

THE BEHAVIOUR, OVERVIEW OF DAMAGE AND RETROFIT OF STEEL BUILDINGS AFTER THE EARTHQUAKES OF SEPTEMBER 1985

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SYNOPSIS

A brief introduction on the earthquake history of Mexico is made.

A description of the various types of steel structures built in Mexico City is made, including comparisons of the other types of steel construction with more modern practices.

Performance of steel buildings in the September 1985 earthquake are discussed and related to the local geotechnical conditions, including foundation behaviour.

The evolution of seismic design Codes in Mexico City is presented and the Emergency Provisions recently issued, are discussed.

Finally, some ideas of repairing damaged steel structures to improve their seismic performance meeting the higher demands of the reformed Code, are given as a retrofit.

INTRODUCTION

The earthquake which struck Mexico City on September 19, 1985, affected some steel buildings causing them severe damage and/or collapse. Others suffered only minor damage.

The Ms=8.1 earthquake on September 19 and the Ms=7.5 earthquake 36 hours later, targeted Mexico City with its population of 18 million plus and an inventory of approxi-

mately 1 million buildings. Although Mexico City is over 400 km. from the earthquake's epicenter, as shown in Figure 1, its unique soil conditions, including deep saturated lake bed deposits, created a condition where the surface ground motions exceeded those in the epicentral region. (Reference 1).

Estimates of damage and casualty totals for Mexico City resulting from the earthquake vary widely but it is generally agreed that they are in excess of 8,000 lives

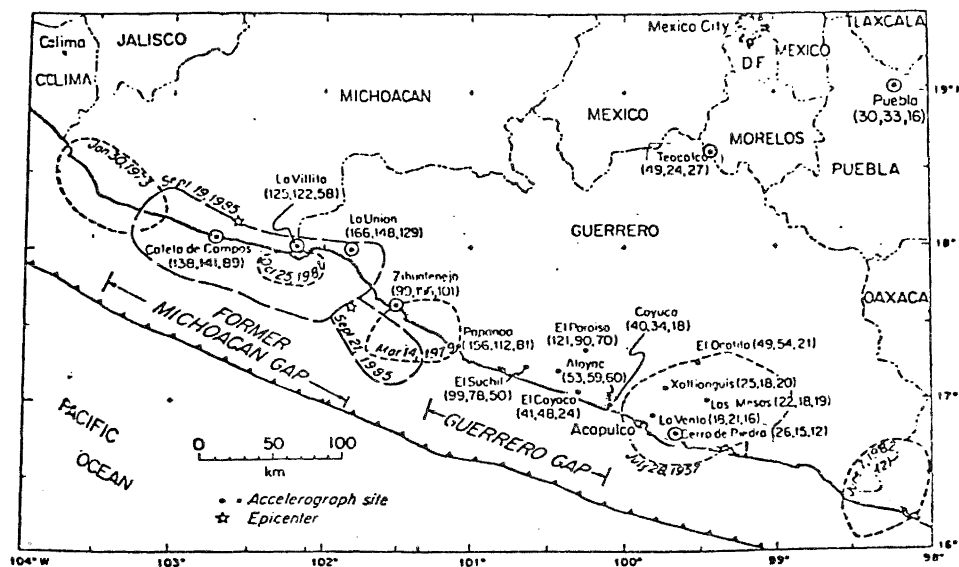


FIG. 1 - LOCATION, DATE AND ACCELERATIONS (...) OF NOTABLE EARTHQUAKES IN MEXICO

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lost, and more than 40,000 left homeless. The total number of collapsed buildings (estimated at 400) and buildings requiring demolition will probably exceed 1,000.

Epicentral area peak ground accelerations of the September 19 earthquake were measured at 0.15 g while the peak accelerations at Mexico City were measured at between 0.01 g and 0.19 g, depending upon the site ground conditions. Ground motions in the damage area were relatively long period motions having approximately a 2 second period. At least twenty strong large cycles were recorded near the damage area during the strong motion which lasted approximately one minute. These long period motions, resulting from the unique subsoil conditions in the lake bed zone of Mexico City, created an environment particularly damaging to mid-rise buildings in the 6-15 story range. Buildings in this range typically have natural periods that approximated the motions of the earthquake and the underlying soil.

SEISMIC HISTORY OF MEXICO

The Western coast of Mexico is bordered by a seismic zone that runs 1,500 miles along the coast of Central America from Panama to the Central coast of Mexico west of Puerto Vallarta.

The subduction fault zone which was responsible for the September 19, 1985 earthquake results from the "Cocos Plate" moving under the "North American Plate" at an approximate rate of 7 to 8 cm a year of relative movement. This seismic zone has been the site of several significant earthquakes in the Twentieth Century resulting in a frequent return period for significant earthquakes affecting Mexico City. The location, date and registered accelerations of several significant earthquakes appear in Figure 1.

Of the earthquakes listed, the events in 1911 and 1941 hit Mexico City with intensities approaching the level of the September 1985. Since strong ground motion instrumentation was not installed until after 1959, it is difficult to make comparisons. Comparisons of damage with that caused by past earthquakes is also difficult to assess because of the relatively smaller population of Mexico City during past earthquakes and because of the differences in building construction. However, the event that was felt the strongest in Mexico City was the 1911 ($M_s=7.9$) earthquake. The 1932 earthquake had $M_s=8.2$ (the highest since 1858) but the quake originated on the Jalisco coast. Due to the large distance from the epicenter and the aforementioned population and building status in Mexico City, the damage was not spectacular. Similarly, the 1957 quake, which was centered 270 km south of Mexico City, produced less damage than the 1985 incident. Finally, damage of aqueducts as a result of the 1941 earthquake is used to demonstrate it was less intense than the 1985 earthquake.

GEOLOGY AND SUBSOIL CONDITIONS OF MEXICO CITY

Mexico City is located at the geographical center of Mexico in a valley at an elevation of 2,100 m. The City lies in a broad basin formed about 30-million years ago by the faulting of an uplifted plateau. The basin was closed by later volcanic activity which resulted in the formation of numerous lakes. It was upon one of these lakes, Lake Texcoco, that the Aztecs built Tenochtitlan, their capital city, in the 14th Century. Expansion of Mexico City by the Spaniards led to the gradual draining of the lake beginning with the completion of the Nochistongo cut in 1789. Today an extensive drainage system serves the valley.

