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Earthquake Hazard Reduction in the Central and Eastern United States:
A Time for Examination and Action

HAZARD ASSESSMENT/DAMAGE MITIGATION

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ABSTRACT

Two multi-level parking structures for an important medical complex proved an excellent example of the problems facing the owners and designers of conventional structures in low or moderate seismic hazard areas.

The two structures are to be of precast concrete, but differing uses require different design philosophies. In addition, varying subsurface conditions altered the foundation designs and resulted in different earthquake design provision. The design earthquake ground motion levels were 4% and 6% of gravity for the design event. A computer-aided, building code type design was selected for both structures. Additional seismic related construction costs of about 2% for the "rock" site, and 4% to 6% for the "soil" site are estimated.

INTRODUCTION

Seismic hazard studies for major (about $1 billion) structures can cost on the order of one million dollars. Such studies are not easily funded for structures whose total construction cost is in the 3 to 10 million dollar category. The authors' work usually encompasses such "ordinary" structures. Hence, we are forced into evaluating the suitability of the applicable building code parameters and the nature of the subsurface with a degree of conservatism appropriate for a particular design, and with the need to perform these evaluations and analyses in a cost-effective manner. This paper describes two such typical(?) projects, the rationale employed, and the conclusions drawn.

The proposed buildings are parking garages, considered quite prosaic a structure by many. However, conventional parking structures are expansive, open buildings, usually constructed of precast, or sometimes prestressed concrete. They become less simple when earthquake design must be considered. Complicate this problem further by either placing the

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structures near or immediately adjacent to hospital facilities and populate them with both health care personnel and visitors on a 24-hour basis and these prosaic garages become most-important engineered structures.

The sites are just to the north of New York City. The planned structures support different facilities in a huge County medical complex. The County building code would usually govern design. However, there are no earthquake provisions included in either County or State codes. Both BOCA and UBC codes place the site in Zone 2. The owner and designers decided that some seismic resistance should be incorporated into the structures. Two design events were postulated, an expected (elastic design) shock and a "500-year" (elasto-plastic design) earthquake.

FOUNDATION CONDITIONS

General

Paleozoic-aged(?) mica schists of the Manhattan Formation likely underlie both parking structure locations. These rocks stretch northeasterly from the New York City area for some 50 miles. These rocks have been folded and faulted by several series of tectonic events. The region was later subjected to several periods of glaciation.

Site 1

"Decomposed" rock was found either at the surface or below a thin cover of topsoil or fill at the site. The "decomposed" rock was underlain by "sound" rock. All rock encountered within the building area was mica schist with steeply dipping foliation. The site apparently lies atop a thrust block overlying the (sometimes solutioned) Inwood Marble, another Manhattan Prong metamorphic. The northeasterly trending thrust fault daylights a short distance to the west of the garage site. The schists are believed to extend at least 40 feet below present grade at the garage location. Foundations are planned to be installed at shallow depths on the firm but weathered rock, using a design pressure of 8 tons per square foot.

Site 2

Mica schist probably underlies Site 2 as well. However, no test borings drilled to date have reached rock, although this site lies less than ¼ mile from Site 1. The regional geologic information indicated that rock was likely shallow, however, test borings drilled to more than 50 feet encountered only glacially derived soils. In general, dense strata of likely both glacio-fluvial and mechanically transported materials were found in the test borings. The very dense nature of even the glacio-fluvial soils below depths of 8 to 10 feet indicate that one or more glacial advances probably overrode the Site 2 area. It is likely that a significant northeasterly trending fault underlies this site. Foundations are planned to be installed at a depth of ± 10 feet below grade with a design bearing pressure of 4 tons per square foot.

STRUCTURAL DESIGN

General

Different structural schemes were evaluated for each facility. They included: 1) Cast-in-place, post-tensioned concrete frame; 2) Structural steel frame with cast-in-place concrete composite deck; and 3) Precast, prestressed concrete frame with cast-in-place composite topping.

These three systems are generally accepted as the most cost-effective structural frame alternatives for the relatively long spans required to achieve "column free" supported parking (clear spans of ± 60 feet).

Site 1

A functional 3-bay plan was established and the structural system selected. The authors' firms collaborated on the foundation design and seismic solution(s). The structure is to be built into a slope of the site, as a result of the owner's preference for a functional plan placing the structure nearer the point of use. To minimize excavation costs, the easterly third (bay) of the building was deleted at grade. This created a continuous retaining/support wall along Column Line "B" which is substantial enough to resist all the north-south lateral forces due to wind or earthquake—provided the forces could be directed toward it.

