is to retrofit one of the two bridges on their mainlines to reduce the probability of damage given a major earthquake. This would allow CN-IC to keep traffic on their lines as much as possible, capacity permitting. I assumed the CN-IC would choose to retrofit the Metropolis bridge because: 1) it handles more traffic, 2) it is smaller in length and 3) it is in a lower ground motion area (meaning a higher seismic resistance gained by retrofitting). CN-IC provided data on the number of approach spans and main spans for the CN-IC river-crossing bridge at Metropolis (Luciano, 2002). The following input parameter values were used to determine the EMV risk cost associated with retrofitting and not retrofitting for the Metropolis bridge:

- Bridge length = 5,660 feet
- Number of main spans = 7
- Number of approach spans = 39
- Annual MGT = 50 MGT

- Detour costs: variable cost = \$2.5/mile (Railroad Facts, 2000)
  fixed cost = \$62/train (Railroad Facts, 2000)
  lost revenue cost = \$0.024/mile (Railroad Facts, 2000)
- Repair cost for F_1 type failure = \$2,100/track foot
- Repair cost for F_2 type failure = \$700/track foot
- Cost to improve bearings = \$15,000/bearing for approach spans (Dooley, 2002)
  \$65,000/bearing for main spans (Byers, 2002)

A discount rate of 5% (average between year 2000 and 2001) was used for the NPV analysis (Council of Economic Advisors, 2002). If the risk cost exceeds the retrofit cost, retrofitting railway bridges might be more cost-effective. The model output shows if the detour length and time to repair are large, there are plausible scenarios in which retrofitting may be a rational choice (Table 4.1).

Detour Length (miles)	Time to Repair (days)	EMV Risk Cost (\$)		Risk Cost (\$)	Retrofit Cost (\$)	Risk - Retrofit (\$)
		Retrofit	No Retrofit			
70	F_1=365 F_2=14	492,120	815,294	323,174	4,160,000	-3,836,826
250	F_1=750 F_2=28	2,986,536	4,872,563	1,886,026	4,160,000	-2,273,973
280	F_1=1,500 F_2=90	6,735,651	11,071,406	4,335,755	4,160,000	175,755

Table 4.1. Model Output from CN-IC Example Scenarios

4.7 Model Sensitivity Analysis

The example presented above shows there are scenarios where retrofitting railway bridges may be feasible; however, it also shows that the time to repair must be quite long (F_1 = 1,500 days & F_2 = 90 days) in order for this to be the case. To further examine the cost-effectiveness of retrofitting railway bridges, I performed a sensitivity analysis on two variables: 1) time to repair for an F_1 type failure and 2) detour mileage. The remaining input parameters were held constant using data for the Metropolis, Illinois bridge:

- ♦Repair cost: F_1 type failure = \$2,100/track foot
  F_2 type failure = \$700/track foot
- ♦Retrofit cost: Main span bearing = \$65,000/bearing
  Approach span bearing = \$15,000/bearing
- ♦Bridge length = 5,660 feet
- ♦Number of main spans = 7
- ♦Number of approach spans = 39
- ♦Discount rate for NPV analysis = 5%
- ♦Annual tonnage = 50 MGT
- ♦Detour costs: variable cost = \$2.5/mile (Railroad Facts, 2000)
  fixed cost = \$62/train (Railroad Facts, 2000)
  lost revenue cost = \$0.024/mile (Railroad Facts, 2000)
- ♦Time to repair F_2 type failure = 30 days

For certain combinations of detour length and time-to-repair (for an F_1 type failure), retrofitting is cost-effective (Figure 4.6). The solid horizontal line represents the retrofit cost of the CN-IC Metropolis bridge. The intersection point between the horizontal line representing retrofit cost, and the sloping lines representing increasing detour length, is the point where the risk cost exceeds the retrofit cost. Any point along the increasing detour length lines above the horizontal retrofit cost line represents a detour length and time-to-repair combination where retrofitting is cost-effective. The results show that when the detour length is short and the time-to-repair is small, it is not cost-effective to retrofit. As the detour length and repair time increase, the cost-effectiveness of retrofitting becomes plausible. Analyzing this example, for detour lengths up to 400 miles, the F_1 time to repair has to be 1,080 days or greater for retrofitting to be cost-effective.

