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Performance of structures during the September 19 2017 Puebla Morelos Earthquake in Mexico

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ABSTRACT

A magnitude 7.1 earthquake occurred in central Mexico on September 19, 2017 in the states of Morelos and Puebla, including parts of Mexico City. The Canadian Association for Earthquake Engineering sent a team of geotechnical and structural engineers to the earthquake-stricken region to investigate the effects of the earthquake from Canadian seismic design perspectives. Structural damage was primarily observed in non-ductile reinforced concrete frames with and without masonry infills, confined masonry buildings, and traditional non-engineered unreinforced load bearing masonry and adobe buildings. The building inventory consisted of pre-1985 poorly designed reinforced concrete frames and post-1985 well-designed buildings with seismic force resisting systems consisting of reinforced concrete frames or frame-shear wall interactive systems. A great majority of buildings performed well. Unlike the interplate earthquake of 1985, which occurred off the coast along the Middle America Trench, 350 km west of Mexico city and caused widespread damage to long period structures in the central part of the city, the intraplate earthquake of September 19, 2017 with different frequency content produced spectral peaks in the transition zone between 1.0 and 2.0 sec, damaging mid-rise buildings. Of particular interest was the performance of buildings with two different retrofit strategies implemented after the 1985 earthquake; column retrofit involving externally placed welded steel angle/strip cage for improved shear capacity, and lateral bracing system. It was observed that the former performed poorly whereas the latter showed superior performance. In addition, some minor damage was observed in bridge infrastructure. The paper presents the findings of the reconnaissance team in terms of structural performance.

Keywords: Puebla Morales Earthquake, reconnaissance visit, structural performance, seismic retrofit, seismic damage.

INTRODUCTION

A magnitude 7.1 earthquake occurred in central Mexico on September 19, 2017 at 1:14 pm local time, causing widespread structural damage in the states of Morelos and Puebla, including parts of Mexico City, resulting in 369 casualties. The epicentre of the quake was 120 km southeast of Mexico City, 12 km southeast of the city of Axochiapan, Morales on the boundary between Puebla and Guerrero. It occurred only 11 days after the Mw 8.1 September 8, 2017 Mexico Earthquake, located further southeast, offshore Chiapas, Mexico. The September 19 event coincided with the 32nd anniversary of the tragic Mw 8.1 Michoacan earthquake of 1985, reported to have resulted in over 10,000 deaths and 30,000 injuries. The Canadian Association for Earthquake Engineering sent a team of geotechnical and structural engineers to investigate the effects of the earthquake from Canadian seismic design perspectives.

Structural damage was primarily observed in non-ductile reinforced concrete frames with and without masonry infills, confined masonry buildings, as well as unreinforced load bearing masonry and adobe residential buildings. The building inventory consisted of pre-1985 poorly designed buildings and post-1985 well-designed buildings with seismic force resisting systems consisting of reinforced concrete frames or frame-shear wall interactive systems. It was observed that some of the post-1985 frame buildings were also damaged due to poor seismic detailing. Unlike the interplate earthquake of 1985, which occurred off the coast along the Middle America Trench 350 km west of Mexico city and caused widespread damage to long period structures in the central part of the city with least favorable soil conditions (Zone IIIc and IIId), the intraplate earthquake of September 19, 2017 with closer proximity and different frequency content produced spectral peaks in the transition zone (Zones IIIa and IIIb) between 1.0 and 2.0 sec., damaging mid-rise buildings. The Seismic Zones are illustrated in Fig. 1. The figure also shows the locations of collapsed building in the transition zone between the former Texcoco lakebed in central Mexico City and the surrounding hilly terrane. The zones are defined based on the fundamental site period, which is essentially a function of the thickness of soft lacustrine clay of the region, changing between 1 sec in the hill zone having no lacustrine clay and 4 sec in central Mexico with a thickness of up to 60 m of clay.