The fact that Mexico City is constructed on an old lake bed has led to significant correlations between the geographical distribution of damage from the September 1985 earthquake and the type of subsoil underlying the damage area. That part of the city that rests on the old lake, which consists of very deep deposits of saturated soft clay, is particularly vulnerable to earthquakes. These lake deposits produce different intensities of ground movement at sites where the stratigraphy and mechanical properties of the soil are different. The seismic shaking in this area of soft clay subsoil is modified from what it would have been on firmer soil.

Mexico City has been classified into three subsoil zones: the "hill zone" of firm soil; the "lake zone" of deep deposits of very soft clay, described above; and, between these, a "transition zone" where clay deposits are shallow. (Figure 2).

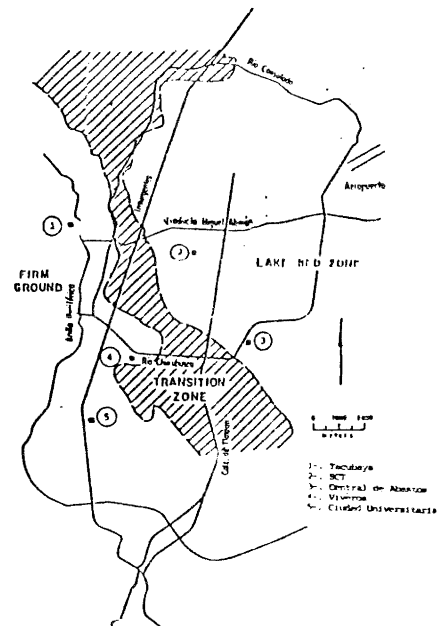


FIG. 2 - ZONIFICATION OF MEXICO CITY AND LOCATION OF ACCELEROGRAPHS

Further details regarding the soil characteristics in the lake zone help to explain why the soil amplification, and hence the earthquake damage, in this area

was greater than in the other two zones. The old lake bed zone contains lacustrine deposits of volcanic sediments of different thicknesses. Decomposed rocks from the surrounding hills, residual clays, pyroclastic materials, gravels and sands have all accumulated in the basin and formed a series of deposits of gravel, sand and silty clay of several hundred feet thick. Following a thin archaeological fill, the deposits of soft silty clay are found to depths of 30 m or more before the first "hard" layer, consisting mostly of sand, is met. This layer is from 1.5 to 3.0 m in depth and is underlaid by a slightly stiffer clay down to between 40 and 50 m. The highest rate of damage in the September 1985 earthquake occurred in the lake zone in Regions where many susceptible buildings existed.

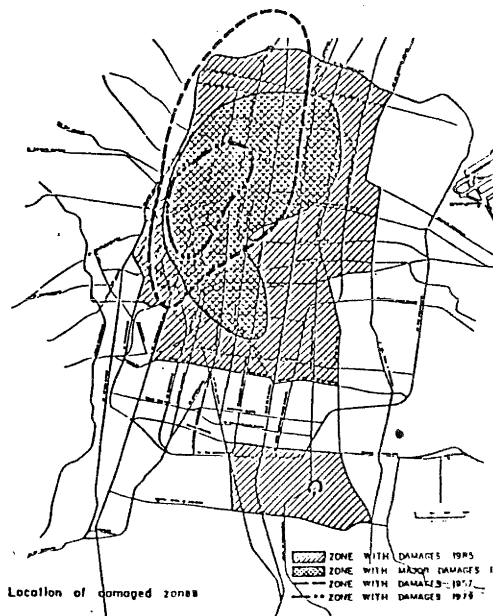


FIG. 3 - DAMAGED ZONE LOCATIONS

SEISMOLOGICAL ASPECTS

The main earthquake shock occurred at 7.18 a.m. Mexico City time with a Richter magnitude of 9.8 (Ms). The epicenter is located near the Pacific Coast at a focal depth of about 33 km; approximately 400 km WSW of Mexico City. (Figure 1). The main shock, which was essentially two subevents 26 seconds apart, was followed by several aftershocks, the most significant occurring 36 hours later with a magnitude of 7.5 Ms. The earthquake was caused by the reactivation of the Michoacan Gap which underlies the Port of Lazaro Cardenas, the point at which the event was focussed. The Guerrero accelerometer array produced 17 strong motion records near the epicentral area. Measured peak accelerations were 0.15 g and the duration of shaking with accelerations of not less than 0.1 g exceeded 10 seconds.

In Mexico City five stations, shown

on Figure 2, measured ground motions which varied considerably between stations. The stations at Tacubaya (T), Ciudad Universitaria (CU) and Viveros (VIV) are located on firm ground and recorded peak ground accelerations from 0.01 g to 0.04 g. As might be expected, little damage occurred, in the firm ground zone.

The stations in the lake zone, SCT and Central de Abastos (CA), recorded considerably more severe motions. For example, the S60E acceleration record at SCT is shown in Figure 4 along with the corresponding spectra. At this station the peak acceleration was 0.20 g, roughly four times that at the CU station. The motions at SCT were particularly damaging because of their increasing amplitude 2 second period motion. The severity is shown in the spectra which shows a spectral acceleration in excess of 1.0 g at two second period. In their recent paper, Rosenbluth and Meli (Reference 2) said of this motion: "There is no other recorded ground motion in history having a spectral acceleration as high as 1.17 g at a 2 second period and a 5 percent damping ..."

In order to better understand the amplification of the accelerations produced by the different ground conditions, Figure 5 plots vertically the ratio of the maximum acceleration obtained from the SCT response spectra divided by the correspondent maximum accelerations obtained from the CU response spectra (this is soft ground accelerations versus firm ground accelerations), and calls it "amplification factor". On the horizontal axis are the correspondent natural periods in seconds.

It is easy to observe that for a 2-second period, amplification factors up to 8.0 are obtained. That means that accelerations up to eight times stronger were transmitted to structures at the soft ground, compared as those experienced at the firm ground.