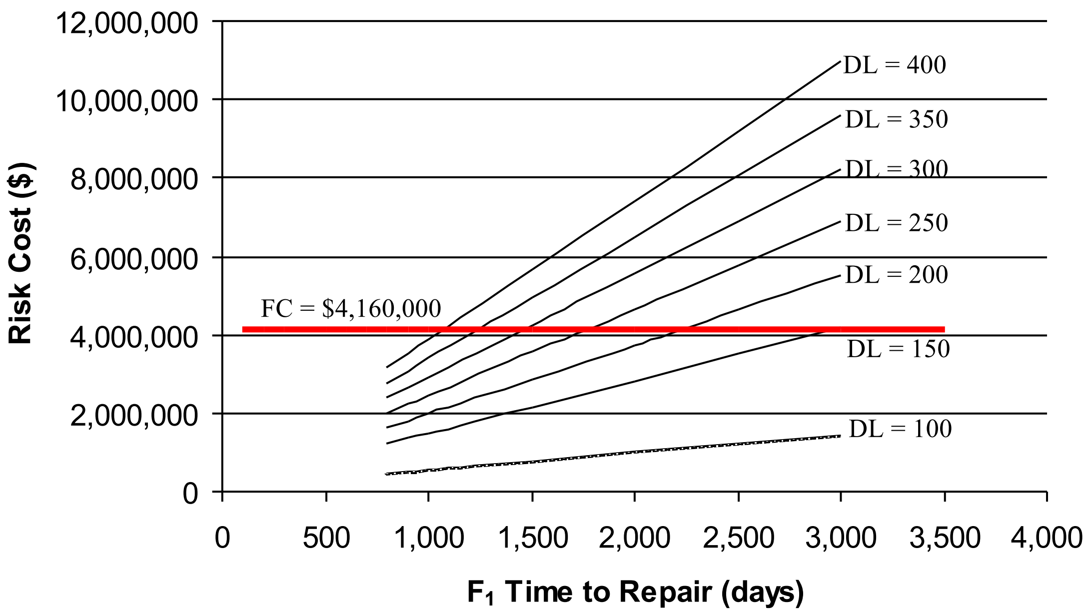

Figure 4.6. Model Sensitivity for the CN-IC Metropolis Bridge (DL = Detour Length (in miles) and FC = Retrofit Cost)

Bearings are considered to be the most important railroad bridge element requiring retrofit, and for this sensitivity analysis, bearings were the principal cost element considered. However, if other options for retrofitting were also employed (such as encasing piers, increasing footing sizes, etc.), the cost to retrofit would increase. This would increase the horizontal line in Figure 4.6, thereby increasing the threshold repair time for retrofitting to be cost-effective. Similarly, if the discount rate increased, the NPV of the risk cost would be reduced. This, too, would increase the necessary time to repair for retrofitting to be cost-effective.

CHAPTER 5
PRELIMINARY REROUTING ANALYSIS

5.1 Princeton Transportation Network Model Introduction and Data Acquisition

The detour scenarios described in Chapter Four assume that CN-IC would reroute individual trains. In reality, given a major network disruption in the central United States, the railroads would probably respond through a combination of rerouting trains, reconfiguring trains and rerouting traffic. For business reasons, railroads are expected to prefer using their own routes whenever possible. Depending on their geography and line characteristics, some railroads will have more flexibility than others. For example, long-distance eastward and westward traffic might be rerouted to more northerly or southerly gateways, thereby completely bypassing the Mid-America region. The scenarios presented in Chapter Four considered the simplest case of rerouting trains via the closest available detour. However, service quality, economic efficiency and capacity constraints are better addressed by making greater use of the complete rail network. The preliminary rerouting analysis presented in this chapter considers detouring traffic in this manner.