As a result of the sloping site and the existence of interior ramps, other walls were introduced along Column Lines "A" and "C," further stiffening the structure along the north-south axis. To minimize volumetric forces, an expansion joint was introduced in the east-west direction at or near the centroidal axis. This joint effectively creates two separate structures since the sloping floor (middle bay ramp) is confined to the northern half of the building. Lateral frame analyses were performed for both halves for wind and earthquake forces.

Site 2

The authors once again collaborated on the foundation design and seismic solution(s) after the functional plan (2-bay—122 feet by 456 feet) was established. A precast, prestressed frame with 3-inch cast-in-place composite topping was selected as the structural system.

In order to minimize volumetric forces, an expansion joint was introduced in the east-west direction. As a result of site restrictions, the expansion joint created two separate structures, the northern half (228 feet long, with sloping ramps) and the southern half (168 feet long, flat deck with two percent cross slope for drainage). Frame analyses were performed on both halves for wind and earthquake.

SEISMIC HAZARD ANALYSIS

General

The steps of present-day seismic hazard analyses may be briefly characterized as follows: 1) Reviewing the available regional seismic history; 2) Reviewing the available regional tectonic history; 3) Correlating earthquake activity with geologic structure; 4) Estimating maximum expected event magnitude by extrapolating recurrence curves to desired "design" recurrence intervals, or by estimating the maximum magnitude event from fault length versus magnitude plots; 5) Assigning maximum-sized events and recurrence rates to the appropriate seismic sources; 6) Performing probabilistic analyses for each source and attenuating the various design event ground motions to the site of interest; and 7) Using the maximum design acceleration, time history, or response spectrum in a dynamic structural analysis.

Unfortunately, for low hazard seismic regions, this apparently simple process lacks: A) Enough historical instrumented earthquake data to characterize the various possible sources in the region and the nature of possible source mechanisms; B) The effects of fault directivity and travel path geology; C) Appropriate local attenuation relationships; D) A reliable mathematical model that simulates the effects of site geology upon the variable incoming wave trains; E) An estimate of the effect of structural feedback to the site; and F) A useful estimate of the variability of site and structural damping, as well as the energy absorbing effects of structural ductility.

Despite the lack of data, many of the investigative steps are similar to those previously noted, although a great deal of judgement must replace mathematical and statistical sophistication in low hazard, normal budget projects.

Seismic Activity

Of prime significance in any hazard study is the historical seismicity of the source region(s). In this instance the available information indicates that no historic earthquakes larger than Modified Mercalli Intensity (MMI) VI have occurred within some 100 kilometers of the site. MMI VII events have been noted within the surrounding region, i.e., Warrensburg, NY; Attica, NY; Wilmington, DE; and East Haddam, CT. The closest large events, MMI VIII, have occurred near Massena, NY, and Cape Ann, MA, hundreds of miles away, in a different tectonic environment.

The available earthquake record for the site locale officially starts in 1737 with an MMI VI event centered near Rockaway Beach, NY. Obviously, any large event, such as a MMI VIII event, would have been noted by any of the earlier settlers of the region. For example, the first unambiguous record of a northeastern U.S. earthquake, for which a specific date can be set, is June 11, 1638, for a shock which occurred in the vicinity of Boston, MA (Stevens, 1991).

Probability of Occurrence

In most typical hazard studies a recurrence curve of events that occur in a source area is plotted for a nominal 10,000 square kilometer area. Such a curve for the site area is shown on Figure 1 (New York State Emergency Management Office [NYSEMO], 1986).

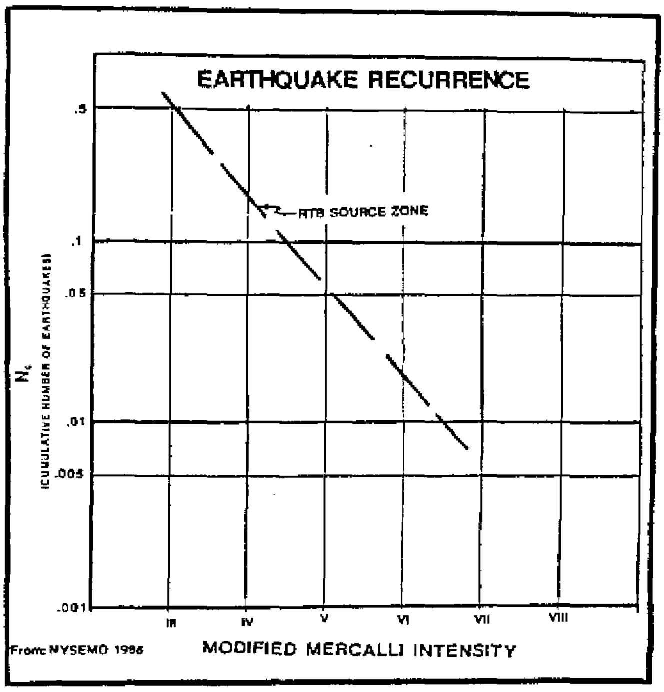

Figure 1

Using these data, an event with a 500-year return period would be about MMI VIII. A 500-year event is used in many building codes. However, in utilizing any such recurrence data, consideration must be given to:

1. The quality of the subsurface conditions of the site of interest.

2. The size of the source area in relation to the expected extent of the highest damage (MMI) area from any particular earthquake.