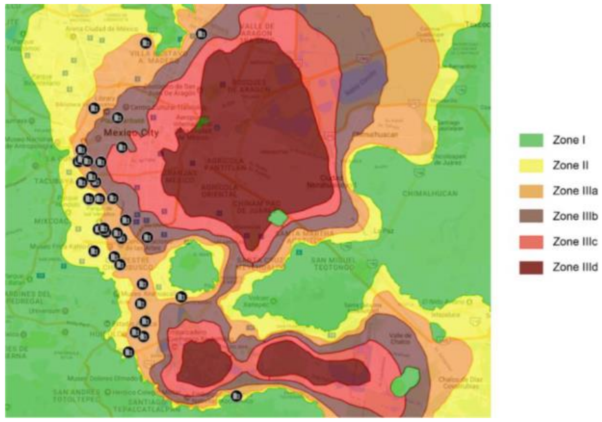


Figure 1. Map showing seismic zones and locations of collapsed buildings https://www.sismosmexico.org/mapas

Typical response spectra of seismic records in two selected zones are illustrated in Fig. 2, demonstrating the significance of the effects of prevailing soil conditions [1]. Figure 2 also shows the design response spectra for Vancouver as the most populous city in Western Canada with similar seismicity as Mexico City, specified by the 2015 National Building Code of Canada (NBCC) [2]. The shear wave velocity in different zones of Mexico City varies between 117.4 m/s and 81.8 m/s for Zones I and Zone III(d) [3], which corresponds to Site Class E (Vs< 180 m/s) in the NBCC site classification, indicating soft soil. Comparison of design spectral values with selected spectra of the recorded ground motions indicates that buildings having about 1.0 s period in Vancouver could be vulnerable to similar ground shaking if located on similarly soft soil without sufficient ductility. The average peak ground acceleration (PGA) recorded during the earthquake at an average epicenteral distance of 115 km indicates a variation between 0.155g and 0.107g [3]. In comparison, the NBCC specified maximum PGA for Vancouver is 0.369g.

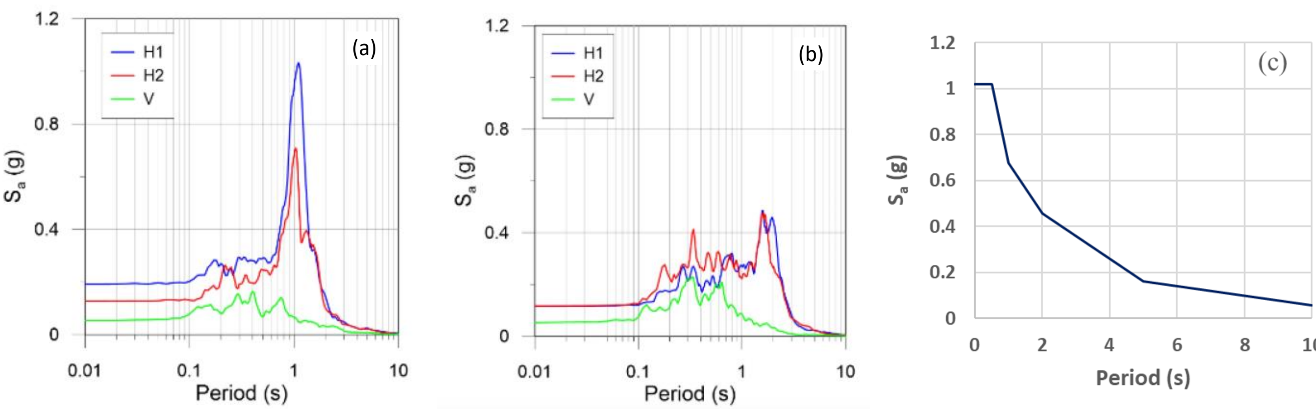


Figure 2. Acceleration response spectra, (a) at Station DX37 Transition Zone II, (b) at Station CI05 Lake Zone IIIa, (c) NBCC-2015 UHS for Vancouver, Site Class E (CAEE 2019)

Performance of structures observed during the reconnaissance visit are summarized below. The geoscience aspects of the earthquake are presented in a companion paper by Lo and Yniesta [1]. Detailed discussion of observations made are provided in the CAEE Reconnaissance Report [3].

DAMAGE TO BUILDING STRUCTURES

Reinforced concrete buildings

One of the common features of reinforced concrete frame buildings, often with masonry infill walls, was lack of ductile detailing in reinforced concrete columns and use of strong beams and weak columns. Figure 3 illustrates a 3-storey reinforced concrete frame building with exterior masonry infill walls. It was being used as a sports facility prior to the earthquake with open interior space. The building suffered from hinging of columns, top and bottom, when the strong and rigid floor system transferred seismic forces to the supporting columns. The failure was in the form of collapse of second and third floor columns.