To evaluate various rerouting scenarios, I used the ALK Technologies, Inc. Princeton Transportation Network Model (PTNM). The PTNM provides the ability to flow traffic over the North America rail network. The model contains a variety of functions that allow the user to create a rail network, edit the existing rail network, estimate the traffic flows for various network scenarios and generate output in the form of maps. The model utilizes Dyalog Advanced Programming Language (APL) to carry out its functions.

The PTNM uses the Surface Transportation Board (STB) Carload Waybill Sample data to flow traffic over the rail network. The Waybill Sample is an annual sample (approximately 2.8%) of carload waybills for U.S. terminated shipments by rail carriers. Railroads are required to submit waybill sample data if they terminated 4,500 or more revenue carloads on their lines in the U.S. in any of the three preceding years (Loren Data Corporation, 2001). The 1999 sample contains 566,829 waybills. The PTNM uses specific data from the Waybill Sample to flow traffic over the rail network. The model requires the waybill data to be formatted so that each record contains the following information:

- Railroad carrier
- Origin point
- Destination point
- Freight tons carried
- Service Type

The PTNM contains two workspaces for network analysis: 1) TRGRAPH and 2) ALKFLOW. TRGRAPH contains the functions and variables required for editing the rail network and drawing flow maps. Some of the TRGRAPH functions utilized in this analysis include:

- PICKNET – allows the user to choose and load a rail network
- USMAP – draws a map of the continental United States
- SHOWLINKS – draws links for the specified railroad
- LABEL – labels cities on the network map
- LINKEDIT – allows the user to edit links on the rail network

The ALKFLOW workspace contains functions to flow traffic over a network. The main function utilized in the ALKFLOW workspace is called FLOWΔWINDOWS. This function allows traffic to be flowed over the rail network using the waybill data. Inputs necessary to flow traffic include the rail network, the origin-destination (O-D) file that specifies rail carrier, origin node, destination node and volume for each waybill. The PTNM allows the traffic data to be flowed using a BESTROUTE algorithm, a DIST algorithm, or a user-defined algorithm. The BESTROUTE function forms the railroad route that is the most efficient path over the network. Efficiency in the BESTROUTE function is defined by travel distance, interchange frequency, and trackage rights. The DIST function forms the railroad route that is the shortest path over the network. For this analysis, I used the BESTROUTE algorithm to simulate the most realistic and practical movement of rail shipments.

5.2 Waybill Data Separation Methods and Flow Analysis

The basic PTNM model allows the user to flow traffic in two ways:

1) Using the existing rail network with all rail carriers active (multi-carrier method)
2) Using the existing rail network with one fictitious carrier (single-carrier method)

5.2.1 Multi-Carrier Method

For the multi-carrier method analysis, the waybill data were separated into individual origin-destination movements. The PTNM reads input waybill data containing one railroad carrier and one O-D pair. Approximately 20% of the waybills have one or more interchange points. The waybills containing multiple railroad carriers were separated using Statistical Analysis Software (SAS). I imported the waybills containing multiple rail carriers into SAS and developed a program to format the data. Each multi-interchange waybill was separated into individual records, one for each origin-destination pair (Table 5.1). After separating the waybills, the total number of records input into the PTNM equaled 678,881.

The multi-carrier analysis uses the existing PTNM rail network (called RWY99WB) to evaluate how the traffic flows over the network. Each of the railroads was defined with a three digit numeric code. The input waybill data contains the railroad numeric code for each origin-destination pair. The PTNM uses this numeric code to flow traffic between an OD-pair on the actual rail carrier.

