ANALYSIS OF A TYPICAL MIDWESTERN STRUCTURE SUBJECTED TO SEISMIC LOADS

A Senior Honors Thesis

By

JASON FRAZIER HART

Submitted to the Office of Honors Programs
& Academic Scholarships
Texas A&M University
In partial fulfillment of the requirements of the

UNIVERSITY UNDERGRADUATE
RESEARCH FELLOWS

April 2000

Group: Physical Sciences

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RESEARCH FELLOW

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April 2000

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ABSTRACT

Analysis of a Typical Midwestern Structure Subjected to Seismic Loads. (April 2000)

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The extent of damage and casualties in Midwest cities such as St. Louis during an earthquake caused by the New Madrid fault system will be due in part to the performance of buildings. Dynamic nonlinear analysis of a reinforced concrete building not designed for seismic loads is one method used to assess an existing building's ability to withstand an earthquake. Many researchers have studied the earthquake resistance of structures, and often analytical studies have used recorded ground motions such as the 1940 El Centro, California, earthquake. Reports from past experimental studies and observations of damage caused by seismic events have been valuable for evaluating the performance of specific components of a building and overall performance of buildings subjected to this type of ground motion. This research study differs from previous research in that it focuses on the Midwest United States and uses synthetic ground motions developed specifically for an earthquake that would occur in this region. Research of the performance of a five-story, reinforced concrete, moment frame building in the Midwest United States is discussed in this thesis. In order to estimate the performance of a typical building in this region, the building was designed based on

codes from the mid-1980's, prior to the seismic design standards of today requiring a ductile structural system.

The study building's performance is evaluated using the dynamic nonlinear analysis computer program DRAIN-2DM. Dynamic analysis of the structure is performed using synthetic ground motions for the Midwest produced by Y.K. Wen of the Mid-America Earthquake Center. The analyses of the building were performed using twenty ground motion records. Ten ground motions are for earthquakes with 2 percent probabilities of exceedance in 50 years, and ten are for 10 percent probabilities of exceedance in 50 years. Results of the analyses are discussed in this thesis and are used to estimate the damage to the structure.

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CHAPTER I

INTRODUCTION

The evaluation of a building by dynamic nonlinear response analysis is one method used to determine the structural performance during an earthquake. This study uses this analytical approach for the specific case of a building in St. Louis in combination with a ground motion typical of those in the Midwest region. Synthetic ground motions that have recently been produced are expected to yield a better estimate of ground shaking for evaluating the seismic performance of a St. Louis building than those recorded during California earthquakes.

At the start of this study, an attempt was made to obtain design drawings of a St. Louis building built in the early 1980's, prior to St. Louis being assigned to seismic zone two of the *Building Officials and Code Administrators (BOCA) Basic/National Code* (*Building officials* 1987). Design drawings for a typical building from this time period could not be obtained. Designing a building according to the codes used by designers during the early 1980's was judged to be an equal alternative case study structure. Engineers with design experience in the St. Louis region provided a starting point for the design of the study building by responding to questionnaires. The subjects of the questions ranged from the typical number of stories in an office building to types of floor systems commonly used in the region. Once the overall size and structural system of the reinforced concrete office building were determined, the detailed design of the building

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This thesis follows the style and format of the *Journal of Structural Engineering.*

was performed using the ninth edition of the BOCA code released in 1984 and the 1983 American Concrete Institute (ACI) *Building Code Requirements for Reinforced Concrete* (*Building code* 1983). This design using these codes is representative of similar buildings designed while the ninth edition of the BOCA code was used.

Analysis of the study building was performed using DRAIN-2DM, a dynamic nonlinear analysis computer program that has been used widely for this type of research. The program was developed at the University of California-Berkeley, and this version, which features reinforced concrete beam and slab models, was enhanced at the University of Michigan. The slab model of DRAIN-2DM is particularly beneficial to this research because it is capable of predicting punching shear, a failure mode that is frequently found in buildings with flat-slab floor systems damaged by earthquakes (Hueste and Wight, 1997).

CHAPTER II

REVIEW OF PREVIOUS RESEARCH

A wide variety of research that is applicable to this study is available in the literature. Two components of the study that received focus when examining reports documenting previous research were dynamic nonlinear analysis using DRAIN-2DM and the behavior of reinforced concrete structural members subjected to seismic loads. This literature review provided insight into the behavior of the study building and data that could be compared to this study's results.

DYNAMIC NONLINEAR ANALYSIS USING DRAIN-2DM

The dynamic nonlinear analysis of a reinforced concrete building using DRAIN-2DM has been documented in numerous structural engineering journals. Studying the publications of previous researchers yielded an efficient procedure for this study of a building specific to St. Louis, Missouri. Many articles were consulted throughout this research, and several were important in developing a procedure for the analysis of the study building.

A four-story reinforced concrete building was studied by Hueste and Wight (1997 and 1999). This building, located in Northridge, California, was damaged during the 1994 Northridge Earthquake. The most prevalent form of damage to this building was punching shear failures at the slab-column connections. Their research focused on post-calculating the damage with several analysis techniques, including dynamic nonlinear analysis. One aspect of Hueste and Wight's research that was consulted is the method used to model slab members with the two-dimensional analysis computer program DRAIN-2DM. The model of a slab member used in the analysis of the St. Louis study building was developed by Hueste and Wight and is capable of predicting a punching shear failure. The model also accounts for the loss of stiffness in a slab member following a punching shear failure.

Shooshtari and Saatcioglu (1998) modified DRAIN-2D to include models for inelastic flexure, axial force-moment (P-M) interaction, anchorage slip, and shear. This version is called DRAIN-RC. Though DRAIN-RC is not used in this study, Shooshtari and Saatcioglu's conclusions are beneficial. They found that for a ductile, moment frame building, the results of an analysis that include P-M interaction and shear models did not significantly differ from an analysis that did not include the models. Analysis with the anchorage slip model predicted building drift values that were twice as large as those predicted by analysis without the model. The anchorage slip model developed by Shooshtari and Saatcioglu is not included in this study, but reinforcement that is anchored into a support was checked to ensure that it has sufficient length to be developed. If the reinforcement cannot be developed, its capacity to carry tensile force is reduced in the analysis.

REINFORCED CONCRETE BEHAVIOR UNDER SEISMIC LOADING

Reinforced concrete slab behavior during earthquake loads has frequently been studied by leading structural engineering researchers. This research has provided empirical and mathematical models of the behavior of slab members under lateral loading. A review of this research provides a background for this study.

The design of the study building includes shear capitals at interior slab-column connections. Research of the behavior of slab-column connections with shear capitals was conducted by Wey and Durrani (1992). This research provided experimental data for the connections that can be compared to results obtained from the study building. Moment-rotation plots provided by Wey and Durrani are especially valuable because they document the inelastic behavior that test specimens experienced. In addition, the data from Wey and Durrani's research contributed to the development of the slab element used in DRAIN-2DM (Hueste and Wight 1997). The conclusions of this research provide methods of design that result in proper performance at slab-column connections during earthquakes. One notable conclusion of this study is that the length of a shear capital does not significantly increase in the initial stiffness of a slab-column connection, but an increase in depth does provide a significant increase in initial stiffness.

The general earthquake resistance of reinforced concrete slab-column connections was studied by Durrani and Wight (1987). This study provides a correlation between inter-story drift and connection response. The study documents that the reinforcement in the slab is expected to yield at an inter-story drift of 1.5 percent. Inter-story drift less than two percent is not expected to cause damage due to a shear failure. Durrani and Wight found that in the case of a slab with unequal amounts of top and bottom reinforcing steel and subjected to cyclic loading, bond deterioration and bar slip problems will be more significant for the bottom steel.

Pan and Moehle (1992) studied the effect of gravity load on building drift during an earthquake. Four interior slab-column connections were modeled at 60 percent of full scale for their study. Biaxial and uniaxial lateral loads were applied to the specimens. The study concluded that biaxial loading reduces the allowable drift capacity of slab-column connections. The research described in this thesis does not include biaxial lateral loading, and the response to an earthquake that causes the building members to be loaded in a biaxial manner may be more severe than that reported in this thesis. Pan and Moehle also concluded that the magnitude of the gravity load shear carried by the slab is a primary variable affecting the behavior of reinforced concrete slab members. Like Durrani and Wight (1987), the study recommends that inter-story drift values not exceed 1.5 percent. The data and conclusions of this research were important contributions to the development of the punching shear model for the slab element used in DRAIN-2DM (Hueste and Wight 1997).

CHAPTER III

NEW MADRID SEISMIC ZONE

The largest series of earthquakes known to have occurred in North America is the series known as the New Madrid Earthquakes. The epicenters of these earthquakes were near the town of New Madrid, Missouri. The New Madrid Earthquakes consist of three major earthquakes and 203 damaging aftershocks that took place in the winter of 1811-12. The body-wave magnitudes of the three large earthquakes were 7.35, 7.2, and 7.5. Body-wave magnitudes for the aftershocks are estimated to have been between 5.0 and 6.7 (Nuttli 1982). Due to the low population in this region in the early nineteenth century, there were few casualties. A similar earthquake today would be catastrophic. In 1975, there were 12.6 million people living in the damage threshold of the New Madrid Earthquakes. The damage threshold is any area where the Modified Mercalli intensity is VII or greater (McKeown 1982). While earthquake research for the Western United States has been conducted for decades, regional seismic monitoring in the Midwest did not begin until 1974 (McKeown and Pakiser 1982). In order to appreciate the importance of earthquakes in the Midwest, it is helpful to examine the history of Midwest earthquakes and the differences between Midwest and Western United States earthquakes.

HISTORY OF MIDWESTERN UNITED STATES EARTHQUAKES

Prior to the New Madrid Earthquakes, five earthquakes are known to have occurred in the region. These five earthquakes took place between 1776 and 1804, but few facts concerning these earthquakes are known. On December 16, 1811, the first of the series of New Madrid Earthquakes was felt over an area of 5,000,000 km^2 and caused damage to structures within a 600-km radius. The Modified Mercalli intensity of this earthquake is estimated as either X or XI. Table 1 is a Modified Mercalli intensity index, which shows the type of response that corresponds to a particular intensity.

