DESIGN LESSONS FROM RECENT DESTRUCTIVE EARTHQUAKES

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1. Introduction

Recent severe earthquakes have selectively damaged or destroyed many large modern buildings. The destructive earthquakes at Caracas in Venezuela, July 1962, near Tokachi-oki in northern Japan, May 1968, and the two earthquakes at Manila in the Philippines, August 1968 and April 1970, have all highlighted important factors which should be considered during the earthquake-resistant design of buildings. Reports on these earthquakes which directly supplement this paper are listed in the references.

Very severe microzone effects were caused by areas of alluvium in Caracas and in Manila. Since tall buildings were selectively attacked microzone effects will be observed more frequently as high-rise buildings proliferate.

Many buildings had their response to earthquakes dominated by low-strength panels. In almost all cases these panels seriously reduced earthquake resistance, and they frequently suffered expensive non-structural damage. Some of the stronger "non-structural" panels completely destroyed adjacent columns while others caused severe diagonal tension cracks in columns.

The earlier Manila earthquake demonstrated that short shear walls may increase considerably the earthquake resistance of frame buildings. Such short shear walls were introduced to strengthen a number of the buildings which required repair after the earlier Manila earthquake, and the value of these added shear walls was confirmed during the later earthquake. There is evidently some advantage in introducing such shear walls during the earthquake-resistant design of many frame buildings.

The buildings damaged and destroyed in Caracas were designed under a code which was based upon the Zone 2 provisions of the Uniform Building Code of California, 1955.

The 2 to 4 storey buildings damaged or destroyed by the Tokachi-oki earthquake were designed according to the existing Japanese code which called for earthquake design loads of 0.18 of gravity. Although the building code for Manila did not require specific attention to earthquake-resistant design for buildings less than 100 feet high, engineers were generally aware of earthquake problems and frequently designed for earthquake resistance. However many building designs appeared to follow the American Concrete Institute provisions for non-seismic areas.

2. Microzone Effects

During destructive earthquakes in Caracas and Manila there have been pronounced microzone effects in the damage inflicted on modern multi-storey buildings. There was no ground damage during the July 1967 earthquake at Caracas and none during the August 1968 and the April 1970 earthquakes at Manila. Local increases of damage were therefore due to local increases in the inertia attack arising from ground acceleration. Despite substantial differences in the character of the two Manila earthquakes, they both delineated the same microzone areas.

Some recent earthquakes have had local areas of severe damage caused by soil rupture or liquefaction. The inertia attack may not have varied much throughout Anchorage during the very severe earthquake of March 1964. However part of the city was destroyed by rupture and slipping of the ground near the edge of the plateau on which it stood. During the June 1964 earthquake at Niigata City there was severe local damage due to liquefaction of local unconsolidated silts. There were also areas of soil liquefaction during the Tokachi-oki earthquake. Some soils in Manila city are similar to those which liquefied during the Niigata and Tokachi-oki earthquakes and it is probable that they also will liquefy if subjected to a very severe earthquake.

Caracas city was uniquely qualified to demonstrate microzone effects in the earthquake attack on tall buildings since it contained over 1000 modern buildings of 10 or more storeys, designed to resist moderately severe earthquakes, all with the same basic form of construction, and situated on a wide range of ground types from areas with 300 feet of alluvium to areas with surface rock. Since the epicentres of the 1967 earthquake was some 45 miles north-north-west of the city all points within it were effectively equidistant from the epicentre.

The main inertia attack at Caracas was to those buildings with 10 to 20 storeys which were situated in an area which contained alluvium to a depth of about 300 feet. Almost identical buildings in areas with a small depth of alluvium and in areas with surface rock suffered little or no damage. In the area of deep alluvium four buildings of 10 to 12 storeys collapsed, a number of 10 to 20 storeys were close to collapse and many of 10 to 20 storeys were severely damaged.

The Caracas earthquake of 1967 appeared to provide all the conditions necessary to give very pronounced local variations in the inertia

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attack. The earthquake was distant (45 miles) and of large magnitude (6.5, Richter) so that rock vibrations would have a moderate intensity and relatively long periods. The city contained an area of deep flexible alluvium with well defined boundaries and also areas with shallow alluvium and with surface rock. The alluvium would have had a severe resonance at a well defined period. The relatively low strains would have limited energy losses and changes of resonant periods. The tall buildings were rendered more vulnerable to attack by panels which usually limited structural damage to first storey columns. The limited region of damage in the buildings reduced the plastic reserve and prevented large changes in the building period. These soil and building characteristics could be expected to give a severe and period-selective attack on the tall buildings.

In some areas of Caracas, where the alluvium was of intermediate depth, there was considerable damage to buildings a few storeys high. However one and two-storey buildings did not suffer significant damage anywhere in the city.