Former experiences (Reference 3) had detected amplification factors between 4 and 6 only and periods of 2.35 and 2.55 sec, the difference being regarded as the peculiar characteristics of this recent earthquake. Perhaps the two-main slips of the tectonic plates at a very close time interval transmitted the seismic waves at about 3.3 km/sec from the epicentral area to the Mexico City area, arriving with the same time difference in which were produced (26 sec). Thus, once the first seismic waves reached the Mexico City area, travelled at about 0.8 km/sec. speed in the clayed bed, and received a second set of waves of the same intensity, causing unpredicted disturbances not thoroughly understood yet.

The higher amplification factor observed at severe damaged area was undoubtedly due to a resonance effect of the ground's natural period at that location, with the excitation waves. For instance, if we take the expression given in references 4 and 5, to calculate the natural period of the clay layer $T=4D/S$, where:

T is the fundamental resonance period of

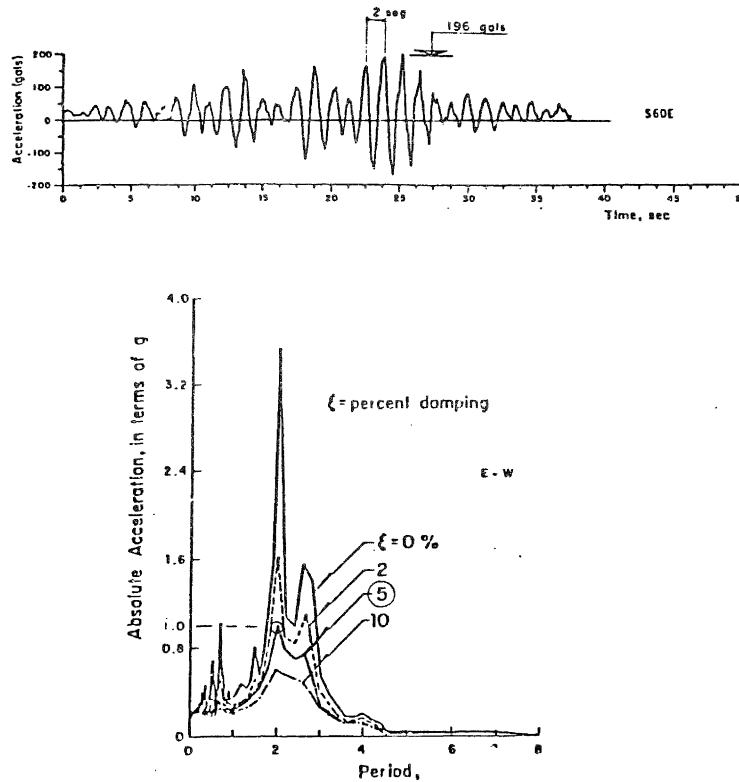


FIG. 4 - ACCELEROGRAMS AND RESPONSE SPECTRA AT SCT LOCATION

OVERVIEW OF DAMAGE

Amplification of Acceleration from SCT / C.U.

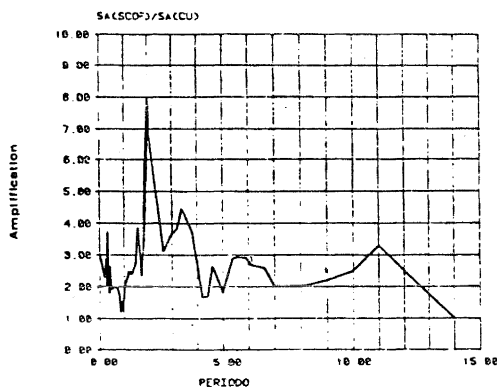


FIG. 5 - AMPLIFICATION CURVE

Prior to focussing the damage to steel buildings in particular, it is proper to understand first the magnitude of the earthquake damage in general.

Mexico City has a population of approximately 18 million people and the inventory of buildings is estimated to exceed 1 million. The city is a mixture of old colonial buildings dating back several hundred years and modern steel skyscrapers such as the 54 story Pemex Building and the famous 43 story Latino Americana Tower.

Damage from the 1985 earthquake was concentrated in the lake bed zone as shown in Figure 3. Areas of damage from the 1957 and 1978 earthquakes are also shown. In each of these previous earthquakes many buildings in the downtown area of the city were damaged or destroyed. Several of these previously damaged buildings which were not strengthened either adequately or sufficiently, did not survive the most recent earthquake.

the stratum.
 D is the stratum thickness in metres, and S is the average shear wave velocity in m/sec.
 Then we have for the Mexico City clay soil, $S=80$ m/sec, and for the SCT location, $D=40$ m. Thus, for this particular location $T=4 \times 40 / 80 = 2$ sec, which was in fact the same rythmn at which seismic waves travelled in the clayed area.

Table 1 presents a damage summary for various types of construction. This data was prepared by the Institute of Engineering of the National Autonomous University of Mexico (UNAM). Their objective in preparing the evaluation was to identify the most relevant structures associated with seismic damage. As such, the data does not represent a total of all severely damaged or collapsed buildings and may differ from other damage

| TYPE OF STRUCTURE | EXTENT OF DAMAGE | YEAR WHEN BUILT | | | NUMBER OF STORIES | | | | TOTAL |
|-----------------------|---------------------|-----------------|-------|-------|-------------------|------|-------|----|-------|
| | | <1957 | 57-76 | >1976 | <5 | 6-10 | 11-15 | >6 | |
| Steel frame | collapse | 7 | 3 | 0 | 4 | 3 | 1 | 2 | 10 |
| | severe | 1 | 1 | 0 | 0 | 0 | 2 | 0 | 0 |
| R C frame | collapse | 27 | 51 | 4 | 27 | 46 | 8 | 1 | 82 |
| | severe | 16 | 23 | 6 | 10 | 28 | 6 | 1 | 45 |
| Waffle Slab | collapse | 8 | 62 | 21 | 36 | 49 | 5 | 1 | 91 |
| | severe | 4 | 22 | 18 | 5 | 26 | 12 | 1 | 44 |
| Masonry bearing walls | collapse | 6 | 5 | 2 | 11 | 2 | 0 | 0 | 13 |
| | severe | 9 | 13 | 1 | 22 | 1 | 0 | 0 | 23 |
| Other | collapse | 4 | 8 | 2 | 12 | 2 | 0 | 0 | 14 |
| | severe | 0 | 4 | 2 | 2 | 4 | 0 | 0 | 6 |
| Total | collapse and severe | 82 | 192 | 56 | 129 | 161 | 34 | 6 | 330 |

TABLE 1 - STATISTICAL SUMMARY OF DAMAGE TO BUILDINGS

estimates. The table summarizes building damage as a function of type of construction, year of construction and height. The breakdown in year of construction coincides with major revisions to the building code.