TABLE 1. Modified Mercalli Intensity Index [United (1999)]

Index Value	Description
I	Not felt except by a very few under especially favorable conditions.
II	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
III	Felt quite noticeably by persons indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibrations similar to the passing of a truck. Duration estimated.
IV	Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
V	Felt by nearly everyone; many awakened. Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop.
VI	Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.
VII	Damage negligible in buildings of good design and construction; slight to moderate in well-built structures; considerable damage in poorly built or badly designed structures; some chimneys broken.
VIII	Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks,columns, monuments, walls. Heavy furniture overturned.
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent.
XI	Few, if any (masonry) structures remain standing. Bridges destroyed. Rails bent greatly.
XII	Damage total. Lines of sight and level are distorted. Objects thrown into the air.

The second of the three New Madrid Earthquakes took place on January 23, 1812. Like the first earthquake, this earthquake was of a Modified Mercalli intensity of X or XI, and was felt over a 5,000,000 km^2 area. The third earthquake, with a Modified Mercalli intensity of XII, occurred on February 7, 1812, and was the largest of the series. It was felt over an area greater than 5,000,000 km^2 (Nuttli 1982). Though these earthquakes are large, only recently have buildings been designed for these earthquakes because the recurrence interval is estimated to be at least 600 years (McKeown 1982).

If the New Madrid Earthquakes were the only earthquakes to have occurred in the Midwest, it could be reasoned that the ratio of earthquake recurrence ratio to building life is so high that buildings should not include the stringent seismic design criteria that Western United States buildings require. While many design professionals are aware of the New Madrid Earthquakes, they may not be aware that twenty damaging earthquakes with Modified Mercalli intensities VI or greater have occurred in the New Madrid Seismic Zone (NMSZ) since 1812. These twenty earthquakes alone would make the region the most seismically active area in the Central and Eastern United States (Nuttli 1982). More details about these twenty earthquakes are shown in Table 2.

The New Madrid Seismic Zone began to receive national attention in the mid-1970's. Until 1973, most investigations of the NMSZ were made by students and faculty at St. Louis University with limited financial resources (McKeown and Pakiser 1982). In 1974, the Central Mississippi Valley Seismic Network (CMVSN) began monitoring seismic activity in the region. CMVSN was sponsored by the United States Geological Survey (USGS) and operated by St. Louis University. Memphis State

University later joined St. Louis University in operating the network (Shedlock and Johnston 1994). The Nuclear Regulatory Commission (NRC) sponsored an investigation into the potential seismic hazards to nuclear power plants in 1976 (McKeown and Pakiser 1982).

TABLE 2. Damaging Earthquakes in the NMSZ Since 1812 [Adapted from *Nuttli* (1982)]

Date	Epicenter Latitude	Epicenter Longitude	Current Nearest City	Modified Mercalli Intensity	Body-Wave Magnitude	Felt Area (km^2)	Distance from St. Louis (km)
Jun. 9, 1838	38.5°N	89°W	Mount Vernon, IL	VII-VIII	5.7	500,000	90
Jan. 4, 1843	35.5°N	90.5°W	Harrisburg, AR	VIII	6.0	1,500,000	340
Oct. 8, 1857	38.7°N	89.2°W	Mount Vernon, IL	VII	5.4	200,000	90
Aug. 17, 1865	35.5°N	90.5°W	Harrisburg, AR	VII	5.3	250,000	340
Apr. 12, 1883	37.0°N	89.2°W	Sikeston, MO	VI-VII	4.0	Not Available	200
Oct. 31, 1895	37.0°N	89.4°W	Sikeston, MO	IX	6.2	2,500,000	200
Apr. 29, 1899	38.8°N	87.0°W	Vincennes, IN	VI-VII	5.0	100,000	270
Nov. 4, 1903	36.9°N	89.3°W	Sikeston, MO	VII	5.3	340,000	200
Aug. 21, 1905	36.8°N	89.6°W	Sikeston, MO	VI-VII	5.0	325,000	200
May 26, 1909	42.5°N	89.0°W	Beloit, WI	VII	5.3	800,000	440
Jul. 18, 1909	40.2°N	90.0°W	Springfield, IL	VII	5.3	100,000	180
Sep. 27, 1909	39.5°N	87.4°W	Terre Haute, IN	VII	5.3	250,000	260
Nov. 26, 1922	37.5°N	88.5°W	Paducah, KY	VI-VII	5.0	130,000	200
Oct. 28, 1923	35.5°N	90.4°W	Harrisburg, AR	VII	5.3	120,000	340
Apr. 26, 1925	38.3°N	87.6°W	Evansville, IN	VI-VII	5.0	250,000	230
May 7, 1927	35.7°N	90.6°W	Jonesboro, AR	VII	5.3	300,000	320
Dec. 16, 1931	34.1°N	89.8°W	Oxford, MS	VI-VII	5.0	220,000	510
Aug. 14, 1965	37.1°N	89.2°W	Sikeston, MO	VII	3.8	700	200
Nov. 9, 1968	38.0°N	88.5°W	Mount Vernon, IL	VII	5.5	1,600,000	160
Mar. 24, 1976	35.6°N	90.5°W	Harrisburg, AR	VI	5.0	280,000	340

Further attention was brought to the New Madrid Seismic Zone following the Loma Prieta, California, earthquake in 1989. This earthquake led the United States Congress to order the USGS to prepare a plan for intensified study of the NMSZ. The National Earthquake Hazards Reduction Program (NEHRP) designated the NMSZ as a priority research area in 1990. The NMSZ research program consists of four components: tectonic framework studies, seismicity and deformation monitoring and modeling, improved seismic hazard and risk assessment, and cooperative hazard mitigation studies (Shedlock and Johnston 1994). This research has led to knowledge specific to NMSZ earthquakes, rather than the attempt to characterize NMSZ earthquakes using earthquake data from other regions.

CHARACTERISTICS OF NMSZ EARTHQUAKES

The New Madrid Seismic Zone is a clustered pattern of potential hypocenters between 5 km and 15 km deep. The NMSZ has been active since the Cretaceous period. It is possible that it was active as long ago as the Holocene. During NMSZ earthquakes, liquefaction of the soil can occur. In the NMSZ, liquefaction will occur locally at a Modified Mercalli intensity of VIII, and widespread liquefaction can be expected at an intensity greater than IX (Wheeler, Rhea, and Tarr 1994). Liquefaction was one of the damaging problems created by the New Madrid Earthquakes of 1811-12.

Several interesting differences exist between NMSZ earthquakes and those that occur in the Western United States. The most important difference is that the crust in the Midwest region attenuates energy 25% as effectively as crust in the Western United States. This means that seismic wave amplitudes will travel much farther in the Midwest. The crust in NMSZ also reflects seismic waves in some locations. It has been determined that these reflections lead to a focusing effect near Memphis, Tennessee, and St. Louis. Seismic amplitudes can be up to 1000% greater near these cities due to the focusing effect. This corresponds to an increase of at least three Modified Mercalli intensity units (Shedlock and Johnston 1994).

Another significant difference between NMSZ earthquakes and Western earthquakes is the recurrence interval. The recurrence interval for large NMSZ earthquakes, such as the New Madrid Earthquakes in 1811-12, is 600 years, while the corresponding recurrence interval for the Western United States is 100 years. This results in the probability of exceeding a particular ground motion in NMSZ being smaller than that of the West by a factor of two to three. Tectonic movement occurs in the Western United States at a rate of two to three cm per year, while movement in the NMSZ is 0.008 mm per year. Like the West, the NMSZ has a large number of faults. Most faults in the NMSZ are less than 15 km long (McKeown 1982).

It is clear that a future earthquake--- whether large or small--- is likely to occur in the NMSZ. In addition, it is realized that this earthquake will have the unique characteristics of earthquakes of the NMSZ. Any estimation of a current or future building's performance must take these two issues into account.

CHAPTER IV

SYNTHETIC GROUND MOTIONS FOR THE MIDWEST

Ground accelerations, or ground motions, are the sources of the damage generated by earthquakes. A ground motion, in conjunction with the mass of the building, can lead to a dramatic increase in the forces within components of a building. The ground acceleration usually required to damage weak construction is ten percent of gravitational acceleration (g). Between 0.1g and 0.2g, most people will have trouble keeping their footing, and sickness symptoms may result. A Modified Mercalli intensity of VII and IX correspond to ground accelerations of 0.1g to 0.2g and 0.5g, respectively (Arnold 1998). The peak acceleration of a ground motion record often will not cause the most damage in a structure. These high accelerations often correspond to high frequencies that are out of the range of the natural frequencies of most buildings. Many times a ground motion with a moderate peak acceleration and a long duration will cause severe damage (Singh 2000). Because a building's response may not depend solely on the magnitude of ground acceleration, a set of moderate and severe earthquake ground motion records are used in this research.

Accurately predicting the response of a Midwest United States building to an earthquake requires ground motions from this region. No recorded data from large NMSZ earthquakes are available, but synthetic ground motions are available. Y.K. Wen has used statistics and geoscience techniques to formulate synthetic ground motions for Memphis, Tennessee; Carbondale, Illinois; and St. Louis, Missouri, as part of a project for the Mid-America Earthquake Center (Wen and Wu 2000).

Ground motions for the city of St. Louis, Missouri, were used in this research. Ground motions are available for a representative soil for St. Louis and for bedrock. Because soil can affect the ground motion of an earthquake by amplifying the accelerations, it is an important parameter. The representative soil ground motions were chosen, and the composition of this soil is shown in Figure 1.

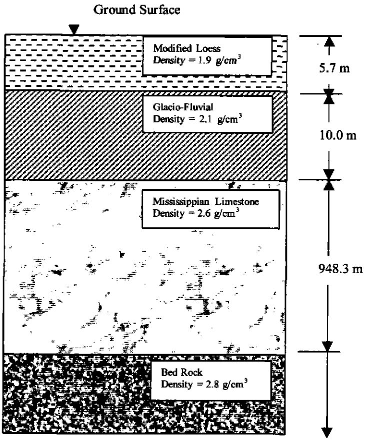


FIG. 1. Representative Soil for St. Louis [Adapted from Wen and Wu (2000)]

Ten ground motion records are available for each of two probability of exceedance levels: two and ten percent in 50 years. The report accompanying the ground motion records instructs the user to find the response of the structure being studied for each of the ten ground motion records of a probability of exceedance interval. The actual two percent or ten percent in 50 years response of the structure is the median response within the set of ten. This means that the within a set of ten ground motion records, the records with the fifth- and sixth-largest responses give the best estimates of the actual building response for the corresponding probability of exceedance level (Wen and Wu 2000). Details of each ground motion record used in the study are shown in Table 3 and Table 4. Plots of the ground motion records are shown in Appendix A.