3. The location of previous events (in the northeastern U.S. there appears to be a tendency for recent earthquakes to cluster in both time and general location).

4. The possibility of tectonic conditions at the site localizing seismic activity.

5. The maximum credible earthquake that can be expected in a source area. For example, the plot of Figure 1 could be extended to MMI XII, the largest event on the MMI scale, but is such an event physically possible?

Thus, it can be seen that even using a statistical evaluation of the historical records, a great deal of judgement enters into any postulation of the possibility and nature of earthquake effects at the site.

Regional and Local Tectonic Conditions

Numerous small earthquakes have occurred in the area surrounding the site, although the only event that caused even slight structural damage even at the epicenter (MMI VI) was the 1874 Irvington, NY, earthquake, some 10 to 15 miles from the site. The northeast trending contact between the Manhattan Schist and the Inwood Marble shown on geological maps of the area is believed to lie just to the west of the rock-supported garage site. However, no detailed geologic mapping has been performed in the immediate vicinity of the two sites. Where definitive work has been completed in the general locale (e.g., Dames & Moore, 1977, and Hough and Seeber, 1991), numerous small northwest trending features have been found. The tectonic conditions at the medical center are believed to be consistent with the nearby areas which have experienced small, likely shallow earthquakes, probably occurring on northwest-trending features, often at intersections with other, frequently northeast-trending, faults.

Maximum Credible Earthquake for the Region

The largest historic earthquake of the Figure 1 source zone is apparently MMI VI. Using these recurrence curves, one can examine the probability of events larger than those that have already occurred, as well as attempting to extrapolate them to the "500-year design event." The minimum time interval in which earthquakes were recorded in the source zone of interest is at least 255 years (1737-1992). If an event of MMI VII or greater were to have occurred, it is likely that in the over 300 years in which European settlers have been in the New York City area an earthquake of this size would have been recorded in the regional histories. Hence, for a larger ($\geq$ MMI VII) event we have over 300 years of record.

Assuming a random occurrence process that estimates the probability of occurrence (or non-occurrence), the chance of seeing an earthquake of MMI VII or larger during the minimum period of record (255 years) is about 93%, while the chance of seeing the same event in the more reasonable >300 years of record is greater than 95%. Thus, if the well-researched catalogue of Nottis, 1983, is used as the official earthquake record of the area, one would not expect to see events larger than MMI VI during any reasonable economic life of the proposed garages.

Hazard Summary and Conclusions

Both the tectonic history of the site locale and the regional pattern of earthquake activity are complex. As a result of the nature of the measured in situ stresses in the region and the previous occurrence of numerous, small, relatively nearby earthquakes, the probability of some earthquake activity affecting both sites is quite high. The question then becomes, what size event can reasonably be expected and what level of conservatism should be used in design at both locations?

It seems quite reasonable to assume that the proposed garages will be subjected to an MMI VI level of motion during their economic lifetimes. Such shocks are fairly common in the source region within which the site is located. In reality, we would expect that no structural damage would be experienced in a well-designed and well-constructed, reinforced concrete structure at this level of motion. Design acceleration levels for a MMI VI event are shown on Table 1.

TABLE 1

Sustained Ground Motion

Maximum Regional Intensity (MMI)	3 Cycle Acceleration (%g)	
	Site 1	Site 2
VI	4	6
VIII	7	10

The selection of the "500-year" design event is far more ambiguous. In general, the 500-year exposure period is consistent with typical building code conservatism in that the probability of such an event occurring during any 50-year period is less than 10%. However, using the available earthquake data base (from 1737 to present), it is possible to show that the 500-year event is either MMI VIII or MMI VI, using the same statistical model. The only difference is the assigned size of several older New York City area earthquakes.

Utilizing a quite conservative evaluation of future seismicity and a realistic evaluation of the effects of subsurface conditions on site motions, the MMI VIII acceleration values shown on Table 1 were selected to evaluate the likely performance of structures designed elastically for MMI VI motion under a presumed MMI VIII design event.