Many of the factors which gave severe microzone effects in Caracas were also present in Manila. The earthquakes were distant (about 125 miles) and of a magnitude of about 7.2 Richter. There was an area with very soft alluvium down to 100 to 150 feet. However large buildings in Manila had a much wider range in types of construction than those in Caracas and some were more than 50 years old. In Manila the very severe damage and the two collapsed buildings were confined to the area of deep alluvium, with the most severe attacks on buildings of 6 to 10 storeys in August 1968 and on buildings of 3 to 10 storeys in April 1970. Despite the similar bearings and distances to the epicentres of the two earthquakes there were considerable differences in the character of the attack, the second being quite direct and of much shorter periods than the first. However during both earthquakes the very severe damage was confined to multi-storey buildings on the deep alluvium, while there was moderate damage in areas with alluvium to an intermediate depth and no damage in areas with surface rock.

It is interesting to note that if the buildings in the cities of Caracas and Manila had been limited to no more than 2 storeys and to earlier forms of construction there would have been little damage, probably no casualties, and no evidence of microzones, during these three earthquakes. Both cities had considerable shanty-town areas of very inferior construction, those in Caracas were built with hollow bricks and those in Manila were mainly of wood. None of these shanty-town buildings suffered any damage.

Microzone inertia effects are very pronounced during moderate earthquakes but can be expected to be less pronounced during very severe earthquakes, when plastic deformation of soil layers would be limited. If buildings are designed for these very severe earthquakes then during moderate earthquakes only non-structural damage should occur. If buildings are designed for less severe earthquakes or less than the maximum design effect, then during moderate earthquakes they could collapse during the moderate earthquake in which severe microzone effects may occur.
very extensive these buildings had a consider­
able reserve of resistance after the earthquake.

Many frame buildings in Manila had two
further factors which reduced the number of
columns suffering plastic deformation. Many
had deep exterior beams which stiffened exter­
ior columns and hence increased their share of
lateral loads. Furthermore, a number of build­
ings, including the collapsed Ruby Towers and the
almost collapsed Philippine Bar Association
building, had effective shear walls across one
end further restricting the number of columns
which deformed plastically.

Where columns had a large width-to-height
ratio, often because the effective height was
reduced by strong panels, the initial column
damage was by diagonal tension cracking with
low plasticity rather than by bending, which
normally exhibits much higher plasticity.
Severe cases of diagonal tension failure
occurred during the May 1968 Tokachi-oki earth­
quake in northern Japan. Column shear failures
were very severe in a number of 2 to 4 storey
reinforced concrete school buildings. These
buildings were less than 10 years old and were
designed to resist a base shear of about 0.18
of gravity. The columns were considerably
shortened by concrete panels and the primary
failure was by diagonal tension cracks in the
first storey columns. The low plastic reserve
of this failure mechanism resulted in very
severe damage. It is understood that Japanese
experts now consider that design against dia­
gonal tension failure should be based on twice
the current shear coefficients. Diagonal
tension must be resisted by adequate transverse
ties and stirrups. These ties must be secure
against unwrapping under damage conditions and
the spacing must be much less than the smaller
column or beam dimension.

When a column or beam develops plastic
hinges under end moments a high plasticity
factor is dependent on effective containment
of the concrete region. The need for this containment has been established
by many laboratory tests and has been demon­
strated frequently during recent earthquakes.
Concrete containment requires adequate close­
spaced ties with end anchorages which are
effective even after the exterior concrete is
spalled off and the interior concrete is
cracked. Close-spaced spiral ties are partic­
ularly effective as transverse reinforcing
for square or circular columns.

Inadequate ties have been an important
factor in the observed failures of a large
number of columns. Cases of damage under forces
which were predominantly bending or shearing or
crushing have been observed and also cases with
combinations of these forces. Instances of tie
steel failure in tension have been very rare.
In almost every case the ties have unwrapped.
A common error is excessive spacing between
ties. There have been a number of instances of
very irregularly placed ties with some spaces
more than twice the design value. The effect­
iveness of properly placed spiral reinforcing
was demonstrated in the case of some greatly
overloaded columns in the P.B.A. building,
Manila, Fig. 1.

The earthquake resistance of many buildings
was greatly changed by unreinforced panels of
ceramic or concrete hollow blocks. In Caracas
the panels were of very low strength bricks
which usually contained about 80% voids. These
panels were completely neglected during the
design of the reinforced concrete frames for
seismic loads. These panels showed surprising
strength, particularly when they had small or
no openings and were framed by beams and
columns. In the first storey there were usually
few panels and these were shattered in severely
attacked buildings. Panels in the second and
third storeys usually suffered some damage,
but insufficient to allow plastic hinges to
form in columns or beams. In general the panels
had insufficient strength to cause direct damage
to frame members. However their stiffening
action greatly favoured column damage at a
single storey level (usually storey one) rather
than beam damage over several storeys, since
the latter requires the simultaneous shatter­
ing of a much larger number of panels. Only for
buildings with much higher resistance in the
columns than in the beams, such as the 18-storey
Residencia Union and the 19-storey Blue Palance,
were beam hinges formed over several lower
storeys. These beam hinges were necessarily
associated with severe panel damage over all
the lower storeys of the two buildings.