TABLE 3. Two Percent Probability of Exceedance in 50 Years Ground Motion Records Set [Wen and Wu (2000)]

Ground Motion Record	Peak Ground Acceleration (% of g)	Duration (seconds)	Body-Wave Magnitude	Focal Depth (km)	Epicentral Distance from St. Louis (km)
l02_01s	23	70	8.0	17.4	267.0
l02_02s	25	70	8.0	9.10	229.5
l02_03s	83	10	5.4	2.10	28.70
l02_04s	25	45	7.1	5.50	253.1
l02_05s	19	55	8.0	17.4	254.3
l02_06s	24	40	6.8	5.80	224.8
l02_07s	24	70	8.0	33.9	196.3
l02_08s	24	35	8.0	9.10	260.7
l02_09s	25	35	8.0	9.10	280.5
l02_10s	54	20	5.9	4.40	47.70

TABLE 4. Ten Percent Probablity of Exceedance in 50 Years Ground Motion Records Set [Wen and Wu (2000)]

Ground Motion Record	Peak Ground Acceleration (% of g)	Duration (seconds)	Body-Wave Magnitude	Focal Depth (km)	Epicentral Distance from St. Louis (km)
110_01s	13	25	6.0	2.70	76.40
110_02s	10	40	6.9	9.30	201.5
110_03s	9.0	40	7.2	4.40	237.5
110_04s	11	25	6.3	9.80	252.2
110_05s	13	20	5.5	2.90	123.1
110_06s	11	30	6.2	7.70	207.6
110_07s	10	40	6.9	1.70	193.7
110_08s	12	25	6.2	27.6	174.5
110_09s	11	30	6.2	6.50	221.3
110_10s	8.0	40	6.9	2.70	237.2

CHAPTER V

DESIGN OF STUDY BUILDING

The objective of this study is to model the performance of a typical reinforced concrete (RC) office building in St. Louis during an earthquake. The building is a moment frame system and is not specially designed for ductile behavior. Elevator and stairwell shafts were omitted to simplify design and analysis. The floor system is a flat-plate slab with perimeter beams designed to resist lateral loads.

Exterior dimensions for the building were chosen based on the responses of practicing engineers to a questionnaire. A copy of this questionnaire is provided as Appendix B. The building is 140 feet long by 112 feet wide, and each bay is 28 feet between the centerlines of the columns. Practicing engineers recommended a five-story office building as a typical height RC structure to the Midwest region. The first story is 15 feet high, and the heights of the remaining four stories are 13 feet. After the type of structural system and exterior dimensions were determined based on input from practicing engineers, the 1984 BOCA code was used to compute the design loads for the building.

1984 BOCA DESIGN LOADS

Design loads for buildings are discussed in chapter nine of the 1984 BOCA code (*Building officials* 1984). The code requires that a 20-pounds-per-square-foot (psf) partition load be applied to each floor in addition to the weight of the structural members. The weight of exterior cladding is taken into account as a 15-psf load applied to each perimeter beam based on a vertical tributary area. The live load for this office building is 50 psf on each floor and 12 psf on the roof. The snow load for the study building is 12 psf, but it was determined that the snow load did not control in any of the *ACI 318-83* factored load combinations. The wind load was applied as a uniform load distributed vertically on the windward and leeward sides of the building and horizontally on the building's roof. On the windward side, the pressure is 9.6 psf onto the building. The pressure applied to the roof is 12 psf upward, and the leeward wall suction pressure is 6.0 psf.

St. Louis was a part of seismic zone one of the 1984 BOCA code. Seismic loads were applied as a percentage of the base shear at each floor of the building. The code specifies that the base shear for the building is calculated as follows:

$$V = ZKCW \qquad \text{(Eq. 1)}$$

where Z = Seismic zone factor = 0.25

K = Structural system factor = 1.00

C = Coefficient based on fundamental period of building = 0.063

W = Weight of Structure = 13,330 kips.

The base shear calculated using equation one for the study building is 210 kips. This is 1.6 percent of the building's structural weight. Many buildings that are in use today have been designed by assuming that the perimeter frames resist seismic design loads, and this approach was used for the study building. Because the building's perimeter frames were designed to resist the full seismic design loads, half of each floor's portion of the base shear was applied to each of the two exterior frames. The equation given in the 1984 BOCA code for the distribution of the base shear to each floor is shown below.

$$F_x = \frac{V w_x h_x}{\Sigma w_i h_i} \qquad \text{(Eq. 2)}$$

where F_x = Portion of base shear at a given floor level

V = Base shear, as calculated using equation 1

w_x , w_i = Weight of a given floor level

h_x , h_i = Height of a given floor level.

The loads at each floor level of the two exterior frames are shown in Table 5. An elastic structural analysis was performed for each load case, and the factored load combinations of *ACI 318-83* were used to compute design forces. Factored load combinations that include seismic loads controlled the negative moment reinforcement design for the perimeter beams.

TABLE 5. Portion of Base Shear Distributed to Each Floor

Floor Level	Floor Weight (kips)	Floor Height (ft.)	F_x (kips)
2nd	2764	15	8.14
3rd	2748	28	15.1
4th	2748	41	22.1
5th	2748	54	29.1
Roof	2325	67	30.6

DESIGN OF STRUCTURAL MEMBERS

ACI 318-83 was used as the guide for design of the building's structural members. Concrete for this building has a compressive strength (f_c') of 4,000 psi, and the steel reinforcement has a yield strength (f_y) of 60,000 psi. The Direct Design Method for two-way slab design, described in chapter thirteen of *ACI 318-83,* was used in the design of the floor and roof slab systems. The two-way slab is 11 inches thick at every floor level and at the roof level. The perimeter beams are 16 inches wide by 24 inches deep for the second through fifth floors, and the roof perimeter beams are 22 inches deep. Columns are 20 inches square. Shear capitals of 4-inch thickness are used at all interior slab-column connections, including those of the roof. An elevation view is shown in Figure 2, and a floor plan is shown in Figure 3. Table 6, Table 7, and Table 8 show the quantities of reinforcement in the beams, two-way slabs, and columns, respectively. Figure 4 and Figure 5 show important details of the steel reinforcement within the slabs and beams, respectively.

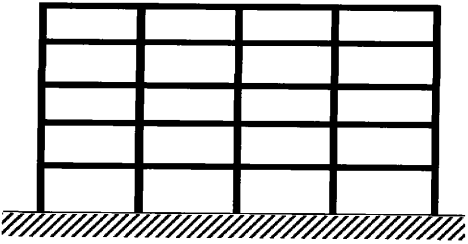


FIG. 2. Elevation View of Exterior Frame of Study Building

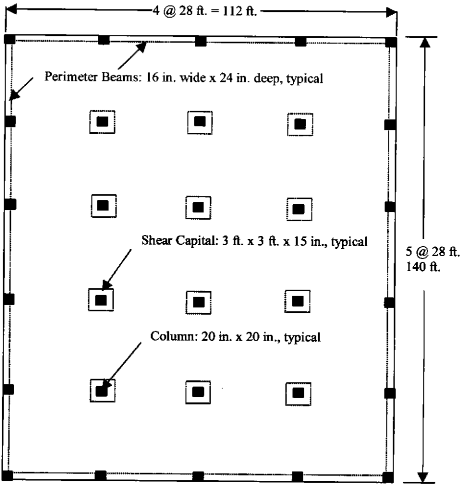


FIG. 3. Plan View of Study Building

TABLE 6. Reinforcement in Perimeter Beams

Floor Level	Beam Width (in.)	Beam Depth (in.)	Pos./Neg. Moment Reinforcement	Num. Of Bars*	Bar Size
2nd-3rd	16	24	Positive	3	#8
			Negative	7	#8
4th	16	24	Positive	3	#8
			Negative	6	#8
5th	16	24	Positive	3	#8
			Negative	5	#8
Roof	16	22	Positive	3	#8
			Negative	6	#8

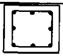
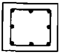
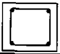
* Number of bars required where magnitude of moment is maximum. Some bars are cut off within beams.

TABLE 7. Reinforcement in Two-way Slab Members

Floor Level	Span	Support	Strip	Strip Width (ft.)	Pos./Neg. Moment Reinforcement	Num. of Bars*	Bar Spacing* (in.)
2nd-5th	Exterior	Exterior	Column	14	Positive	66	2.0
				14	Negative	78	1.7
			Middle	14	Positive	17	9.4
				14	Negative	17	9.4
	Exterior	Interior	Column	14	Positive	66	2.0
				14	Negative	100	1.2
			Middle	14	Positive	17	9.4
				14	Negative	17	9.4
	Interior	Interior	Column	14	Positive	38	3.9
				14	Negative	92	1.3
			Middle	14	Positive	17	9.4
				14	Negative	17	9.4
Roof	Exterior	Exterior	Column	14	Positive	46	3.2
				14	Negative	54	2.6
			Middle	14	Positive	34	4.4
				14	Negative	17	9.4
	Exterior	Interior	Column	14	Positive	46	3.2
				14	Negative	70	1.9
			Middle	14	Positive	17	9.4
				14	Negative	17	9.4
	Interior	Interior	Column	14	Positive	28	5.5
				14	Negative	64	2.1
			Middle	14	Positive	17	9.4
				14	Negative	17	9.4

* #4 bars used for all two-way slab reinforcement

TABLE 8. Reinforcement in Columns

Column Type	Story Level	Column Width (in.)	Num. of Bars	Bar Size	Cross Section
Exterior	1st-5th	20	8	#9	
Interior	1st	20	16	#9	
	2nd	20	8	#9	
	3rd-5th	20	4	#9	

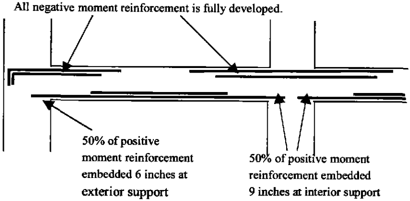


FIG. 4. Details of Slab Reinforcement for Study Building [Adapted from *ACI 318-83* (1983)]

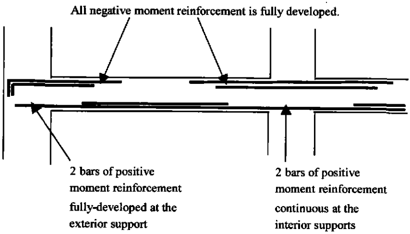


FIG. 5. Details of Beam Reinforcement for Study Building [Adapted from *ACI 318-83* (1983)]

LIMITATIONS OF STUDY BASED ON BUILDING DESIGN

Though the goal of the study was to obtain a building that is most typical of those in St. Louis, there are numerous possibilities of designs for this type of building. In order to clearly define the extent to which these results can be applied, design details that can affect the performance of a building under seismic loading are discussed in this section.