Most of the buildings in the survey fall in the 7-11 story height range and were built prior to 1976. It is interesting to note, however, that the majority of concrete and masonry buildings were built between 1957 and 1976, while most of the steel buildings were built prior to 1957.

Several classic causes of damage have been identified. Causes, along with the percentage of occurrence, were identified by UNAM personnel as listed in Table 2: In many cases more than one failure characteristic was observed, thus the total will exceed 100 percent.

| Characteristic Observed | Percent of Cases |
|---------------------------------------|------------------|
| Pronounced asymmetry in stiffness | 15% |
| Corner building | 42 |
| Weak first story | 8 |
| Short columns | 3 |
| Excessive mass | 9 |
| Previous differential settlement | 2 |
| Unsatisfactory foundation performance | 13 |
| Pounding | 15 |
| Previous earthquake damage | 5 |
| Punching in waffle slabs | 4 |
| Upper story failure | 38 |
| Intermediate story failure | 40 |

TABLE 2 - TYPIFICATION OF FAILURE MODES

The large number of damaged and collapsed buildings six stories and less with fundamental periods of less than one second) is explained as follows. Under moderate excitation the brittle concrete or masonry infill systems crack. This initial damaged state make the system more flexible (longer periods) resulting in greater response, etc. As the cracking proceeds under the high acceleration-long duration earthquakes, the structure is driven into resonance with the earthquake, resulting in substantially higher lateral forces which, in turn, resulted in the severe damage or collapse noted.

PERFORMANCE OF STEEL BUILDINGS

Steel buildings in Mexico City date back over sixty years. As a construction system in Mexico steel is regarded to be, at least in its basic prices, more expensive than concrete, but it is surprising to find so many structures steel framed. Many of the old colonial landmarks were constructed with a steel frame surrounded by unreinforced concrete or masonry. Examples include among others, the Palace of Fine Arts, the Monument of the Revolution and the Department of Federal District headquarters, which survived the earthquake undamaged.

Steel construction began in the late 20's when the main railroad station, the Museum of Natural History, banks, department stores and hotels were constructed using imported rolled shapes in riveted simple-framed construction. Lateral resistance was provided either by masonry infill or by steel knee-brace frames. In the next two decades, many steel buildings were built in similar types of construction. Pipe and boxed channel columns are found in construction of this time period. The same can be said for rails, which in some instances were used as secondary floor beams in vault-types of flooring.

The multistory construction formally started in the 1940's with the National Lottery (28-story), La Mariscal (25-story)

and the Roble buildings (24-story), and continued into the 1950's the Latino Americana Tower (43-story), the Seguros Anahuac (27-story), the Seguros La Comercial buildings (25 and 28-story), the Banco Internacional building (32-story) and others. Some of these buildings were finished in the beginning of the 1960's. The common type of construction was moment-resisting framing, with imported wide-flanged columns and beams or three-welded-plate sections. Column construction usually consisted of two channels with riveted cover plates. Most joints were field riveted.

For the last three decades the importation of structural steel has been practically nil. Limitations in the Mexican steel mills to produce rolled sections up to 45.6 cm in depth (18") has required the fabrication of columns with horizontal members fabricated from open or solid webs.

In order to optimize cost, some designers have taken advantage of relatively inexpensive local labour and frequently design lighter structures using floor truss-girders and truss beams.

Most modern steel buildings in Mexico City have been constructed with frames in which all joints are moment frames using welded box columns and open floor framing in composite action with the concrete deck (either plain or on steel deck). These highly redundant structures appeared to have produced global ductilities greater than might be predicted from a single rigid joint. Some others, like the Foreign Relations Headquarters (21-story), incorporated concrete shear walls to work in composite construction with some slender steel columns in a tube-like concept.

Undoubtedly the most famous steel structure in Mexico City is the 43-story Latino Americana Tower designed by Dr. Leonardo Zeevaert with consultation provided by Dr. Nathan Newmark. Built in the early 1950's, this structure has withstood the 1957, 1978, 1979 and 1985 earthquakes with no structural damage. The 1985 earthquake produced nonstructural damage estimated at about \$5,000 U.S. Dollars while many buildings in the general vicinity collapsed. A visual and x-ray examination of the structural frame by the building engineers revealed no structural damage.

The Latino Americana Tower is constructed from rolled structural shapes. Lateral resistance is provided by riveted moment connections and composite floor beam. As R.W. Binder noted in his paper at the 1957 AISC Annual Convention (Reference 6):

"Torre Latino Americana is a structure which will endure because of its strength, resilience, toughness and the energy absorbing qualities of the ductile material of its 43 story frame; brittle material or materials with limited ductility used for architectural purposes are so placed and attached as not to be affected by the necessary deflections of the structure."

That prediction was made after the tower survived the 1957 earthquake and now, three significant earthquakes and almost forty years later, the prediction holds true - a testimony to the foresight of its creative designers and to the toughness of its steel frame.

With the exception of Pino Suarez, which will be discussed in detail later, the performance of steel structures built in the last 30 years was exemplary, especially those buildings with a period near the two second critical range which were subjected to forces so far in excess of those contemplated in its design. As an example, the new National Lottery Building, a triangular building with built-up box columns and truss girders, suffered no damage in spite of its location, shape and its soft first floor. The building was exposed to significant ground motion as witnessed by the severe cracking and settlement of the sidewalk around the building.

Dozens of modern steel buildings have been constructed in the lake bed area and received no damage. Examples of two such buildings include the 27-story Seguros Anahuac and the 19-story Banco Mexicano.

Unfortunately, not all steel buildings survived the earthquake as proudly as the Latino Americana Tower and the others before mentioned did. In their survey of damaged buildings, UNAM (Reference 7) identified 330 collapsed or severely damaged buildings, of which 12 were of steel. A review of the UNAM data indicates that all of the steel buildings except those four at Pino Suarez complex were built in the 1940's.