SAS Input Waybill

Waybill ID	Origin Railroad	1st Bridge Railroad	Termination Railroad	Origin Node	Interchange Node	Interchange Node	Termination Node
12345	CP	NS	UP	19564	16764	16588	10599

SAS Output Waybill

Waybill ID	Origin Railroad	1st Bridge Railroad	Termination Railroad	Origin Node	Interchange Node	Interchange Node	Termination Node
12345	CP	-	-	19564	-	-	16764
12345	NS	-	-	16764	-	-	16588
12345	UP	-	-	16588	-	-	10599

Table 5.1. Example Separation of Multi-Interchange Waybills in SAS for the Multi-Carrier Analysis

Based on the risk analysis presented in chapter four, the bridges at greatest risk in a major Mid-America earthquake are the UP bridge at Thebes, IL, the CN-IC bridge at Cairo, IL and the BNSF (CN-IC) bridge at Metropolis, IL. The 2% peak ground accelerations with a fifty-year return period at these locations all exceed 0.6 g, which is above the threshold for severe damage (> 0.4 g). The waybill data were flowed over the original PTNM rail network as a baseline for comparison (Figure 5.1 and Figure 5.2). The units in the legend represent freight tons. The flow labels on the links represent the freight tons flowed over specific links by direction. Twenty million freight tons (38.5 MGT) is roughly equivalent to 17 average-sized freight trains per day (Appendix B). For the original network, the total freight ton-miles flowed over the network equaled $1.49*10^{12}$, which corresponds with the overall freight ton-miles statistic from the Association of American Railroads (AAR) (AAR, 2000). The link volume flows are also consistent with gross tonnage figures obtained from the USDOT Bureau of Transportation Statistics year 2000 GIS dataset.

The PTNM does not implicitly account for line capacity. A rough scale for maximum rail line capacity was developed based on the amount of tracks, traffic control system (where CTC = Centralized Traffic Control) and distance between sidings. The following assumptions were used to develop the scale: 1) homogeneous traffic mix, 2) 50-mph track speed with rolling terrain, 3) no priority movements or passing, and 4) no maintenance work on the line.

Figure 5.1

Figure 5.2

Using these assumptions, the following scale for maximum line capacity (trains/day includes both directions) was developed (Salzman, 2002):

Single Track

2.5 mile sidings, spaced 11 miles apart with train orders ~ 10-15 trains/day

2.5 mile sidings, spaced 11 miles apart, operated with CTC ~ 20-25 trains/day

2.5 mile sidings, spaced 7 miles apart, operated with CTC ~ 30-35 trains/day

5 mile sidings, spaced 7 miles apart, operated with CTC ~ 40-45 trains/day

Alternating Single and Double Track

10 miles double track, 30 miles single track with two 2.5-mile sidings, operated with CTC ~ 50-55 trains per day

10 miles double track, 10 miles single track, operated with CTC ~ 60-70 trains/day

Double

Full double track, operated with CTC ~ 75-125 trains/day

5.2.2 Rerouting Scenarios for Multi-Carrier Analysis

After generating the baseline flow volumes, I edited links in the network to simulate a major bridge or combination of major bridges being out of service (Table 5.2). The link attribute, mainline code (MLC), which acts as an impedance variable, was set to its maximum level (99) to simulate a bridge being out of service. As expected, the major rerouting occurred in the Mid-America region, so the following maps are centered in the central U.S. to distinguish the differences in traffic flow on the edited rail network and original rail network.

Scenario	Bridge(s) Out of Service	2% PGA (g's)
1	UP Mississippi River Bridge at Thebes, IL	0.8
2	CN-IC Ohio River Bridge at Cairo, IL	0.9
3	BNSF (CN-IC) Ohio River Bridge at Metropolis, IL	0.6
4	Bridge at Thebes, IL and Bridge at Cairo, IL Concurrently	0.9

Table 5.2. Rerouting Scenarios for Traffic Flow Analysis

In the first scenario, I removed the link representing the UP bridge over the Mississippi River at Thebes, IL. For this scenario, the PTNM model predicts UP would transfer a majority of its traffic from the dual north-south mainlines through Arkansas onto the Desoto subdivision (Figure 5.3). However, according to UP personnel, the Desoto subdivision has limited capacity to accommodate additional moves (Salzman, 2002). Other rail lines in the region saw incremental changes, but these increases would probably not exceed the other lines' capacity.