One feature of the study building that affects its seismic performance is the long span of each bay. Surveys of practicing engineers indicate that a typical center-to-center of columns span length is 28 feet. The clear span between columns of this building is 26 feet, 4 inches. Such a long span results in a thick floor slab in order to control deflection (*Building code* 1983). This reduces the building's susceptibility to punching shear failure near its interior slab-column connections. A building with smaller spans may have a thin slab, and its performance during an earthquake will not match the study building's performance.

The size of individual reinforcing bars also affects the performance of a building during an earthquake. When exposed to the cyclic loads of an earthquake, the perimeter beam and slab members may be forced to bend at the supports in a direction opposite to that for which they are designed. Positive moment reinforcement at the support will be forced to carry this new load. When compared to the quantity of negative moment reinforcement, the quantity of positive reinforcement at the support is small. In addition, *ACI 318-83* allows the bottom reinforcing bars in beams and slabs to be cut at fixed distances from the support. These fixed distances do not depend on bar size. In this study, it is assumed that this steel reinforcement can develop a tensile force in the following manner (Aycardi, Mander, and Reinhorn 1994):

$$F_t = \frac{l_{embedment}}{l_{development}} A_s f_y \qquad \text{(Eq. 3)}$$

where F_t = Tensile force

$l_{embedment}$ = Embedment length of a reinforcing bar

$l_{development}$ = Development length of a reinforcing bar (*Building code* 1999)

A_s = Area of steel reinforcement

The proportional relationship of embedment length and development length results in the size of the reinforcing bar greatly affecting the moment capacity of a member at the support. In the study building, #4 reinforcing steel bars are used in the slab, and #8 bars are used in the beams. Before applying this study's results to another building, the size of the reinforcing bars relative to the embedment provided should be examined for that building.

CHAPTER VI

DYNAMIC ANALYSIS USING DRAIN-2DM

DRAIN-2DM calculates the forces that the study building experiences during a ground motion by performing a dynamic time-history analysis of the structure. The program is capable of modeling the behavior of the structural members of the building in the elastic and inelastic ranges. This means that the building's performance will be accurately modeled after permanent deformation has taken place. Inelastic, or nonlinear, behavior is common in buildings that are subjected to seismic loads, and using a dynamic nonlinear analysis computer program is the standard for this field of research.

Over the duration of an earthquake, the ground experiences varying magnitudes of acceleration. A collection of these accelerations with their corresponding time of occurrence forms the ground motion record. The behavior of the building during this ground motion record can be calculated by solving the differential equation of motion shown below.

$$[M]\{a\} + [C]\{v\} + [K]\{u\} = -[M]a_g \qquad \text{(Eq. 4)}$$

where $[M]$ = Mass matrix

$\{a\}$ = Acceleration vector

$[C]$ = Damping matrix

$\{v\}$ = Velocity vector

$[K]$ = Structural stiffness matrix

$\{u\}$ = Displacement vector

a_g = Ground acceleration.

DRAIN-2DM uses the Newmark integration method to solve the differential equation of motion at each time step (Kanaan and Powell 1973). Time steps for this numerical integration can be smaller than the increment of time used in the ground motion record. A time step of 0.002 seconds was used for each analysis in this study. The Newmark integration method assumes a constant acceleration within each time step, and the user must input a Newmark integration factor that specifies how the program will determine the acceleration. In this study, a Newmark integration of 0.5 was used. This value corresponds to an average acceleration during the time step.

A reinforced concrete building is expected to provide between two percent and five percent critical damping when it suffers light to moderate damage during an earthquake (*Response* 1988). Parameters that incorporate damping were included in the model. These parameters were selected based on critical damping of two percent and estimated values of the natural periods of the first and second modes of vibration. The resulting critical damping in the DRAIN-2DM model was calculated as 5.0 percent using the logarithmic decrement of roof displacement amplitude plotted on a time scale (Richart, Woods, and Hall 1970). This amount of damping is reasonable, and the damping parameters were not adjusted further.

ELEMENTS IN DRAIN-2DM

One of DRAIN-2DM's features as an analysis tool is the set of elements that model reinforced concrete columns, beams, and slabs. These elements were developed to duplicate behavior observed in experimental research, and each element models behavior in the elastic and inelastic ranges. At beam-column and slab-column connections, DRAIN-2DM is capable of modeling rigid connections (Kanaan and Powell 1973). This capability yields more accurate results for concrete structures because significant rotation does not occur within the joint of a concrete beam-column or slab-column connection. Rather, rotation will occur at a point outside the joint. The rigid end zones used to define the joint region in the model are shown for a beam-column connection and slab-column connection in Figure 6.

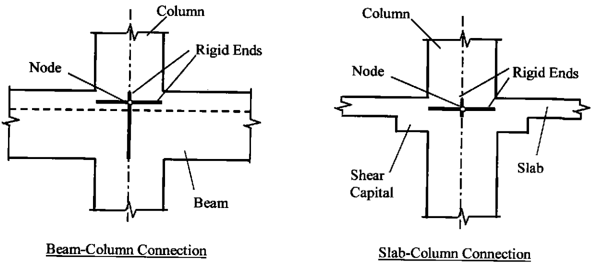


FIG. 6. Rigid End Zones for DRAIN-2DM Model [Adapted from Hueste and Wight (1997)]

Reinforced concrete beam-column elements (Element 2) were used for all columns in the model of the study building. RC beam-column elements require stiffness parameters and yield interaction surfaces to describe the members. Stiffness parameters include cross-sectional area, Young's modulus of elasticity, strain hardening modulus, and moment of inertia. The post-yield stiffness used in this study is two percent of the initial elastic stiffness. Flexural stiffness is assumed to be concentrated at the member ends. After the stiffness parameters are set, values are input into the DRAIN-2DM program so that a yield interaction surface is defined for use in determining when inelastic stiffness takes place. Figure 7 shows a yield interaction surface for a reinforced concrete beam-column element. Axial force (P) and moment (M) values must be input for points A, B, C, D, E, and F. Output from a DRAIN-2DM analysis provides the axial force, shear force, bending moment, displacements, and rotation at each end of a column for a set of time steps defined by the user (Kanaan and Powell 1973). When the axial force and moment at a given time step reach the yield interaction surface, flexural yielding occurs. The bilinear relationship used to model the elastic and inelastic behavior at the member ends is shown in Figure 8.

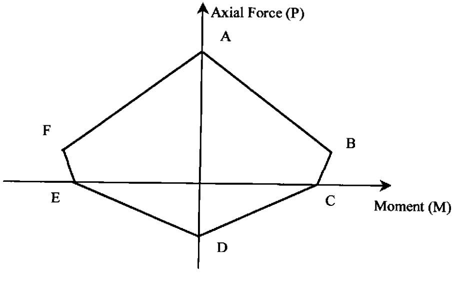


FIG. 7. Yield Interaction Surface for Reinforced Concrete Beam-Column Elements [Adapted from Soubra, Wight, and Naaman (1992)]

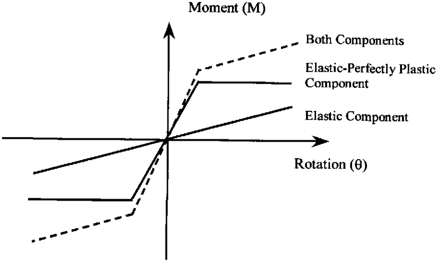


FIG. 8. Bilinear Moment-Rotation Relationship for Reinforced Concrete Beam-Column Elements [Adapted from Soubra, Wight, and Naaman (1992)]

Perimeter beams within the study building are modeled with DRAIN-2DM's reinforced concrete beam element (Element 8). The input of stiffness parameters is identical to that of RC beam-column elements. RC beam elements require the input of the positive and negative yield moments and yield curvatures at each end of the element. In this study, the moment capacity of the beam without a strength reduction factor was selected as a reasonable estimate of the yield moment capacity. As discussed in Chapter V, only the steel reinforcement that can be developed according to *ACI 318-99* was used in the moment capacity calculation. Yield curvature was calculated as follows:

$$\phi_y = \frac{M_y}{EI} \qquad \text{(Eq. 5)}$$

where ϕ_y = Yield curvature

M_y = Yield moment

E = Young's modulus for concrete

I = Moment of inertia for beam cross-section.

The moment of inertia used in the model of the study building assumes that the member is cracked. For beams, the cracked moment of inertia value is the gross moment of inertia multiplied by a factor of 0.35. The corresponding factors for column and slab members are 0.70 and 0.25, respectively (*Building code* 1999). Additional parameters input for RC beam elements are the locations of inelastic flexural springs, or plastic hinges (Raffaelle and Wight 1992). Plastic hinge locations are input by the user as an instruction to the program to monitor a particular location for inelastic rotation. In this study, inelastic flexural springs are assigned to RC beam elements at the point where the beam meets the face of the column. These inelastic flexural spring locations are shown as part of the idealized beam element in Figure 9. The general form of the hysteretic model in a reinforced concrete beam element that deforms beyond the elastic region during cyclic loading is shown in Figure 10. This model describes the moment-rotation behavior for the inelastic flexural springs based on input parameters defined by the user. The parameters used in this study were a pinching factor of 0.75 and an unloading stiffness factor of 0.30. No strength reduction was used for the hysteretic model.