SEISMIC DESIGN

The designers selected the 1990 BOCA, Section 1113, approach to seismic design whereby the potential seismic event is broken into a series of smaller forces applied throughout the building at the load bearing support points. These seismic forces are "assumed to act non-concurrently" in the direction of each main axis of the structure in accordance with; $V = 2.5\ A_v IKCSW$.

The SAP 90 computer program (Computers and Structures, Inc., Berkeley, CA) was used. From the hazard study, the authors selected lateral force coefficients (4% and 6%) of the gravity loads applied at the appropriate load support points. The design loads included future floor seismic forces. The computer program provided all member forces and moments in the columns, piers, and girders. All joint rotations and moments, and support reactions were determined as well. The earthquake forces and moments typically exceeded the wind loading throughout the building.

COMPARISON OF RESULTS

Site 1

From the computer-aided analysis it was determined that the column sizes selected (24 inch by 24 inch typical) would accommodate the magnitude of the shear and moments expected during the 500-year earthquake with only a relatively small increase in the reinforcement required for gravity and wind loads.

Connections were introduced between the columns and floor beams to transfer moments by providing threaded inserts in the precast columns with threaded rods extending into the 3-inch poured-in-place concrete topping over the precast, double-tee floor beams. Column base reactions, shears, and moments were generated by the computer program as well as for the bottom of pier/top of footing. As a result of the relatively high column loads (because of the long spans), the column base design was straightforward.

The preliminary foundation sizes were selected on the basis of gravity loads, with some allowances for wind, using the allowable soil bearing value recommended for the weathered rock strata—8 tons per square foot. The additional pressures created by a seismic event were all within 50% overload, or 12 tons per square foot, which we believe the "rock" could easily accommodate during a short-lived seismic event. Therefore, the foundation footprint sizes were largely unaffected by introducing the seismic loading. However, the footing thicknesses and reinforcing were increased over the gravity load requirements. The ACI 318-89 "Building Code Requirements for Reinforced Concrete" was used to design the foundations as well as other building elements.

The retaining wall/bearing wall designs proved to the most complicated because of the varying conditions expected. Most of the bending moments from wind and/or earthquake forces would be "dragged over" to these walls as a result of their increased stiffness compared to the columns on the "C" and "D" lines. These moments and shear were distributed over the full 27-foot bay to check the wall sections between the column/pier(s). The wall sections at the column/pier(s) were designed to accommodate all of the shear and moment, assuming the forces could only be distributed a limited distance to each side of the columns, per ACI code. The bearing wall designs were checked for the initial building, and with two possible future levels as well.

Site 2—Northern Half

In developing the frame to resist the lateral forces in the east-west direction (short direction), the designer chose to use a one-way brace frame system (which only develops moment capacity for applied loads in one direction). The frame moments at the ends of the floor members must be transferred to the columns, the load path will be from the floor member through its connection to the supporting beam and then transferred along the length of the beam to column. This was achieved with threaded rods and inserts from the columns to the floor beams and dry packing the bottom of the floor beams on each side of the columns.

In developing the frame in the north-south direction (long direction), the designer chose to use the continuous ramp portion of the structure to develop the lateral force. The lateral resistance was achieved by truss action, the floor acting as web members and the columns acting as cord members.

Southern Half

The frame to resist the lateral forces in the east-west direction was the same as the northern half. In developing the frame design in the north-south direction, the designer chose to use shear walls (12 inches thick). Two loaded bearing shear walls (with openings) were introduced along the middle column grid (B line) and the remainder of the walls used to resist the lateral forces were tower walls, thus permitting the structural beams to be designed as simply supported members.

Foundations

The preliminary foundation sizes were selected on the basis of gravity loads, with some allowance for wind, using a soil bearing value of 4 tsf. The additional pressure created by a seismic event increases the foundation foot print approximately 7% to 9% (depending on foundation location), thus increasing the footing thickness and reinforcing. As a result of the nature of the foundation subgrade materials, no increase in bearing capacity was allowed for seismic loading.

CONCLUSIONS AND SUMMARY

As a result of normal budget limitations and lack of information on earthquake effects on local structures, the seismic design of conventional structures in low to moderate hazard areas can be a most difficult and often unrewarding task. Many of the concepts developed for major facilities should be used in defining the level of hazard in an area. However, in applying the concepts and procedures, judgement often replaces data and analysis. Computer-aided structural design is virtually a must, but the designer must understand that earthquake motion is not uni-directional and not a hammer striking one end of the structure.

The inclusion of seismic analyses did generate a significant increase in analysis time. More importantly though, the owner will have paid only a small premium (approximately 2% for the rock-supported structure and 4% to 6% for the more structurally complex, soil supported structure) for earthquake resistant construction.

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