For the second scenario, I removed the link representing the CN-IC bridge over the Ohio River at Cairo, IL. For this scenario, the model predicts a majority of the CN-IC traffic would be rerouted from the mainline through Cairo (west main line) to the main line through Metropolis (east main line) (Figure 5.4). Discussions with CN-IC personnel verified that in situations where one of the CN-IC mainlines is out of service, the other mainline could generally handle the extra freight traffic.

For the third scenario, I removed the link representing the BNSF (CN-IC) bridge over the Ohio River at Metropolis, IL. In this case, the model rerouted the CN-IC traffic over the Cairo mainline (Figure 5.5). The amount of CN-IC traffic to be rerouted is approximately 20 trains per day. The BNSF traffic using the bridge was routed through Memphis and then north on the CN-IC. The rerouted BNSF traffic totaled approximately two trains per day. As in scenario two above, the capacity of the affected lines is expected to be adequate to handle the rerouted traffic.

In the fourth scenario, I removed a combination of links. The most likely combination of bridges being out of service concurrently is the Thebes bridge and the Cairo bridge. I removed the links for these bridges and flowed the traffic in the PTNM. The flow map indicates the UP would reroute a majority of their traffic on the DeSoto subdivision (Figure 5.6). However, as stated above, this line is already close to capacity, so in reality, UP would need to find another way to detour this traffic. The CN-IC again uses their mainline through Metropolis to handle the rerouted traffic. Various other rail lines on the rail network see marginal changes, but none greater than 8 trains per day.

Figure 5.3.

Figure 5.4.

Figure 5.5

Figure 5.6

The multi-carrier analysis has some limitations. First, as illustrated above, the model does not implicitly account for capacity. Routes with limited capacity may be used for detouring large amounts of traffic. This produces unrealistic rerouting of traffic for some scenarios. Second, by separating all the waybills into single O-D pairs, the model's predictions of traffic rerouting are constrained. For example, a waybill recording a shipment from Dallas, TX to Louisville, KY uses two railroads, UP and CSXT. The interchange point for this shipment is Memphis, TN. By splitting up the multi-interchange waybill data into individual records, each containing one O-D pair, the model reads the input data as two shipments, one from Dallas, TX to Memphis, TN on UP, and the second from Memphis, TN to Louisville, KY on CSXT.

In the multi-carrier analysis, if the link connecting West Memphis and Memphis (representing the UP bridge over the Mississippi River) is removed, the PTNM flows the two waybills individually. For the shipment from Dallas to Memphis, the model requires the shipment to travel through Memphis using UP trackage (if possible). For this scenario, the shipment from Dallas to Memphis was flowed over the UP from Dallas to Tamaroa, IL. At Tamaroa, the shipment was interchanged onto the CN-IC south to Memphis. In reality, this shipment most likely would have used another gateway, such as St. Louis or Chicago, as an interchange point between UP and CSXT to get to Louisville. To counteract this limitation, I performed a second analysis assuming only one carrier.

5.2.3 Single-Carrier Method

For the single-carrier analysis, a single artificial carrier name was created to represent all the railroads on the U.S. rail network. Creating an artificial carrier allows the entire network to be used for routing traffic, rather than constraining traffic flows to particular railroads and interchange points. As mentioned earlier, the PTNM reads input waybill data containing one railroad carrier and one O-D pair. For the single-carrier analysis, the waybills with multiple interchange nodes were separated into one record containing the origin point and the termination point. The interchange nodes were eliminated so the model would flow traffic utilizing the entire network rather than forcing traffic through a specific interchange point. I imported the waybills containing multiple rail carriers into SAS, and developed a program to format the data. Each multi-interchange waybill was separated into a single record containing the origin and termination nodes (Table 5.3). KD represents the artificial carrier that accounts for all the carriers on the network.