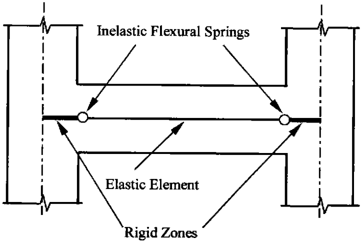


FIG. 9. Idealized Reinforced Concrete Beam Element [Adapted from Raffaelle and Wight (1992)]

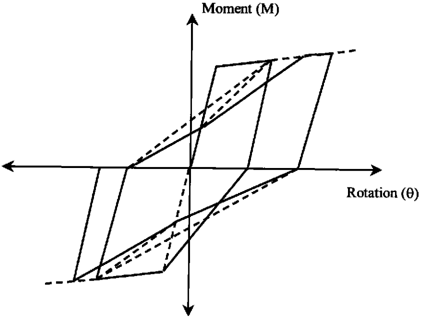


FIG. 10. Generalized Model for the Hysteretic Behavior of the Reinforced Concrete Beam Element [Adapted from Raffaelle and Wight (1992)]

The study building's two-way slab floor and roof system are modeled with reinforced concrete slab elements (Element 11). The input and model behavior of a RC slab element is similar to those of a RC beam element. The primary difference between the two elements is that the RC slab element is capable of predicting a punching shear failure near a slab-column connection (Hueste and Wight 1997). In a building, a punching shear failure is a rupture of the slab along a perimeter around a column. A segmented linear model is used for predicting punching shear failure. The two parameters for this model are gravity shear ratio (V_g/V_o) and critical rotation (θ_{cr}). The gravity shear ratio is the ratio of the shear at a slab-column connection due to gravity loads and the unreduced vertical shear strength of the critical section around the column, described in Section 11.12.1.2 of *ACI 318-99.* Because the interior slab-column connections for the study building include shear capitals, the gravity shear ratio must be calculated for the critical sections around the column and the shear capital. The maximum value of the gravity shear ratio is input into the program. The critical rotation (θ_{cr}) for the model shown in Figure 11 was determined as the average negative rotation that occurs in a slab element when the building's lateral drift is 1.25 percent based on recommendations by Hueste and Wight (1999). This value was determined by conducting a static pushover analysis until a 1.25 percent average building drift was reached. A triangular distribution of lateral forces over the building was assumed based on the distribution from the code static force procedure (*Building officials* 1984). If a slab element rotates to a value beyond the model boundary shown in Figure 11 for that element's gravity shear ratio, a punching shear failure is predicted. After a punching shear failure is predicted, the moment capacity for the slab element is reduced to ten percent of the moment at which the failure occurred. This moment capacity reduction takes place over a period of nine time steps and has the effect of reducing the rotational stiffness of the inelastic flexural spring at that members end.

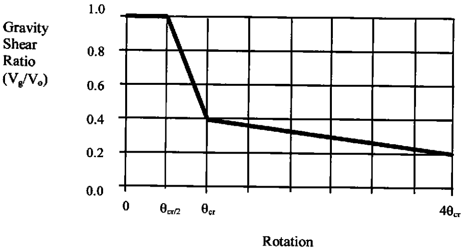


FIG. 11. Punching Shear Failure Model Using Gravity Shear Ratio Versus Member End Rotation [Adapted from Hueste and Wight (1997)]

BUILDING MODEL

Only the forces resulting from the effect of dead loads were applied to the DRAIN-2DM building model prior to subjecting the model to a particular ground motion record. Lumped masses based on the seismic dead load were assigned to the nodes in the model. For the analysis, an exterior frame and two interior frames along the short direction of the building are tied together with rigid truss-type elements. This model takes advantage of the building's symmetry so that only half of the building is analyzed. The model is shown in Figure 12. The truss-type elements used to connect the frames are rigid and transmit only axial force and displacement between frames. These elements are denoted by dashed lines in Figure 12. All vertical members in Figure 12 are modeled as reinforced concrete beam-column elements (Element 2). Horizontal members of the exterior frame shown in Figure 12 are modeled as reinforced concrete beam elements (Element 8). Horizontal members of the two interior frames shown in Figure 12 are modeled as reinforced concrete slab elements (Element 11).

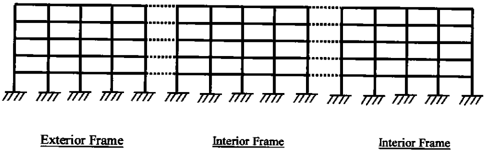


FIG. 12. Model of Study Building Used in DRAIN-2DM Analysis

CHAPTER VII

RESULTS

A dynamic time-history analysis was performed for each ground motion in the two percent and ten percent probability of exceedance in 50 years ground motion record sets described in Chapter IV. In addition, a dynamic time-history analysis using a portion of the ground motion record from the 1940 El Centro, California, earthquake was conducted for this study. Plots of these ground motion records are shown in Appendix A. Output from DRAIN-2DM included the displacements, rotations, forces, and moments for all structural members. In addition, the program produced numerical codes describing the behavior of the beam-column and slab-column inelastic flexural springs. These codes indicate behavior within the elastic and inelastic ranges, including the location on the hysteretic model for Elements 8 and 11.

BUILDING RESPONSE FOR SYNTHETIC GROUND MOTION RECORDS

The response of the study building varied greatly between the two percent probability of exceedance in 50 years ground motion set and the ten percent probability of exceedance in 50 years ground motion set. During six of the ten ground motion records in the two percent probability of exceedance set, the building collapsed. The median response, which is the best estimate of the two percent probability of exceedance in 50 years response, was the collapse of the building. The median response is the average of the responses of the building that cause the fifth- and sixth-largest base shear and building drift values. The l02_07s and l02_10s ground motion records determined the median response for the two percent probability of exceedance in 50 years set.

Shear failure in the first-story columns was the mechanism of collapse for the median response of the two percent probability of exceedance in 50 years ground motion record set. This failure occurred within two seconds of the start of the ground motion. This rapid collapse necessitated examining various modes of failure for the building model in order to determine the actual failure mechanism. Modes of failure examined were beam shear failure, column shear failure, and slab punching shear failure. In the cases of beam shear failure and column shear failure, the shear strength (V_n) of each member was compared to the shear force within the member at a particular time step. If the force in the member exceeded the member's strength, a failure was judged to have occurred. Beam shear strength was calculated as follows (*Building code* 1999):

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(Eq. 6)}$$

$$V_c = 2f_c'^{1/2} b_w d \qquad \text{(Eq. 7)}$$

$$V_n = V_s + V_c \qquad \text{(Eq. 8)}$$

where V_s = Shear strength of shear reinforcement, psi

A_v = Area of shear reinforcement, inches^2

d = Distance from exteme compression fiber of beam to centroid of longitudinal tension reinforcement, inches

s = Spacing of shear reinforcement in direction of longitudinal tension reinforcement, inches

V_c = Shear strength of concrete, psi

b_w = Web width, inches

The shear strength of columns is calculated using the same procedure, except that the following equation is used rather than Equation 7 for the calculation of shear strength of concrete (*Building code* 1999):

$$V_c = 3.5 f_c'^{1/2} b_w d (1 + N_u/(500A_g))^{1/2} \quad \text{(Eq. 9)}$$

where: N_u = Axial force at time step being examined, lb_f.

A_g = Gross area of column cross section, inches^2.

Slab members were checked for punching shear failures by the model incorporated into the DRAIN-2DM program. This model is described in Chapter VI. The time steps at which these three types of failure occurred were determined, and the failure mechanism that occurred first during the ground motion record was judged to be the actual mode of failure. For the study building's median response to the two percent probability of exceedance in 50 years ground motion records set, shear failure in the first-story columns caused the building to collapse. A description of the building's responses to the ten ground motion records of the two probability of exceedance in 50 years set is shown in Table 9. In this table, building drift is the horizontal roof displacement expressed as a percentage of the building height. Inter-story drift is the relative horizontal displacement of one floor expressed as a percentage of the story height.

The median response of the building to the ten percent probability of exceedance in 50 years set of ground motion records was completely elastic behavior by the building's structural members. While this is an encouraging result, it is noted that a ten percent probability of exceedance in 50 years ground motion is the equivalent of a small earthquake. The fifth- and sixth-largest responses to the ground motion records of the ten percent probability of exceedance in 50 years set that determined the median response were the 110_09s and 110_01s records, respectively. The body-wave magnitudes of these two ground motion records are 6.0 and 6.2, respectively. These values of body-wave magnitude are similar to those of the 20 earthquakes that have occurred in the NMSZ since 1812. Detailed descriptions of the responses by the study building to the ground motion records of the ten percent probability of exceedance in 50 years set are shown in Table 10.

TABLE 9. Response of Study Building to Ground Motion Records of the Two Percent Probability of Exceedance in 50 Years Set

Ground Motion Label	Peak Ground Acceleration (% of g)	Ground Motion Duration (sec)	Base Shear (kips)	Base Shear (% of W*)	Maximum Building Drift (%)	Inter-story Drift Max. (%)	Inter-story Drift Min. (%)
102_01s	23	70	685	10.0	0.54	0.80	0.29
102_02s	25	70	624	9.1	0.55	0.76	0.25
102_03s	83	10	COLLAPSE				
102_04s	25	45	464	6.8	0.33	0.48	0.25
102_05s	19	55	COLLAPSE				
102_06s	24	40	719	10.5	0.55	0.80	0.26
102_07s	24	70	COLLAPSE				
102_08s	24	35	COLLAPSE				
102_09s	25	35	COLLAPSE				
102_10s	54	20	COLLAPSE				

*W = Weight of one exterior frame and two interior frames.

TABLE 10. Response of Study Building to Ground Motion Records of the Ten Percent Probability of Exceedance in 50 Years Set

Ground Motion Label	Peak Ground Acceleration (% of g)	Ground Motion Duration (sec)	Base Shear (kips)	Base Shear (% of W*)	Maximum Building Drift (%)	Inter-story Drift Max. (%)	Inter-story Drift Min. (%)
110_01s	13	25	347	5.1	0.51	0.37	0.10
110_02s	10	40	148	2.2	0.11	0.16	0.07
110_03s	9	40	588	8.6	0.43	0.64	0.17
110_04s	11	25	118	1.7	0.06	0.09	0.08
110_05s	13	20	437	6.4	0.32	0.47	0.13
110_06s	11	30	1000	14.6	1.37	2.76	0.35
110_07s	10	40	146	2.1	0.11	0.16	0.08
110_08s	12	25	902	13.2	0.77	1.24	0.26
110_09s	11	30	294	4.3	0.21	0.32	0.09
110_10s	8	40	145	2.1	0.11	0.16	0.09

*W = Weight of one exterior frame and two interior frames.