In designing a reinforced - concrete frame
building an attempt may be made to ensure that
beam hinges, rather than column hinges, form
during lateral deformations. This damage
mechanism has the advantage that beam hinges
involve low forces in several storeys, although a greater
absorption of energy and the further advantage
that beams are free of axial loads which accel­
erate the collapse of a hinge. However even
incomplete low-strength panels may give so much
resistance to the formation of beam hinges that
the mechanism is changed to the formation of
column hinges at a single storey level. Some­
what strong panels may reduce the effective
lengths of some columns and lead to their fail­
ure by diagonal tension cracking, a mechanism
with a very low plastic reserve.

In the city of Manila the panels in rein­
forced concrete buildings were usually of
concrete hollow blocks, although a few were
ceramic hollow blocks containing about 50%
voids. While some panels were complete, in
most cases they either contained window open­
ings or they extended only part way up the
columns. When these buildings swayed under
earthquake forces there was severe interaction
between the rigid panels and the relatively
flexible frames. During the 1968 earthquake
direct interaction usually resulted in panel
damage. During the apparently shorter period
vibrations of the 1970 earthquake however,
panel forces often caused direct damage to
columns and in some cases caused their total
failure. Such column failures, which result
directly from panel forces, are particularly
liable to result in total collapse of a build­
ing. Initial failure is usually by diagonal
tension cracks in a limited number of columns at
a single storey level, Fig. 2. Since such shear
failures absorb little energy very building
vibrations continue at a high level until the
shear failures may progress to complete
column failure under vertical loads and thence
to the collapse of the building.

As the height of a building is increased the
column strength is increased and there is
therefore less likelihood of direct damage by a given type of panel. Direct attack on panels is therefore more severe for lower buildings. Such lower buildings were attacked more severely during the 1970 earthquake and were associated with more severe column damage. While direct column damage by concrete hollow-block panels is characteristic of lower buildings an increase in panel strength would extend this form of attack to stronger columns and hence to taller buildings.

Panels which extend part way up columns are most likely to cause direct column damage. However even complete panels can cause shear failure in columns, after first separating along their top edge and then failing at their upper corners. The panel then supports the column load of the height and column failure in diagonal tension may occur more. During the 1968 earthquake columns at the upper levels of the 11-storey Diamond Tower suffered moderate damage by concrete hollow block panels, Fig. 3. During the 1970 earthquake the 6-storey United Building had several columns severely damaged by panel forces, Figs. 4 and 5. As a result the building was close to total collapse.

5. Shear Walls in Manila Buildings

No attempt was made to increase the lateral resistance of the reinforced concrete frame buildings in Manila city by the inclusion of shear walls. However fire walls were provided where ever a building shared a boundary with another building and these were constructed as substantial reinforced-concrete structural walls. Many buildings had such structural walls on one or two sides. When a shear wall was located at one end of a long building it probably increased the severity of the transverse attack on the panels and the frame at the far end of the building. In the 6-storey Ruby Towers which collapsed in 1968 and the 6-storey PBA building which almost collapsed were both long buildings with a shear wall across one end. Those long buildings which had a shear wall on one long side but not the other were effectively protected against longitudinal forces but often sustained damage from transverse forces.

A short (in plan view) longitudinal shear wall in the Trinity building increased its earthquake resistance by a method which holds great promise for frame buildings. The Trinity building is 7 storeys high with 6 bays by 4 bays. There is a transverse shear wall at one end. At the far end of one long side is a short longitudinal shear wall occupying only the 6th bay. The building frame had very deep box beams along three exterior walls and flexible interior columns. In the absence of the short shear wall it is probable that the first storey exterior columns would have suffered severe diagonal tension cracks on the long sides and across the end remote from the transverse shear wall. However the short longitudinal wall limited the column damage and distributed it over storeys 2 to 5. Although the short shear wall made an important contribution to the earthquake resistance of the Trinity building it was evidently introduced for architectural rather than structural reasons.

6. Repair and Strengthening in Manila

After the 1968 earthquake, and before the 1970 earthquake, a number of buildings had been repaired and short shear walls had been added to increase their earthquake resistance. These procedures proved generally successful in resisting the 1970 earthquake and should prove very effective if adopted at the initial design stage.

For structural repair small cracks were filled with a pressure grout of epoxy resin and large open cracks were carefully cleared of loose concrete and filled with hardpack concrete. Where damage was severe an additional steel cage was provided. These repairs were evidently successful in a number of members examined after the 1970 earthquake.