The building was designed and built during the pre-1985 era with poor seismic design practices. Another potential factor for the collapse was the rear support of the building, which consisted of a slender confined masonry wall with small size columns and confined brick masonry.

(a) (b) (c)

Figure 3. Lack of sufficient bracing and use of strong beams - weak columns

Lack of column transverse reinforcement was evident in older pre-1985 buildings. Figures 4(a) and 4(b) illustrate wide diagonal tension cracks in one of the first storey columns of a multi-storey reinforced concrete frame building with masonry infill walls, located in the Piedad Narvarte district of Mexico City. The figure also illustrates buckling of column longitudinal reinforcement in the vicinity of the previously opened wide diagonal cracks. Similar diagonal tension cracks were observed in other older residential buildings as shown in Figs. 4(c) because of increased seismic force demands associated with soft stories.

(a) (b) (c)

Figure 4. Diagonal tension cracking of first-storey columns due to insufficient transverse shear reinforcement

Some of the columns also suffered from lack of concrete confinement and buckling restraining ties. Figure 5(a) shows hinging of a concrete column in a residential building. Buckling of longitudinal reinforcement with insufficient column ties of a government office building near San Antonio Abad Subway Station in Mexico City is illustrated in Figs. 5(b) and 5(c). The figure demonstrates that widely spaced small size perimeter and interior ties were not able to prevent buckling of disproportionally larger longitudinal bars in compression. These buildings were designed and built prior to the revisions of the seismic detailing requirements of the code of pracice after the 1985 Mexico City Earthquake. However, some of the newer 5-storery reinforced concrete buildings designed and built recently also suffered column damage though these buildings in general performed well. This is illustrated in Fig. 6. The factors contributing to the column damage in this building was widely spaced column ties and increased seismic force demands due to the short column effect.

A number of reinforced concrete frame buildings were retrofitted after the 1985 Mexico City Earthquake. The retrofit techniques included column strengthening and frame bracing with diagonal steel elements. Figure 7 illustrates a typical column strengthening technique commonly used in Mexico City. The figure illustrates the retrofit of Telmex Mexican Telecommunication Company building after the earthquake with columns that had already suffered some shear cracking. The technique consists of steel angles placed longitudinally at each column corner with steel strips welded to each angle in two transverse directions. It is interesting to note that this building had a concrete shear wall in the short direction and the column shear cracks were associated with shear force reversals in the long direction. The same technique was used in an 8-storey reinforced concrete government building located near the San Antonio Abad subway station. The columns of the building were strengthened as shown in Fig. 8. However, this did not prevent the failure of columns and the collapse of the 4th floor in the corner, while many other columns in the building suffered varying degrees of damage. The same column retrofit strategy was also implemented in one of the residential reinforced concrete frame buildings in the Calle Sonor a Esquina Parque district of

Mexico City. This eight-storey reinforced concrete frame building, shown in Fig. 9, had been retrofitted before the earthquake, possibly in anticipation of increased seismic shear demands at the level below where it was supported laterally by the adjacent building. The building suffered from pounding against the adjacent building and the 6th floor collapsed after shear rupturing of columns.