Two of the older steel buildings identified in the UNAM survey were the Edificio Atlas and the unnamed structure located between Edificio Atlas and Latino Americana Tower. The unnamed structure was constructed with steel columns fabricated from two channels with riveted cover plates and what appears to be rolled floor beams. This type of structure, typical of many structures built during the same time period, received its limited lateral resistance from unreinforced masonry. As it will be pointed out in the discussion on building codes, buildings of this period were not required to be designed against lateral resistance.

The number of damaged buildings in Mexico City was significant and therefore provides an excellent opportunity to extend our knowledge of the behaviour of buildings as they respond to earthquakes. Though the number of buildings damaged was significant, the percentage was very small considering the intensity of the ground motion as compared with the code design levels. The following description will concentrate on damage to four buildings which might be taken as representative of the types of construction in steel (multistory).

These buildings have been selected because they can provide valuable lessons, some of which are yet to be clearly demonstrated.

The Conjunto Pino Suarez building

complex had three identical 23-story buildings and two identical 15-story buildings founded on a common deep stiff foundation mat (a subway station). One of the 23-story buildings collapsed upon a 15-story building, a second 23-story building was approaching collapse and a third 23-story building was badly damaged. The second building, Edificio Atlas, was damaged in the 1957 earthquake, repaired and strengthened, and had a partial collapse in 1985 and is now undergoing demolition. This building can be considered representative of similar buildings built in the 1940's in USA Midwest cities such as St. Louis and Memphis. The third and fourth buildings called the Amsterdam building and the Azucar building, consisted of slender moment frame construction which suffered moderate damage. Some repair suggestions are given at the end of this paper.

CONJUNTO PINO SUAREZ

The Conjunto Pino Suarez was a five-building complex built in the early 1970's. The buildings were built over a subway at an intersection of subway lines. In fact some of the building columns extend through the subway station. The beams were trussed members built with steel angles and angles welded together to make tube sections. The columns were hollow box sections made of four plates fillet welded at the edges. These members had internal stiffeners which were welded to three sides on the interior length and to all four sides at the members end. The buildings were two bays wide (12 metres) and 4 bays long (28 metres).

The trussed beam to column connections were moment resisting connections. The longitudinal direction had one line of V bracing in the exterior wall in the bay of the stairway, elevators, and toilet rooms, and two lines of X bracing, one on either side of the elevators stairway bay.

The most southerly of the three 23-story towers collapsed into the adjacent 15-story tower causing it to collapse.

Because many theories have been suggested for the collapse, its cause is still conjecture. However, it has been reported that the top two floors, and the seventeenth floor, were severely overloaded with storage materials. This is consistent with observations by the UNAM team which identifies overload as a cause of 9 percent of building failures in the buildings they surveyed.

The most spectacular member failures are the local buckling and tearing of the columns. The trussed beam to column connection had local buckling, yielding and fracture. Damage to the bracing and connections was relatively minor.

This building complex will provide the opportunity to study the progressive damage to a steel building leading to collapse. This assumed that the three buildings were nearly identical with identifiable live loads. The large rigid common foundation and closeness to the SCT recorded ground motion provide a good estimate of

input motion. It should be noted that the measured period of the 23-story building prior to the earthquake was 2.0 seconds. The peak spectral response of the SCT record was also at 2 seconds. It is premature to speculate on the behavioural sequences leading to the collapse. Further research is needed to accomplish this goal.

EDIFICIO ATLAS

Originally a 14-story building constructed in the 1940's, Edificio Atlas was damaged in the 1957 earthquake, repaired and strengthened and damaged again in 1985. The original steel moment frame system had steel angle knee-joints to provide lateral stiffness. The strengthening techniques consisted of adding three bays of X bracing in each exterior bay in the longitudinal direction and three bays of bracing in each end frame in the transverse direction of the building.

The upper seven stories of the building collapsed causing the collapse of the front half of the neighbouring seven story reinforced concrete building. It is conjectured that the pounding of the neighbouring concrete building caused the Atlas building to collapse.

The supplemental braces performed as expected in the manner predicted by laboratory tests at the University of Michigan (References 8 and 9).

A major problem which must be considered when adding bracing to existing structures is the connections to the existing elements and the new loads which must be carried by these elements.

OFFICE BUILDING ON AMSTERDAM AVE.

This 12-story single-span rigid-framed steel building (Figure 6) showed

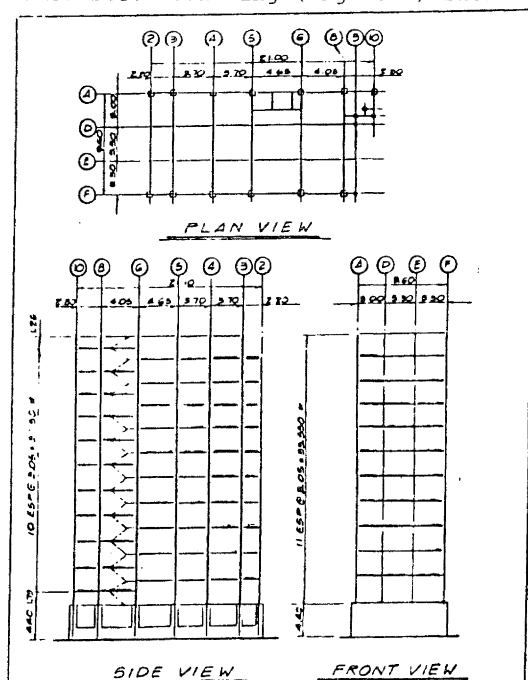


FIG. 6 - AMSTERDAM BUILDING

large lateral displacements hitting adjacent construction. The main damage pattern was cracking of the filler-masonry sidewalls, the breaking of some poor weldments joining the cover plates to the channels in box columns at the rigid joints, and minor local damage at the areas of collision. The main built-up plate girders working compositely with the concrete slab exhibited no distress.

Excessive lateral deflections, rather than insufficient strength, produced the main damage on non-structural elements. The building was partially occupied after the earthquake to allow for repairs.

Dynamic analysis performed recently by the author with a tridimensional model indicated the convenience of changing the dynamic response of the structure, as discussed later.