Near the ends of the long Diamond Tower building, Fig. 6, and of the long section of the Pearl Tower building short transverse shear walls were provided to complement the longitudinal fire walls. Transverse shear walls were also introduced in single bays of the more nearly square Pearl Tower block. The Gocheco building is in the form of a large hollow square and contains no fire walls. Five single-bay shear walls were added, and also 2 U-shaped shear walls associated with stairways and lift shafts. In introducing the single-bay shear walls great pains were taken in an attempt to ensure that they were integral with the building frame and floor diaphragms. This integral action was certainly achieved during the moderately severe 1970 earthquake.

Our method of introducing shear walls was to remove all the non-structural panels in a single bay at a time and then chipped off the beams and columns to expose the main longitudinal steel on all sides of each opening. These openings were then filled with substantial reinforced concrete panels which contained vertical, horizontal, and diagonal steel. A substantial fraction of this panel steel was welded to the exposed main steel in the surrounding columns and beams. An attempt was made to give these shear walls a substantial footing but this was difficult to achieve in the existing buildings.

A second method adopted to introduce short shear walls was to cut out the beams as well as the panels between a pair of adjoining columns. In this case some of the shear-wall steel was welded to exposed floor-slab steel as well as some being welded to exposed column steel.

During the 1970 earthquake the Pearl Tower and the Diamond Tower suffered no significant structural damage and little non-structural damage. The Gocheco suffered moderate structural and non-structural damage and this was distributed over a substantial part of its height. The moderate damage and the large margin of safety against collapse were due in large measure to the strengthening shear walls.

7. Increasing the Earthquake Resistance of Frame Buildings

Short shear walls provide an attractive method of limiting non-structural damage and of providing a large resistance against collapse in reinforced concrete frame buildings.

While it would be possible to take all lateral loads and severely limit deformations by a set of substantial shear walls such a
solution would be considerably more expensive than a simple frame building and would often be functionally unacceptable. Another possible solution is to make a frame which is highly resistant with members proportioned and steel detailed to ensure that hinges with a large plasticity factor form near beam-ends. Panels would be flexible or separated from the frame. This solution tends to be expensive, it applies considerable constraints to the proportioning of the structural frame, and may still only utilize the energy absorption of plastic hinges at a few levels of a tall building before the onset of collapse.

Short shear walls provide an attractive alternative to the above methods of increasing earthquake resistance. The primary function of the short shear wall is to ensure that the interfloor deflections at all storeys are of similar magnitude. Since these short shear walls need occupy only a moderate number of single bays they have little effect on the utilization of the structure. Panels should where possible be detailed so that a considerable interstorey deflection can occur before they are severely damaged. Since the short shear walls ensure that panels at all levels act simultaneously it requires a considerably more severe, and hence much less frequent, earthquake to cause severe panel damage. As a further protection against panel damage the short shear walls can be provided with a moment resistant base to further limit interstorey deflections particularly at the lower levels where they tend to be largest.

To provide high resistance against collapse beams and columns should be proportioned and reinforced to ensure that large deformations occur as plastic hinges, preferably in beams, rather than as diagonal tension cracks. The short shear walls ensure the simultaneous resistance of a very large number of plastic hinges and give the building a large plastic reserve. To ensure the largest possible plastic reserve it is essential to avoid a diagonal tension failure near the base of the short shear walls. The short walls should finally fail with rotation rather than translation of the base. The loss of plastic reserve which is suffered when a short wall fails in diagonal tension was demonstrated when a five-storey frame building was tested to destruction at the Building Research Institute in Tokyo.

If a design procedure accepts panel damage during very severe earthquakes steps should be taken to avoid casualties from falling masonry. A high-strength canopy should extend over the main exits and steps should be taken to prevent masonry from falling in stairways.

8. References


FIGURE 1

Column D4, P.B.A. Building. A spirally reinforced column under a severe overload.

Photo V-3 Column D4, P.B.A. Building; compare photograph 11, 1968 report

FIGURE 2

Column shear damage from an unreinforced concrete hollow block panel to 2/3 of the column height.
Figure 3
Diagonal tension cracks resulting from a damaged panel in Diamond Tower.

Figure 4
Diagonal tension cracks from a damaged panel in the United Building.

Photo V-32 Diagonal tension cracks in second-storey exterior column, G7, of United Building
Figure 5

Composite drawing based on unrelated components showing assumed mechanism of column damage in the United Building.
DIAMOND TOWER APARTMENTS

Cantilever Storey 3 to 10

Additional R.C. Sheet Panels

Elevator

Typical Lower Column 28'x 28"
Typical Lower Beam 12"x 28"

Figure 6

Diamond Tower showing the two bays with added R.C. panels.