No punching shear failures occurred in the building during the ground motions of the ten percent probability of exceedance in 50 years set. As discussed in Chapter VI, part of the basis for DRAIN-2DM's prediction of a punching shear failure is the gravity shear ratio. This is the ratio of the shear that results from the application of gravity loads and the shear strength along the perimeter of a critical section around the column defined in section 11.12.1.2 of *ACI 318-99*. The gravity shear ratio of the second through fifth floors' two-way slab system is 0.27. The gravity shear ratio of the roof's two-way slab system is 0.28. Punching shear failure usually becomes a concern when the gravity shear ratio is 0.40 or greater for building drifts of 1.0 to 1.5 percent (Hueste and Wight, 1997). Because of the low gravity shear ratio in each slab and the fact that the building drift only exceeds 1.0 percent during one ground motion record, DRAIN-2DM's prediction of no punching shear failure during a low-magnitude ground motion is consistent with expectations for this building.

BUILDING RESPONSE DURING THE 110_06s GROUND MOTION RECORD

The building has an inelastic response to several ground motions between the median responses of the two categories. For some ground motion records in the two percent probability of exceedance set during which the building responded below the corresponding median level (collapse) and ground motion records in the ten percent probability of exceedance set during which the building responded above the median level (elastic behavior), inelastic behavior occured. An exact probability of exceedance in 50 years for these ground motion records is not available, but it can be reasoned that the probability of exceedance is between two percent and ten percent. The most damaging building response to the ground motion records that did not cause collapse occurred during the 110_06s ground motion record. For this ground motion, the maximum building drift was 1.37 percent. During the 110_06 ground motion record, the maximum base shear experienced by the building was 1000 kips. The perimeter beams of the study building were designed to resist seismic loads, but the analysis shows that 72 percent of the base shear was distributed to the two interior frames in the model. The remaining 28 percent was distributed to the exterior frame. However, the high percentage of base shear distributed to the interior frames did not seem to be detrimental to the structure for this ground motion. Detailed results of the response of the model to the 110_06s ground motion record are shown in Table 11. The building drift is plotted versus time in Figure 13.

TABLE 11. Response of Study Building to the I10_06s Ground Motion Record

Story	Maximum Inter-story Drift (%)	Time of Max. I-S Drift (seconds)	Maximum Story Shear (kips)	Time of Max. Story Shear (seconds)
5th	0.35	0.61	254.8	0.60
4th	0.67	0.63	521.2	0.58
3rd	1.13	0.71	724.1	0.66
2nd	1.94	1.31	841.9	0.78
1st	2.76	1.18	1000	0.97

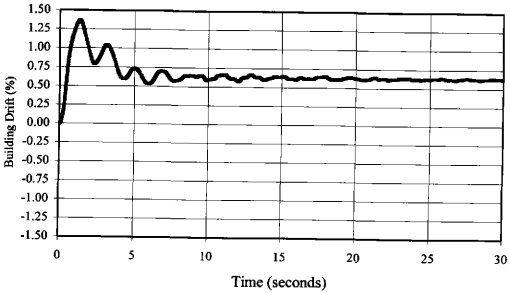


FIG. 13. Building Drift (%) During the I10_06s Ground Motion Record

In response to the I10_06s ground motion, the perimeter beams of the building experienced inelastic rotation at 21 locations on the exterior frame. On the interior frame, the slab elements rotated inelastically at three locations. The locations of inelastic rotations within the beam, slab, and column elements due to the 110_06s ground motion record are shown in Figure 14. As previously discussed, inelastic rotations in the slab members did not reach the level required to cause punching shear failures.

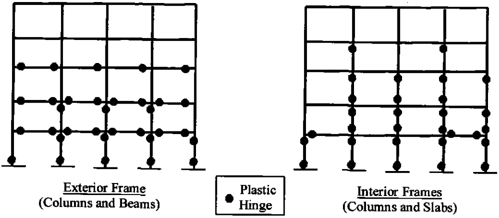


FIG. 14. Locations of Inelastic Rotation in Column, Beam, and Slab Members During the I10_06s Ground Motion Record

The exterior frame of Figure 14 shows the locations of inelastic rotation in the perimeter beams and columns. As the beam members rotate inelastically, their stiffness degrades in the inelastic range of behavior. This is demonstrated in Figures 15 and 16, in which moment is plotted versus rotation for second-floor beams for the inelastic flexural springs located at each end. The plots contain moment and rotation data for the duration of the 110_06s ground motion record. Figure 15 shows this behavior at the exterior column connection of the second-floor, first-span beam. Figure 16 shows the behavior of the same beam at the column face of the first interior column. Both locations of the beam exhibit linearly elastic behavior at the beginning of the ground motion, demonstrated by the moment-rotation plot either passing through the origin or having a slope that passes through the origin.

It is shown in Figure 14 that inelastic rotation occurred in the slab members of the interior frames during the I10_06s ground motion record. Unlike the second-floor beam members of the exterior frame at the beam-column connections, all second-floor slab members *did not rotate inelastically* at the slab-column connections. Figure 17 shows the moment-rotation diagram for the second-floor, first-span slab interior column connection. Figure 18 shows the moment-rotation diagram for the second-floor, fourth-span slab interior column connection. The moment-rotation diagram for the second-floor, fourth-span slab exterior column connection is shown in Figure 19. Comparing Figure 17 to Figure 18 and Figure 19 demonstrates the difference between the linear behavior that results from elastic rotation and the nonlinear behavior of inelastic rotation.

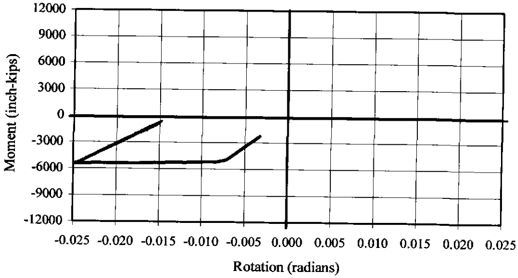


FIG. 15. Moment-Rotation Diagram for Second-Floor Beam at Exterior Column End of the First Span During the I10_06s Ground Motion Record

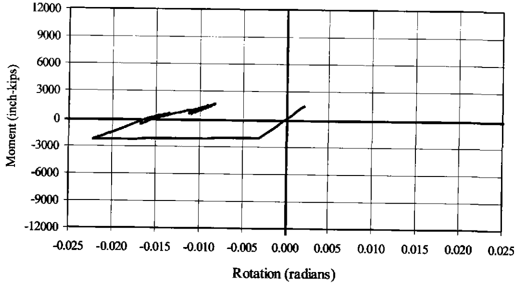


FIG. 16. Moment-Rotation Diagram for Second-Floor Beam at Interior Column End of First Span During the I10_06 Ground Motion Record

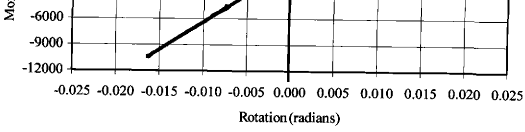


FIG. 17. Moment-Rotation Diagram for Second-Floor Slab at Interior Column End of First Span During the I10_06s Ground Motion Record

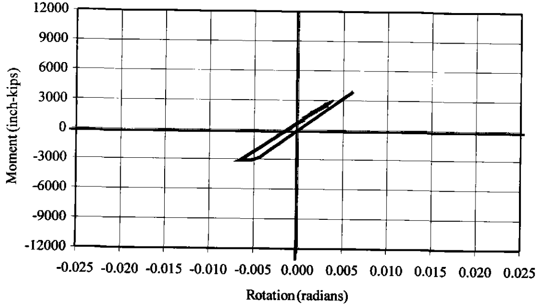


FIG. 18. **Moment-Rotation Diagram for Second-Floor Slab at Interior Column End of Fourth Span During the I10_06s Ground Motion Record**

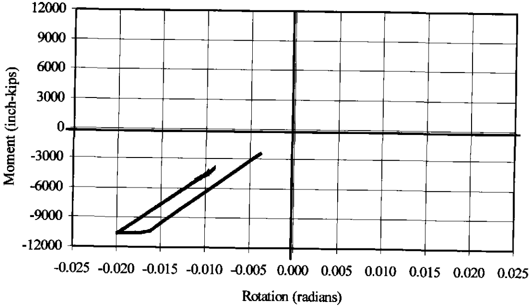


FIG. 19. **Moment-Rotation Diagram for Second-Floor Slab at Exterior Column End of Fourth Span During the I10_06s Ground Motion Record**

BUILDING RESPONSE DURING THE EL CENTRO EARTHQUAKE GROUND MOTION RECORD

A ten-second portion from the 1940 El Centro, California, earthquake was used in an analysis of the study building. Using a ground motion record from a West Coast earthquake provided interesting results and a reference point for comparison to other studies of buildings that have used this ground motion record. The peak ground acceleration of this ground motion record is 0.35g. The maximum base shear experienced by the study building during the El Centro ground motion record was 978.0 kips. The distribution of the base shear between the exterior frame and the two interior frames was identical to that of the 110_06s ground motion record. The distribution of the base shear to the exterior frame was 28 percent, and the interior frames experienced 72 percent of the base shear. Details of the study building's response are shown in Table 12, and the building drift is plotted versus time in Figure 20.

TABLE 12. Response of Study Building to the El Centro Earthquake Ground Motion Record

Story	Maximum Inter-story Drift (%)	Time of Max. I-S Drift (seconds)	Maximum Story Shear (kips)	Time of Max. Story Shear (seconds)
5th	0.60	2.23	564.4	2.49
4th	0.92	5.61	700.0	2.24
3rd	1.25	5.61	783.8	5.59
2nd	0.99	5.32	806.7	5.30
1st	1.25	5.41	978.0	5.40

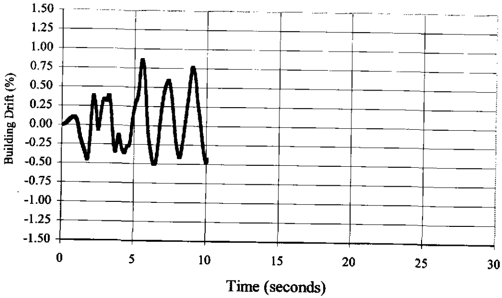


FIG. 20. Building Drift (%) During the El Centro Earthquake Ground Motion Record

During the El Centro ground motion record, no slab members behaved in an inelastic manner. The perimeter beams of the exterior frame did rotate inelastically. The locations of these rotations are shown in Figure 21. The hinging pattern shown in Figure 21 is desirable in that inelastic activity occurs in the beam members only and not in the more critical column members. The moment-rotation diagram for the ends of beam members during the El Centro ground motion record show more cyclic behavior than the moment-rotation diagrams from the 110_06s ground motion record. This increased cyclic behavior is caused by the large acceleration magnitudes of the El Centro ground motion. Examples of the cyclic behavior of the perimeter beams for the El Centro ground motion record are shown in Figure 22 and Figure 23. Figure 22 shows moment versus rotation for the perimeter beam end at the exterior column in the first span of the second floor. Figure 23 shows moment versus rotation at the interior column of the first span of the second floor perimeter beams.