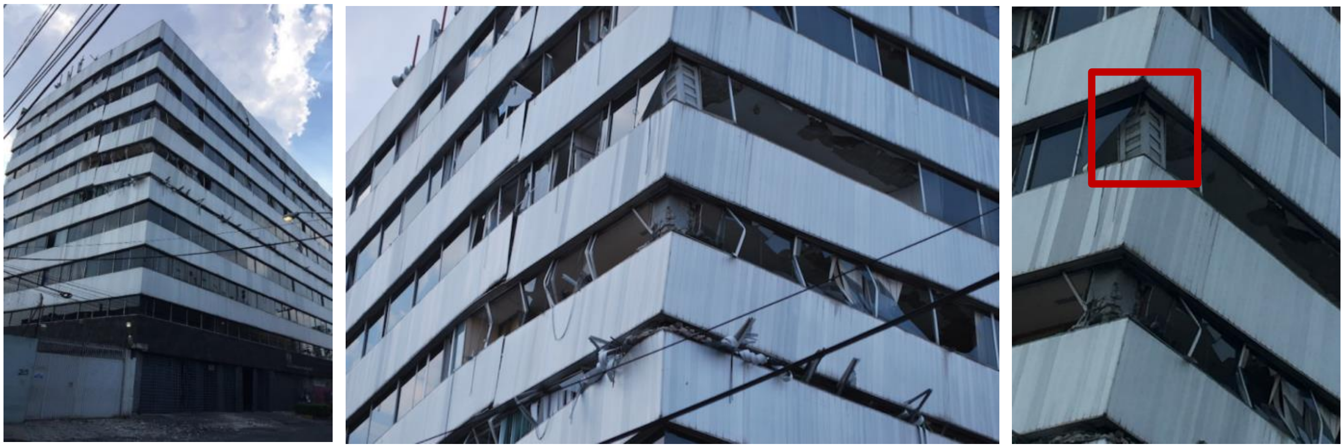
(a) (b) (c)

Figure 5. Lack of concrete confinement and buckling restraining ties in pre-1985 buildings

Figure 6. Lack of column transverse reinforcement and short column effect in a post-1985 reinforced concrete frame building with concrete block masonry infill walls

(a) (b) (c)

Figure 7. Column retrofit technique commonly used n Mexico City



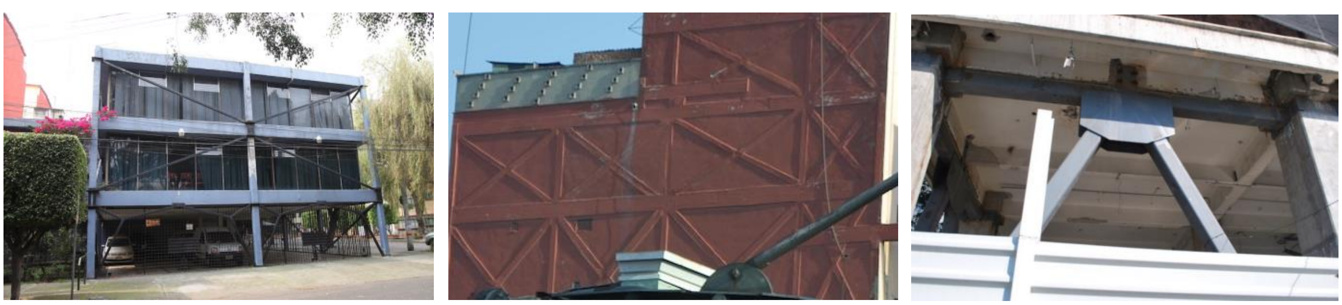
(a) (b) (c)

Figure 8. Eight-storey government office building with retrofitted columns following the technique illustrated in Fig. 7 suffered collapse of the 4th storey columns

(a) (b) (c)

Figure 9. Residential building with column retrofit technique shown in Fig. 7 suffered from pounding of the building against a shorter adjacent building, suffering from shear rupturing of the retrofitted columns

Another seismic retrofit method observed was bracing of frame buildings with structural steel. Two buildings that had been retrofitted with cross bracings before the earthquake survived the quake without any sign of damage. These buildings are shown in Figs. 10. Another reinforced concrete building was in the process of being retrofitted with structural steel braces during the visit as also illustrated in Fig. 10. It appears that, when seismic force and deformation demands are high in soft stories or places of discontinuity, global interference, such as lateral bracing is a better choice than individual column retrofitting.