SAS Input Waybill

Waybill ID	Origin Railroad	1st Bridge Railroad	Termination Railroad	Origin Node	Interchange Node	Interchange Node	Termination Node
12345	CP	NS	UP	19564	16764	16588	10599

SAS Output Waybill

Waybill ID	Origin Railroad	1st Bridge Railroad	Termination Railroad	Origin Node	Interchange Node	Interchange Node	Termination Node
12345	KD	-	-	19564	-	-	10599

Table 5.3. Example Separation of Multi-Interchange Waybills in SAS for the Single-Carrier Analysis

After formatting the waybill data in SAS, the PTNM was used to flow the traffic over the network. As in the multi-carrier analysis, the data were flowed over the original network to act as a baseline for comparison with edited network scenarios. The bridge scenarios presented in the multi-carrier analysis were repeated using the single-carrier network.

The flow diagrams generated for the single-carrier analysis are change-in-flow maps, which are different from the overall flow maps presented in the multi-carrier analysis. In the change-in-flow diagrams, the rail lines that experience a decrease in traffic flow compared with original network (no link edits) are highlighted in red, and the rail lines that experience an increase in traffic flow compared with the original network are highlighted in green.

5.2.4 Rerouting Scenarios for Single-Carrier Analysis

For the first scenario, the link representing the bridge at Thebes was removed (Figure 5.7). For the single-carrier analysis, the PTNM predicted an increase in traffic flow on the southern end of the east UP mainline through Pine Bluff, AR. Most of this traffic flows through Memphis, utilizing the CN-IC north from Memphis. The increase in traffic off the UP onto the CN-IC is approximately18 trains per day. This increase in traffic may exceed the capacity of the line from Memphis to Fulton, KY (the point where the single CN-IC mainline splits into two mainlines). Under this scenario, the BNSF line heading south from St. Louis along the west side of the Mississippi River (single track, CTC, 11-mile siding spacing) is also expected to receive a large increase in traffic. This line has an estimated capacity of approximately 20-25 trains per day. The predicted increase in traffic is approximately 20 trains per day, which exceeds this line's capacity. The NS line from Tolono, IL to St. Louis is predicted to increase by approximately 9 trains per day. The rail line predicted to see the largest decrease in traffic is the west UP mainline through Texarkana, Arkansas. The model predicts a decrease of approximately 14 trains per day. Other rail lines in the area would see marginal changes in flow volume (less than 8 trains per day), but most should be able to handle the increases in volume.

For the second scenario, the link representing the bridge at Cairo was removed (Figure 5.8). For the single-carrier analysis, the PTNM predicts the CN-IC traffic over the west mainline through Cairo would be rerouted on UP from Desoto, IL to Rockview, MO, and then south on the BNSF to Memphis. The model predicts an increase of approximately 2 trains per day. The CN-IC line through Cairo handles mostly passenger (two daily) and intermodal trains. Due to the low tonnage per car characteristic of these types of trains, the number of rerouted trains per day would actually higher. The model predicts the east CN-IC mainline would see an increase of less than 1 train per day. In reality, the CN-IC would most likely route the displaced traffic completely onto their east mainline through Metropolis and this line would be able to handle the increase.

Figure 5.7

Figure 5.8

For the third scenario, the link representing the bridge at Metropolis was removed (Figure 5.9). As was the case in the multi-carrier analysis, the PTNM predicts most of the traffic would be rerouted on the east CN-IC mainline through Cairo. The increase in the number of average-sized freight trains per day is approximately 20. This increase in traffic will probably not exceed capacity on the CN-IC mainline through Cairo. The model predicts the remainder of the traffic would be rerouted via the UP and BNSF to Memphis. The increase in traffic on the UP and BNSF is approximately 11 trains per day. From the maximum capacity scale presented earlier, this would increase traffic on the BNSF line from Rockview, MO to Memphis, TN above its capacity. To counteract this, the trains could remain on the UP from Thebes to Memphis, assuming sufficient excess capacity is available on their mainline south through Arkansas.