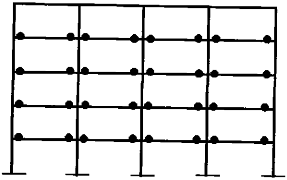


FIG. 21. Locations of Inelastic Rotation in Perimeter Beam Members of Exterior Frame During the El Centro Earthquake Ground Motion Record

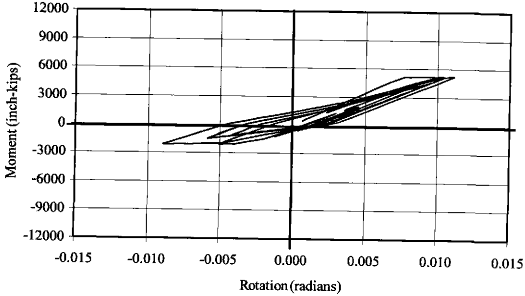


FIG. 22. Moment-Rotation Diagram for Second Floor Perimeter Beam at Exterior Column End of First Span During the El Centro Earthquake Ground Motion Record

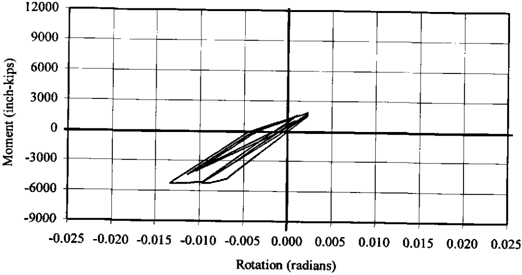


FIG. 23. Moment-Rotation Diagram for Second Floor Perimeter Beam at Interior Column End of First Span During the El Centro Earthquake Ground Motion Record

CHAPTER VIII

CONCLUSION

The most disturbing conclusion from this study is that a typical reinforced concrete office building similar to the study building in St. Louis will be severely damaged in a low probability, New Madrid Seismic Zone earthquake. During the shaking produced by the two percent probability of exceedance in 50 years ground motion record, the building collapsed within seconds. The magnitude of this ground motion record is similar to the magnitudes of the three large New Madrid Earthquakes of 1811-12, and it is concluded that the recurrence of such an earthquake would result in the collapse of St. Louis buildings that are similar to the study building.

During small, higher-probability earthquakes, a building like the study building should perform in a satisfactory manner. The structural members of the study building showed elastic behavior when subjected to a ground motion with a ten percent probability of exceedance in 50 years. This is an encouraging result because twenty earthquakes of magnitudes similar to that of a ten percent in probability of exceedance in 50 years ground motion record have occurred since 1812. It is likely that this type of earthquake will be experienced by buildings that currently exist in St. Louis.

For earthquakes with probabilities between the two percent and ten percent probabilities of exceedance in 50 years, definite conclusions cannot be drawn. During some ground motions between these exceedance probabilities, the building will experience damage at its beam-column and slab-column connections without collapse. Punching shear failure was not exhibited in the response of the building to these ground motion records. One reason that punching shear was not an issue in the study building was the fact that the two-way slab was thick in order to control deflections. Reinforced concrete buildings in St. Louis with shorter spans and thinner two-way slab systems may behave in a different manner than the study building.

RECOMMENDATIONS FOR FUTURE RESEARCH

Future research that tests the responses of various types of buildings in St. Louis will increase to the ability of experts to predict the damage due to the next NMSZ earthquake. Examining the dynamic response of Midwestern steel, wood, masonry, and composite structures provides information that can be useful to engineers, geoscientists, and disaster response planners in preparing for future seismic events.

More research on the response of reinforced concrete structures during a NMSZ earthquake is also necessary. This thesis has documented aspects of the building design that could vary within reinforced concrete buildings, such as the length of spans between columns and the size of steel reinforcement within the two-way slab system. These variations can change the response of a building during an earthquake. Because of these variations, more studies that analyze reinforced concrete buildings with different structural details are needed before predictions concerning the behavior of a majority of buildings can be made. The nonlinear dynamic analysis computer program DRAIN-2DM can be enhanced by adding a model that determines whether a reinforced concrete column shear failure occurs. This addition would be of benefit to future studies of reinforced concrete structures.

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APPENDIX A

GROUND MOTION RECORDS USED FOR ANALYSIS

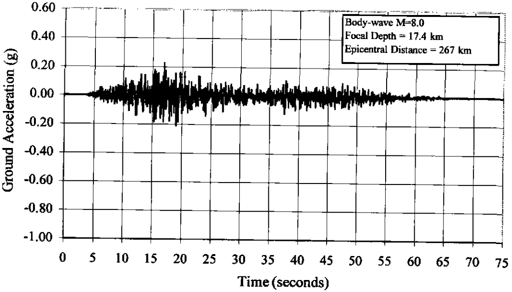


FIG. A1. Two Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 1 [Adapted from Wen and Wu (2000)]

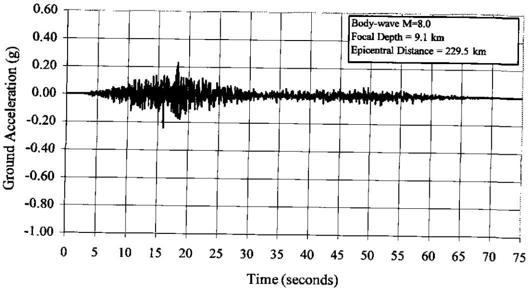


FIG. A2. Two Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 2 [Adapted from Wen and Wu (2000)]

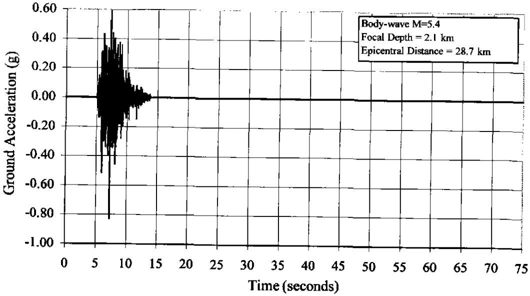


FIG. A3. Two Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 3 [Adapted from Wen and Wu (2000)]

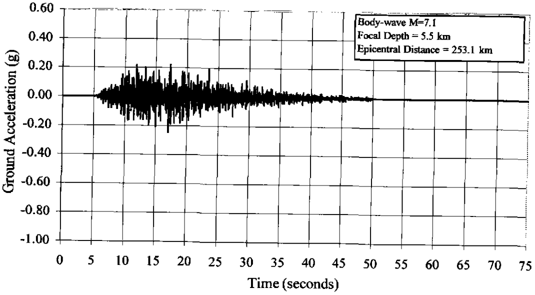


FIG. A4. Two Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 4 [Adapted from Wen and Wu (2000)]

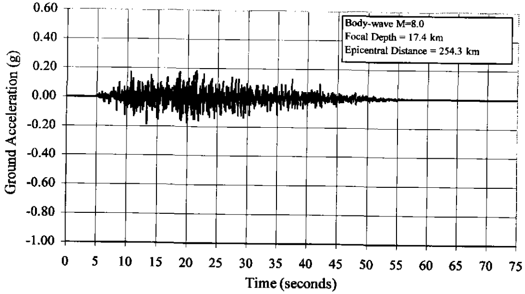


FIG. A5. Two Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 5 [Adapted from Wen and Wu (2000)]

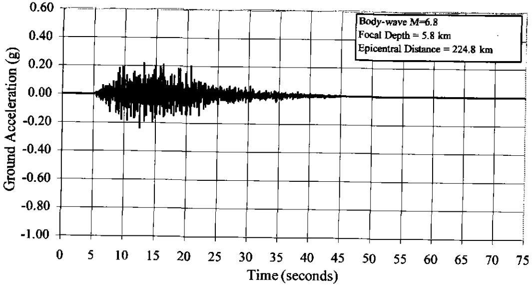


FIG. A6. Two Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 6 [Adapted from Wen and Wu (2000)]

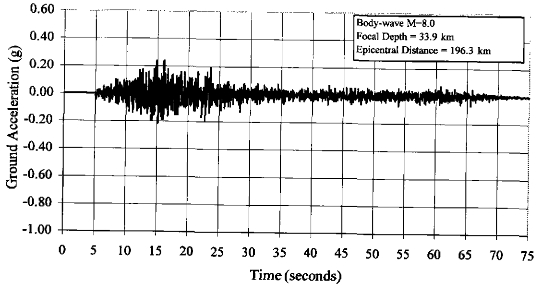


FIG. A7. Two Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 7 [Adapted from Wen and Wu (2000)]

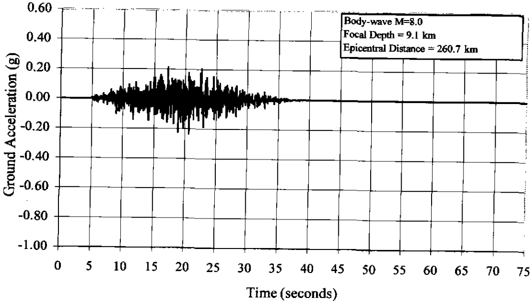


FIG. A8. Two Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 8 [Adapted from Wen and Wu (2000)]

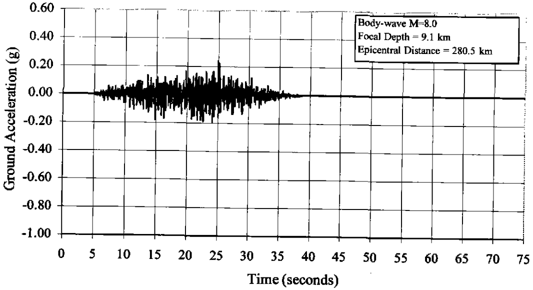


FIG. A9. Two Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 9 [Adapted from Wen and Wu (2000)]