(a) (b) (c)

Figure 10. Seismic retrofit of frame buildings with steel braces

Masonry and traditional non-engineered construction

Masonry structures in Mexico have been used since ancient times. Historical buildings made of unreinforced, mortared stone lacked rigid diaphragms and details that promoted adequate transfer of forces at wall intersections and plan discontinuities. These structures were very vulnerable to seismic excitations, experiencing considerable damage even under small magnitude earthquakes. Starting in the 1940s, masonry construction in Mexico relied on a system of confined, unreinforced, loadbearing masonry panels surrounded by small cast-in-place concrete bond beams and tie-columns. Non-engineered, confined masonry structures built before the 1985 Mexico City earthquake had large spacings between the concrete columns. They also had brittle details, such as the absence of tie-columns at wall intersections and bond-beams at wall ends, which made them vulnerable to seismic loads. The requirements of the current version of the masonry code for confined masonry [4] limit the distance between bond beams to 3.0 m while also limiting the spacing of tie-columns to the lesser of 4.0 m or 1.5 times the height of the building. Accordingly, the tie-columns must be placed at wall ends and wall intersections, and should be provided around openings whenever horizontal or vertical dimension of an opening are larger than ¼ of the distance between the adjacent tie-columns or 600 mm. The seismic reduction factors in the Mexican code are quite stringent [4]. It is permitted to lower seismic forces by a seismic performance factor of Q = 2.0 if the confined masonry has solid units, and Q = 1.5 if it has hollow units. Lateral displacement/drift limit of 0.25% is prescribed for confined masonry buildings with no horizontal reinforcement in the walls, and 0.35% when horizontal reinforcement is provided. These limits are substantially larger than the 0.15% drift limit prescribed for unreinforced masonry buildings. Unfortunately, these requirements were not implemented in many of the older buildings.

In some multistorey residential buildings the combined use of reinforced concrete frames around the perimeter of the buildings with confined masonry construction elsewhere was noted. Some of the older confined masonry buildings constructed prior to 1985 suffered damage due to lack of proper detailing. A number of apartment buildings visited by the team in the borough of Tlalpan, south of Mexico City showed poor detailing of load bearing walls, tie columns and bond beams. Figure 11(a) shows three attached condominium buildings of similar structural configuration. Fig. 11(b) illustrates lack of sufficient column ties and Figs. 11(c) and (d) show lack of continuity of reinforcement between tie columns and bond beams. The interior and exterior masonry walls suffered significant damage in the form of diagonal shear crack. This is illustrated in Fig. 12. However, reasonably good size columns and beams used around the perimeter of the buildings helped maintain the gravity load carrying capacity even after the loss of some of the load bearing masonry walls.

(a) (b) (c) (d)

Figure 11. Behaviour of confined masonry residential buildings

The use of masonry as in-fill walls either as part of the building envelop or partition walls is common in Mexico. These walls, though intended to fulfill their non-structural functions, often participated in the lateral load resistance of buildings because of improper isolation from the enclosing structural framing elements. In buildings with high wall to floor area ratios and in the absence of structural shear walls, they did help maintain structural integrity, but developed excessive cracking and partial collapses. Some of the perimeter walls also suffered from out-of-plane failures.

Traditional non-engineered masonry and adobe construction is very common in rural areas of Mexico. Simple form of confined masonry, built by local tradesmen or homeowners without adhering to any code or standard resulted in massive destruction of buildings. These buildings are often limited to two stories, with wood roofs, sometimes carrying heavy roofing tiles. Adobe houses are built using thick, handmade units. This type of construction is extremely brittle and many suffered from complete collapse during the earthquake.

Buildings with soft stories

The use of open space at the first-storey level of residential buildings is common in Mexico City. This space is often used for parking, creating soft stories and associated vertical stiffness and strength irregularities. Soft stories usually have reinforced concrete frames with small size columns having limited shear capacities, as well as limited inelastic deformability. Upper floors either have load bearing masonry walls or reinforced concrete frames with masonry infills. The use of soft stories is forbidden in the 2004 Mexico City Seismic Code [5]. However, many were built prior to 2004. Figures 12(a) through (g) show reinforced concrete frame buildings with soft stories. The columns of the building suffered damage due to increased force and deformation demands and lack of sufficient transverse reinforcement to meet these demands. Figure 14(h) shows a soft-storey building with narrow shear walls providing sufficient resistance to increased demands without any sign of damage.