AZUCAR BUILDING

The Azucar building is "H" shaped in plan (Figure 7). It is a 12-story building used as a government office and was overloaded with files and paper work. The columns are built-up W sections and girders are open-web joists of two-angle chords and box-angles as diagonals in composite construction with the slab. The two legs are joined at the central area by a concrete shear wall elevator core.

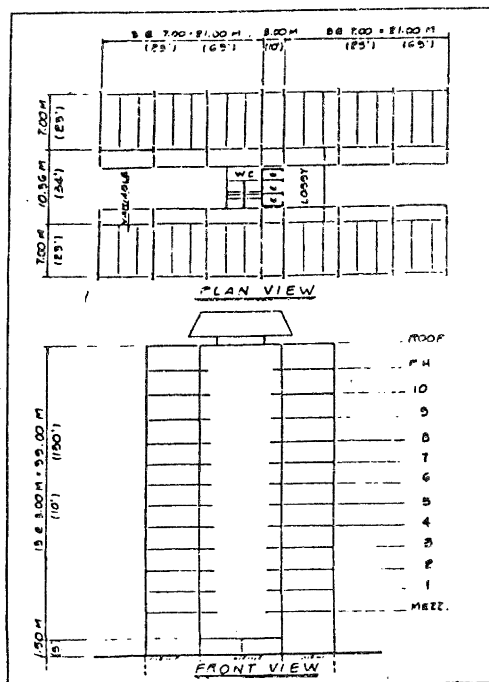


FIG. 7 - AZUCAR BUILDING

The typical damage pattern was local buckling and/or fracture in tension on the main joist girder's web members located normally at the central third of the span and was apparently caused by excessive floor shears transferred from the slab and column rotation at the joints. Some anchor bolts of corner columns were fractured by tension magnifying lateral displacements.

Methods of repairing this building will be discussed later. As in the case of the Azucar building, the excessive lateral deflections of the individual bodies lead to non-structural damage. A concrete building 9-stories high, adjacent to this building suffered pounding which induced the collapse of its three last floors. The steel framed building, as a change, suffered minor structural damage on the contact zones.

BUILDING CODE DEVELOPMENT

The Mexico City building code has been known to have some of the most comprehensive seismic provisions for buildings in the world. Much has been written about building failures in the recent earthquakes; less than one percent of the buildings collapsed or were severely damaged while many thousands of buildings survived as a result of these regulations. This survival is even more remarkable when comparing the actual ground motions with the design ground motions.

Mexico City's first seismic provisions appeared in the 1942 Federal District Building Code. Buildings less than 16 m in height were exempt from the seismic requirements. These provisions were based upon working stress and allowed for a 33 percent increase in allowable stresses for the combination of gravity and seismic loads.

Following the 1957 earthquake, which caused the collapse of several buildings - many under 16 m in height, the 1942 code was replaced by the 1957 Emergency Regulations. While these new provisions continued to be based upon working stress, allowable stresses were allowed a 50 percent increase for the combination of gravity and seismic loads since the base shear coefficients were considered rather high. Many innovations were introduced. The Regulations established base shear coefficients which were specified as a function of a microzonation map of the city as well as the type of structure. Thus, for the first time in the Mexico City Code, the inherent ductility of the structure was recognized.

Ordinary frame structures on the lake bed area were to be designed for a base shear coefficient of 0.17. Base shears were multiplied by an importance coefficient of 2 for important buildings (hospitals and power plants, for example).

The 1966 Building Code continued along these lines but introduced some refinements. The importance coefficient was reduced to 1.3 and the code allowed ultimate strength design. P-delta effect were to be recognized explicitly. Dynamic analysis was allowed provided that the design shear force at any level was not less than 60 percent of that resulting from a static analysis. Buildings larger than 10,000 m² or taller than 45 m were required to be instrumented. Unfortunately, this has not been practiced.

In the 1976 Building Code, the 1957 provisions for microzonation were reinstated. The 1.3 factor for important structures was also retained. Several other innovations were introduced. For most structures the

document was worded in terms of ultimate strength although working stress design was still allowed. Relatively high base shear coefficients were introduced for the lake bed, transition and firm ground zones ranging from 0.24 to 0.16.

These coefficients were to be divided by a ductility factor "Q" equal to 1, 2, 4 or 6 depending on the type of structure and on structural details. The factor of 6 was never used in practice because it required very stringent limitations. The coefficients were calibrated so that for ordinary buildings they would lead to designs roughly the same as those specified in earlier provisions.

For instance, common framed structures on the lake bed were to be designed for a base shear coefficient of 0.24 divided by a ductility factor of 4 resulting in a base shear of 0.06 (as opposed to 0.07 in 1957).

Thirty days after the September 1985 earthquake a new set of Emergency Regulations was issued (Reference 10). The Regulations modified several provisions of the 1976 code

in light of damage that resulted from the 1985 earthquake. Amongst other things, base shear coefficients were increased to a maximum of 0.40 in the lake bed area and the importance factor was increased to 1.5.

The ductility factors were redefined, eliminating the value of 6. The code writers wished to place less reliance on inelastic demand in the long duration earthquakes. Several refinements to design criteria were made, particularly for concrete-framed buildings for which more conservative criteria seemed justified. Provisions were introduced which require review of the design of all substantial buildings by competent engineers and require the permanent presence of a resident engineer during a building's construction.

The design spectra for the three zones for the 1976 and 1985 codes area, is shown in Fig. 8 which compares them with an approximate spectra from the 1985 earthquake. Tables 3 and 4 reproduce a summary of the Code evolution from 1942 to 1966 and from 1976 to the Emergency Code of 1985, respectively.