For the fourth scenario, the links representing the bridges at Thebes and Cairo were removed (Figure 5.10). The PTNM predicts that traffic on the east UP main line through Pine Bluff, AR would increase by approximately 10 trains per day. Most of the traffic flows through Memphis, and connects with the CN-IC. The increase in traffic on the CN-IC mainline through Metropolis is approximately 15 trains per day. Another mainline predicted to see a large increase in traffic is the BNSF line south from St. Louis to Rockview, MO. The model predicts an increase of approximately 21 trains per day. This would increase traffic levels above capacity for this line. The other mainline to see an increase in traffic for this scenario is the NS line from Tolono, IL to St. Louis. The model predicts an increase of approximately 8 trains per day. Other rail lines in the region see marginal changes, but none are heavily impacted.

The single-carrier analysis also has some limitations. The traffic flowed between an origin-destination pair does not have to flow over one railroad. For the single-carrier simulation, one railroad controls the movements over the entire network. This allows the artificial railroad to move traffic using any junction on the network. In reality, some of these junctions may be low use, slow speed connections where detouring trains would be time consuming and impractical. Another limitation of the single-carrier analysis is that the traffic does not necessarily utilize the original rail line as presented in the waybill.

Figure 5.9

Figure 5.10

For example, a waybill with an origin-destination pair of Los Angeles and Chicago has two options for shipment because that corridor is served by two railroads, UP and BNSF. The original waybill for that shipment may have had UP as the rail carrier, but with a single, artificial rail carrier, the shipment may have utilized the BNSF route because it has a shorter distance between Los Angeles and Chicago.

5.3 Summary of Preliminary Rerouting Analysis

Two types of traffic flow rerouting analysis were performed using the Princeton Transportation Network Model: 1) the multi-carrier analysis and 2) the single-carrier analysis. Both of these analyses provide solutions for how the railroads would reroute traffic given a network disruption, but both methods have limitations. The multi-carrier analysis constrains multi-interchange waybills to flow through specific interchange nodes because of how the model requires the data to be formatted. The single carrier analysis resolves the interchange problem, but allows unrealistic flow of traffic between rail carriers. ALK produces another network, called the quanta-network, which allows the user to edit links and define junction penalties. The quanta-network would produce a more realistic routing analysis for the scenarios presented above, but was unavailable at the time of this study.

The major limitation of this analysis is the lack of an implicit variable to account for capacity on the rail lines in the network. The model provides the ability to manually edit rail line capacity, but this was beyond the scope of this study. For the analysis to be more realistic and practical, the factors affecting rail line capacity for each rail line used in rerouting should be more thoroughly evaluated. Performing certain operating procedures, such as fleeting, can increase the capacity of rail lines. Fleeting is the movement of a group of trains in one direction without any meets with train movements in the opposite direction. By fleeting rail movements, the time lost for meets is avoided thus creating a more efficient operation.

Overall, the rerouting analysis provided some useful insights. The PTNM provided possible solutions for rerouting traffic given specific network disruptions, but it has some limitations. A more complete traffic flow analysis should be completed with the quanta-network model. This would provide a better, more realistic evaluation of rail traffic flow in the region.

CHAPTER 6
CONCLUSIONS AND FUTURE RESEARCH

6.1 Conclusions

This study examined the potential impact of a Mid-America earthquake on the rail network. The following is a summary of the results drawn from the study:

- There are a total of approximately 298 railroad route miles in areas with a 5% probability (in the next fifty years) of experiencing peak ground acceleration (PGA) values sufficient to cause moderate damage (> 20% g). Of these, 36% are in areas of high soil liquefaction potential.