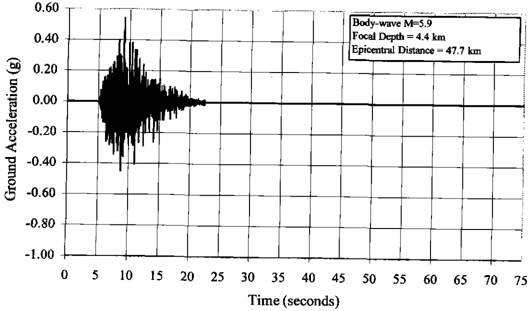


FIG. A10. Two Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 10 [Adapted from Wen and Wu (2000)]

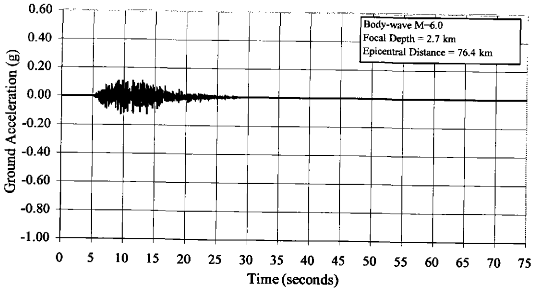


FIG. A11. Ten Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 1 [Adapted from Wen and Wu (2000)]

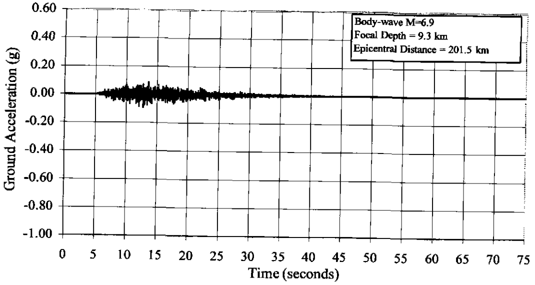


FIG. A12. Ten Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 2 [Adapted from Wen and Wu (2000)]

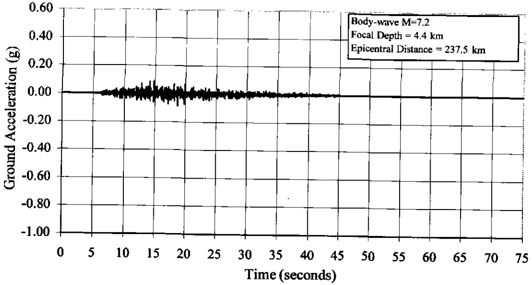


FIG. A13. Ten Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 3 [Adapted from Wen and Wu (2000)]

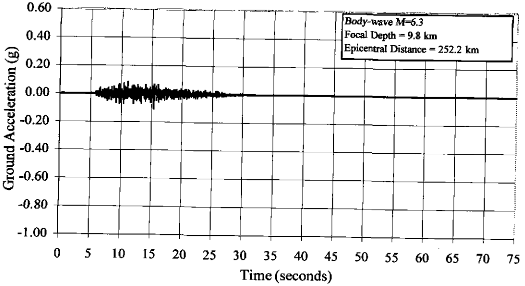


FIG. A14. Ten Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 4 [Adapted from Wen and Wu (2000)]

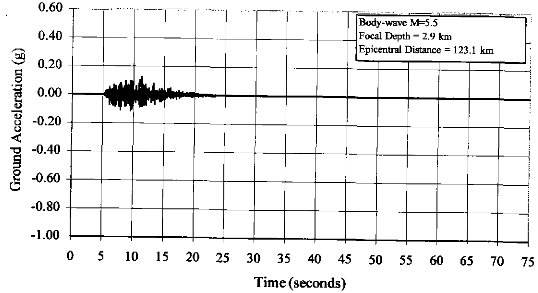


FIG. A15. Ten Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 5 [Adapted from Wen and Wu (2000)]

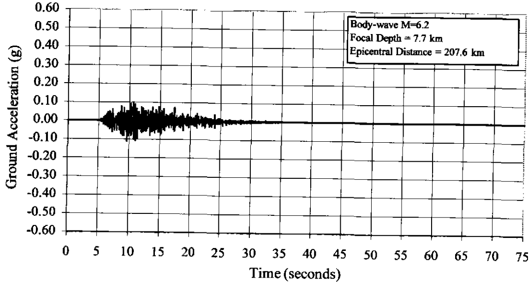


FIG. A16. Ten Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 6 [Adapted from Wen and Wu (2000)]

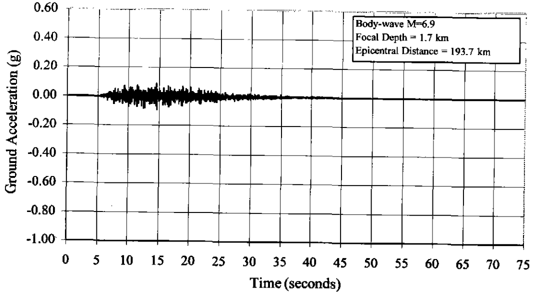


FIG. A17. Ten Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 7 [Adapted from Wen and Wu (2000)]

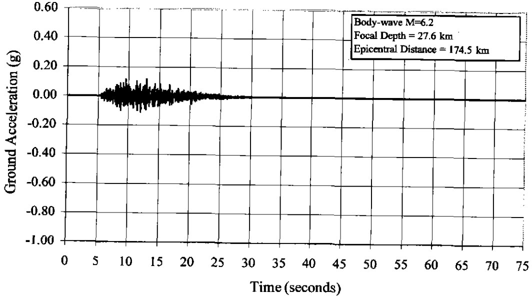


FIG. A18. Ten Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 8 [Adapted from Wen and Wu (2000)]

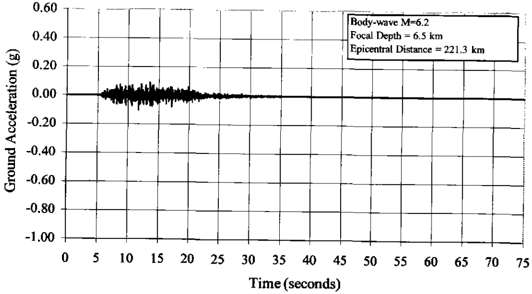


FIG. A19. Ten Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 9 [Adapted from Wen and Wu (2000)]

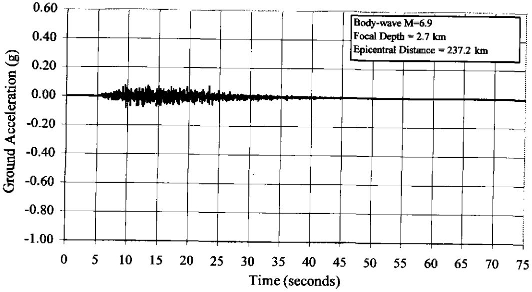


FIG. A20. Ten Percent Probability of Exceedance in 50 Years, Representative Soil, Record No. 10 [Adapted from Wen and Wu (2000)]

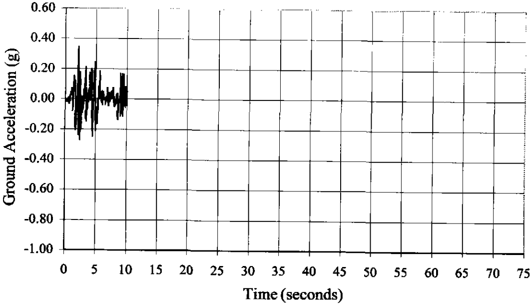


FIG. A21. El Centro Earthquake Ground Motion Record [Adapted from Hueste and Wight, (1997)]

APPENDIX B

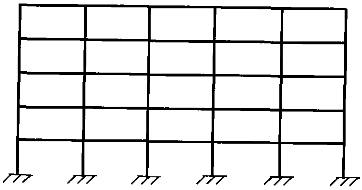
QUESTIONNAIRE SENT TO PRACTICING ENGINEERS

Dear Sir or Madam:

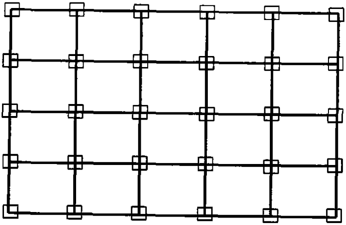
As the basis for my senior thesis, I am modeling the effect of the ground motion record from an earthquake similar to the New Madrid earthquakes in the early nineteenth century on a typical reinforced concrete frame structure in St. Louis or Memphis built before 1990.

Originally, my objective was to obtain design drawings of an existing building to serve as the modeled structure, but I have been unable to obtain these drawings. As an alternative, I will design a frame structure similar to the one shown below:

Elevation view:



Plan View:



Because I would like to obtain results for the Midwest region, it is important that I design this structure to be typical of those found in St. Louis or Memphis. In order to do this, I need recommendations and comments from an engineer who has designed structures in these cities. It would help me tremendously if you would answer the following questions and provide any other comments you feel would be helpful about designing this building or the project as a whole. Thank you for your time and help.

Sincerely,

Jason Hart

1) Is a flat slab floor appropriate?

   1a) If so, would a flat slab with perimeter moment frames be most typical?

   1b) What would be a typical thickness of a flat slab in St. Louis or Memphis?

   1c) If slab-to-column systems are common, should capitals and/or drop panels be used?

2) Is a one-way slab common in St. Louis or Memphis?

3) Are shear walls so common in St. Louis or Memphis that they should be included in this model?

4) Are pan joists so common in St. Louis or Memphis that they should be included in this model?

5) Is there a number of stories for buildings that is exceedingly common in St. Louis or Memphis, or will a five-story building as shown be typical?

6) What would you recommend as the typical story height for buildings in St. Louis or Memphis?

7) Is the dominant design philosophy in the region that the external frames resist all lateral load in the moment frame structure, or are the interior frames also designed to carry a portion of the lateral load?

8) In pre-1990 reinforced concrete structures in the St. Louis/ Memphis region, is discontinuous bottom reinforcement common at interior beam-column connections?

9) What would you recommend as a typical span between columns for a structure in St. Louis or Memphis?

VITA

Jason Frazier Hart was born in Dallas, Texas, on April 4, 1978. After graduating from Red Oak (Texas) High School in 1996, he attended Texas A&M University in College Station, Texas. During four years as an undergraduate student at Texas A&M, he completed a research project as part of the Undergraduate Research Fellows program and the requirements of the Engineering Scholars Program. He graduated in May 2000 with a Bachelor of Science degree in Civil Engineering. His research interests include earthquake engineering and the behavior of reinforced concrete structures. He can be contacted through the following address:

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