(a) (b) (c) (d)

(e) (f) (g) (h)

Figure 12. Reinforced concrete buildings with soft stories

DAMAGE TO BRIDGE INFRASTRUCTURE

Most of Mexico's bridges were designed and constructed before 1970, without consideration of seismic design. More recent bridge designs have adopted the American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications [6], which include seismic design requirements. Despite the majority of bridges lacking seismic design, bridges generally performed well during the September 19, 2017 earthquake. Members of the reconnaissance team were able to visit four damaged bridge sites within the Mexico City area to gather information, as well as bridge performance observation data on lessons learned from the Mexico earthquake experiences. The types of bridge damage observed include plastic hinge formation at a bridge pier column, foundation rocking, torsional movement of bridge girders, damage to shear keys, large movement or deformation of bridge bearings, longitudinal movement of bridge girders and abutment failures. Figure 13 shows Mexico Metro Line viaduct, which consists of two steel girders at each span, simply supported on single 2.0 m diameter circular reinforced concrete column bents with heavy cap beams. The steel girders support a concrete deck of width sufficient to accommodate two parallel tracks for trains running in opposite directions. The viaduct suffered damage to one of its 2.0 m diameter columns at the intersection of Av. Tlahuac and Calle Gitana between the metro stations Alvois and Nopalera of Line 12. Figure 13 (b) illustrates the formation of a plastic hinge at the base, which was subsequently repaired by providing a reinforced concrete jacket.

(a) (b) (c)

Figure 13. (a) Mexico Metro Line viaduct, (b) plastic hinge forming at the base of a bridge column, (c) longitudinal displacement at deck level and lateral displacement

At a distance 1.2 km south-east of the column damage site, at the intersection of Av. Tlahuac and Av. Guillermo Prieto, the Metro Line 12 viaduct has the alignment layout of an S-shaped double curve. At this location, damage was observed at the shear keys of a span with three steel girders. This was attributed to excessive movement of the girders in the longitudinal direction, resulting in complete spalling of the concrete restraint at the shear keys. Significant rocking of the column piers, possibly exacerbated by torsional response of the curved viaduct, was observed. Figure 13(c) shows significant shift of the girders along the longitudinal direction of the viaduct.

Another bridge damage site visited by the reconnaissance team was a highway bridge located at Circuito Interior Avenida Rio Churubuscophotos in Mexico City. The bridge structure is shown in Fig. 14(a). It consists of two parallel underpass bridges. The 5-span bridge has wall piers and unreinforced masonry (URM) abutments. The URM abutments were damaged during the earthquake as shown in Fig. 14(b). Some reinforcement was observed at the exposed upper part of the abutment; but the lower main part of the masonry abutment was unreinforced. Permanent tilting of some of the wall piers was clearly evident after the earthquake as shown in Fig. 14(c).

(a) (b) (c) (d)

Figure 14. (a) Parallel 5-span underpass, (b) damaged URM abutments, (c) rocking of wall pier, (d) tilting of wall pier

CONCLUSIONS

The soft lacustrine clay in Mexico City tend to amplify ground motions over a broad period range varying between 1.0 and 5.0 sec. The damage caused by the September 19, 2017 event concentrated mostly in Seismic Zones of IIIa and IIIb (the transition zone). The frequency content of the ground motions in the region produced spectral peaks between 1.0 sec and 2.0 sec, affecting mid-rise buildings. Reinforced concrete frame buildings with masonry infill walls, confined masonry buildings and non-engineered traditional masonry and adobe buildings suffered the most damage, especially if built prior to the improvements introduced to seismic design practices after the 1985 earthquake. Lack of seismic design and detailing practices in older reinforced concrete columns and improper use of tie columns and bond beams in confined masonry, as well as non-compliant construction in general were found to be the primary causes of damage, in addition to the ground motion amplification effects associated with prevailing soft soils conditions. Newer building built after the improvement of the Mexico City Seismic Code in the post-1985 era performed well. Soft-storey buildings performed poorly, especially if the soft storey columns did not have sufficient capacity to resist increased force and deformation demands. Retrofitted buildings performed well if the retrofit strategy involved cross bracing of frames providing global drift control. However, common form of column retrofit technique, consisting of externally placed steel cage made up of welded steel angles and strips, was not able to provide sufficient resistance to poorly designed columns. Comparison of seismic hazard values recorded after the earthquake with those that are used to design buildings in western Canada indicates that the buildings in Canada could be vulnerable to similar earthquake if located on similarly soft soils and not designed to have inelastic deformability. The bridges fared well with limited damage.

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