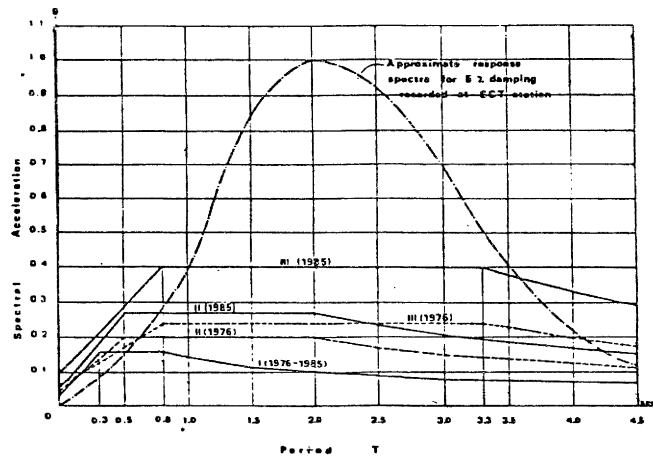


FIG. 8 - DESIGN SPECTRA 1976 VS 1985 CODES

| 1942 CODE | | | | | | | | | | | |
|-----------------------------------|-------|-----------|-----------------------|-----------------------------|-----------------|-------------|----------------|------|---------|-------|--|
| | | | | TYPE I CONSTRUCTION | | C = 0.10 g | | | | | |
| | | | | TYPE II CONSTRUCTION | | C = 0.05 g | | | | | |
| | | | | TYPES III & IV CONSTRUCTION | | C = 0.025 g | | | | | |
| 1957 EMERGENCY CODE AND 1966 CODE | | | | | | | | | | | |
| Year | Group | Structure | ZONE (TYPE OF GROUND) | | | | | | Group A | | |
| | | | I (FIRM) | | II (TRANSITION) | | III (LAKE BED) | | | | |
| 1957 | B | Type 1 | 0.05 | 0.04 | 0.06 | - | 0.07 | 0.06 | x 2.0 | x 1.3 | |
| | | Type 2 | 0.07 | 0.08 | 0.09 | - | 0.10 | 0.08 | | | |
| | | Type 3 | 0.10 | 0.10 | 0.09 | - | 0.07 | 0.15 | | | |

TABLE 3 - 1942, 1957 AND 1966 CODES

| 1976 CODE AND 1985 EMERGENCY CODE | | | | | | | | | | | |
|---|-------|-----------|--------------------|-----------------------|-------|-----------------|-------|--------------|-------|---------|-------|
| VALUES OF SEISMIC COEFFICIENT (C) DIVIDED BY $Q(C/Q)$ | | | | | | | | | | | |
| Year | Group | Structure | Ductility Factor Q | ZONE (TYPE OF GROUND) | | | | | | GROUP A | |
| | | | | I (FIRM) | | II (TRANSITION) | | III LAKE BED | | | |
| 1967 | 1985 | Type 1 | 1 | 0.16 | 0.16 | 0.020 | 0.27 | 0.24 | 0.40 | x 1.3 | x 1.5 |
| | | | 6** | 0.27 | | 0.033 | | 0.040 | | | |
| | | | 4 | 0.040 | 0.040 | 0.050 | 0.068 | 0.060 | 0.100 | | |
| | | | 3 | 0.053* | 0.053 | 0.067* | 0.090 | 0.080* | 0.133 | | |
| | | | 2 | 0.080 | 0.080 | 0.100 | 0.135 | 0.120 | 0.200 | | |
| | | | 1.5 | 0.107 | 0.107 | 0.133 | 0.180 | 0.160 | 0.267 | | |
| 6 | 5 | 1 | 0.160 | 0.160 | 0.020 | 0.270 | 0.240 | 0.40 | | | |

* Structural characteristics for Q=3 were not specifically defined in the 1966 code, but left to the engineer's opinion.
 ** Structural characteristics required for Q=8 were so stringent that almost it was never used.
 Notes: Deflections obtained to be multiplied by Q

TABLE 4 - 1976 CODE AND 1985 EMERGENCY REGULATIONS

POSSIBLE METHODS OF REPAIR TO DAMAGED STEEL BUILDINGS

As mentioned before, most steel buildings performed remarkably well in this earthquake, displaying their reserve of strength and ductile behaviour in such extreme situations.

A few others suffered some damage, mostly caused by pounding against adjacent structures. The following are proposals which have been made for rehabilitating such damaged buildings and apply to particular cases in which drift was the primary cause for damage.

The office building on Amsterdam Ave, for example, required only minor reinforcing elements according to the author, which opposed to previous opinions recommending both a reduction of four floors in height and the reinforcing of the buildings columns. By the addition of some elastomeric dampers in the short direction of the building, at four of the six frames, and two eccentric braced bays, one at each long side, the building significantly improved its dynamic response by increasing its damping to about 20 percent of the critical, resulting in a diminished spectral acceleration which considerably reduced the horizontal inertial forces. (Figure 9). The dampers were installed on the lower vertex of an additional narrow "V" brace, at the connection with the main floor girders, as shown in Figure 10.

The recommended damper material is an elastomeric rubber pad vulcanized to steel connectors.

The second repair case is that of the Azucar Building. The main repairing and reinforcing criteria consisted in reducing both the drift and the overturning moment on the foundation, as well as reinforcing the collapsed web members

on the damaged principal joist-beams (those framing into columns). Such criteria may be summarized in two points:

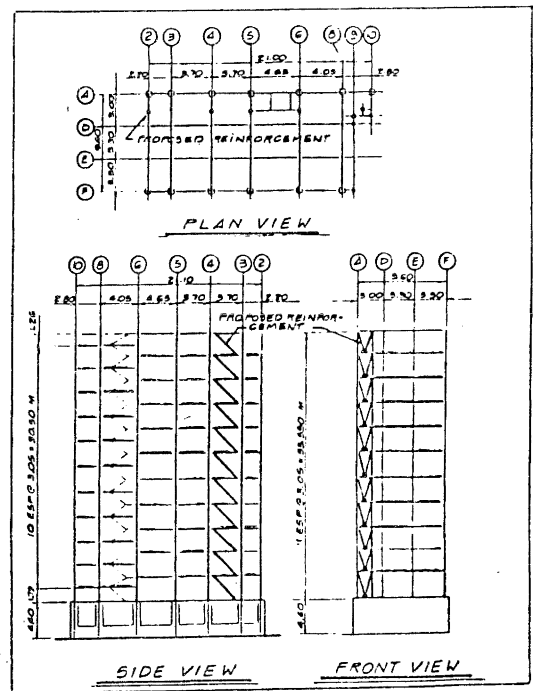


FIG. 9 - REPAIR FOR AMSTERDAM BUILDING

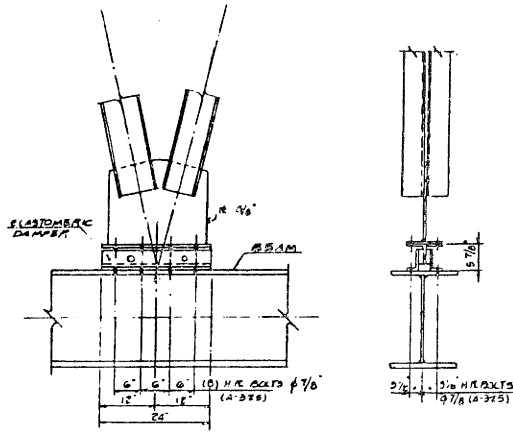


FIG. 10 - DAMPER DETAIL

1) Tying the two bodies together at every floor in order to form a single unit with larger lateral stiffness. This may either provide ample halls at front and back with glass curtain walls or additional floor area. The latter option will require a pair of eccentric braced bays on the short sides at each end of the building which may be concealed by the present facade, as shown in Figure 11.

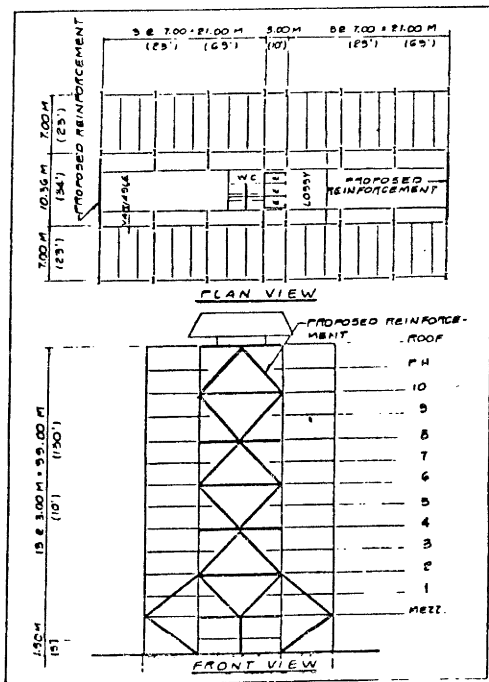


FIG. 11 - REPAIR FOR AZUCAR BUILDING

2) Each joist girder will be reinforced by welding a 3/16" thick plate to each side of the web members in order to restore the

shear strength of such members and improve its ductile behaviour, (Figure 12).

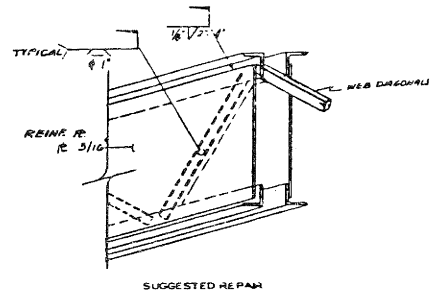


FIG. 12 - JOIST REINFORCEMENT

It is to be noted that both buildings were designed and built according to the 1966 Code in the lake bed area of the city. Nevertheless, their seismic performance was considered quite good and the damage suffered was acceptable. However, the 1985 Emergency Regulations state that a damaged building should not only be restituted in its original capacity, but also must be adequate for the new demands of safety, which can be demonstrated are almost twice as strong as the ones dictated by the 1976 Code. Here exists a very important advantage of steel construction versus concrete. Both of these structures required less than 10 percent of its original steel tonnage to both repair damage and to strengthen them to resist almost twice as much lateral load, as imposed by the Emergency Regulations. The author considers this particular fact a very remarkable advantage of steel construction.

CONCLUSIONS

The September 1985 Mexico earthquake has provided the opportunity to expand our understanding of the response of steel buildings to severe earthquake excitation. The research necessary to extract this information in a reliable, useful format for application in seismic zones is just beginning. Therefore the conclusions are presented as questions to be answered rather than answers themselves.

1. The long period, long duration, large amplitude ground motions experienced in Mexico City (400 km from the epicenter) provides a dynamic input much larger than previously anticipated by building codes for tall buildings. Are there other seismic areas in the world likely to cause similar dynamic input motions? Should mechanical damping devices be utilized to limit dynamic response amplitudes and dissipate energy?

2. Box columns are popular in Mexico and in everywhere else because they can be designed to provide economical flexural resistance about both principal axes. Are current Design Specifications satisfactory for members subjected to large inelastic moment reversals when subjected to large axial compression loads? What ductility levels are necessary for these primary structural members?

3. Effective, economical details for providing ductile connections of beams and

girders to box columns must be demonstrated. How much ductile capacity is needed? Are the requirements for additional detailing of special moment steel frames going to reduce system ductility?

4. Simple structural systems, easy to analyse and easy to construct, are gaining popularity worldwide. Does the absence of significant alternative load paths (redundancy) require substantially more ductility from the only load path available? Do our current seismic requirements ensure adequate ductility for these simple systems? Should highly redundant load path systems permit lower requirements on individual ductile connections for some of the paths?

5. Compact section requirements for rolled sections are specified for special moment resisting frames (ductile moment frames). Should similar requirements be established for trussed girders and beams as commonly used in Mexico City?

6. Most of the modern steel buildings in Mexico City, located on severe excitation sites and with building characteristics for which significant dynamic response was experienced, performed well. Should this give us confidence that buildings designed in accordance with current codes will perform well? What is the current margin between building damage and building failure (collapse)? What should the margin be?

7. Some of the collapsed buildings in Mexico City were found severely overloaded. Others were being used for a different purpose as originally designed. Therefore, should Codes implement temporary inspections to buildings in operation in order to opportunistically detect overloads or misuse of them? Should these inspections be mandatory to "renew" the building "user's licence"?

8. A good part of the observed damage on buildings was related to inadequate foundation performance, since lateral deflections were in many cases magnified by foundation tilting. Should then Codes include requirements to account for the soil-structure interaction? Can this be done without significant complication of the design phase?

9. Are we capable to efficiently design building structures to resist lateral accelerations approaching 1.0 g?; and if so, at what cost?

The eventual resolution to these questions will definitely lead to better designed and safer steel constructions.

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