- There are approximately 2,107 railroad route miles in areas with a 2% probability of experiencing PGA values sufficient to cause moderate to severe damage (> 20% g). Of these, 447 route miles are in areas with the potential of experiencing severe damage (> 40% g). 40% of the 2,107 miles are in areas of high soil liquefaction potential.

- There are eight major river-crossing bridges in the region in areas potentially exposed to moderate to severe PGA values. These eight bridges carry approximately 245 million tons of freight per year, which accounts for 11.4% of the total rail freight originated in the U.S. annually.

- There are approximately 2,082 railroad bridges in the region. Of these, 59% are in areas with high soil liquefaction potential.

- Multiplying the average bridge length (147 feet) for one major railroad in the region by the estimated total number of bridges for all the railroads yielded a total length for all bridges in areas potentially exposed to damaging peak ground accelerations with a 2% probability of occurring in the next fifty years (> 20% g) of 306,784 feet (58.10 miles).

- A model to determine the cost-effectiveness of retrofitting railroad bridges was developed. The input parameters for the model include: repair cost per failure type, bridge length, number of approach spans, number of main spans, annual tonnage, detour length, time to repair per failure type, and discount rate for net present value analysis. A sensitivity analysis was performed for the Ohio River bridge at Metropolis, IL. For detour lengths up to 400 miles, the F_1 (major bridge damage) repair time must exceed 1,080 days for retrofitting to be cost effective.

- A preliminary rerouting analysis was performed using the Princeton Transportation Network Model to evaluate freight traffic flows for various network disruptions (representing one or more major bridges being out of service). Due to a lack of a capacity variable in the model, predicted flows over some lines exceeded their maximum capacity. The results suggest a majority of the rerouted traffic would be detoured in or near the Mid-America region.

6.2 Future Research

The possibility of a major Mid-America earthquake is not remote. This study attempted to quantify the impact of a major earthquake on the rail network in the Mid-America region. The following research tasks concerning the rail infrastructure in the region are advisable:

- Develop railroad bridge fragility curves to evaluate the probability of failure for typical timber, concrete and steel railroad bridges given various earthquake ground motions.

- Study the effects of liquefaction on railroad bridge support systems (foundations, abutments and piers).

- Develop more extensive railroad geographical information systems databases containing specific bridge attributes such as the number of spans, span type (deck plate girder, through plate girder, etc.) and bridge type (open deck, closed deck).

- Further develop the cost-effectiveness model to account for the probability of individual bridge component failure (such as spans, bearings, etc.).

- Perform a more extensive traffic flow rerouting analysis using the ALK quanta-network that allows the user to create a more realistic rail network study. Further discussions with railroad transportation planners would also be appropriate.

6.3 Critical Information to Record following Seismic Events

The completion of this study has provided a better understanding of the railroad infrastructure potentially at risk in a major earthquake. Throughout the research, assumptions were made due to a lack of certain information concerning railway bridges involved in seismic events. Below is a list of data and information that would be beneficial to record when railroad bridges are involved in an earthquake:

- Record detailed information on the bridge components that fail (such as the number and type of bearings, spans, trusses and piers that fail) to aid in the development of railroad bridge fragility curves.

- Based on damage to the bridge, record information on the bridge components that may have benefited from retrofitting. Estimate the forces exerted on individual bridge components and the amount of resistance or added strength that might be gained from various retrofit options.

- Determine and record the local ground motion characteristics near and around affected railway bridges. Detailed ground motion information for each abutment, pier and span would help in better understanding the probability of experiencing differing local ground motions.

- Record the effects of liquefaction near and around railway bridges experiencing seismic activity. Also record the damage to bridge components exposed to soil liquefaction.

- Record the number of railway bridges that experience damage compared to the total number of bridges in the affected area. Also, establish a general PGA damage scale for railroad bridges.

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