The Performance of Low-Rise Industrial Facilities in the 2010 Haiti and 2011 Christchurch, New Zealand Earthquakes

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SUMMARY:
This paper details the performance of low-rise industrial buildings in the 2010 Haiti and 2011 Christchurch, New Zealand earthquakes. Both events caused significant structural and non-structural damage to these types of buildings. The January 2010 Port-au-Prince Haiti, earthquake caused significant damage to many non-ductile reinforced concrete buildings. Another class of buildings, low-rise pre-engineered metal buildings experienced performance issues, which have not been as widely publicized due to the small number of these buildings. Following the February 2011 Christchurch Earthquake, a survey of selected industrial areas east of the Christchurch Central Business District revealed numerous industrial facilities that had experienced both structural and non-structural damage due to the strong ground motions. This paper will detail the types of structural and non-structural damage observed at industrial facilities in Haiti and Christchurch as a result of the earthquakes and will present potential mitigation measures that could be utilized to address these issues.

Keywords: low-rise structures, seismic design, earthquake reconnaissance, non-structural damage

1. INTRODUCTION

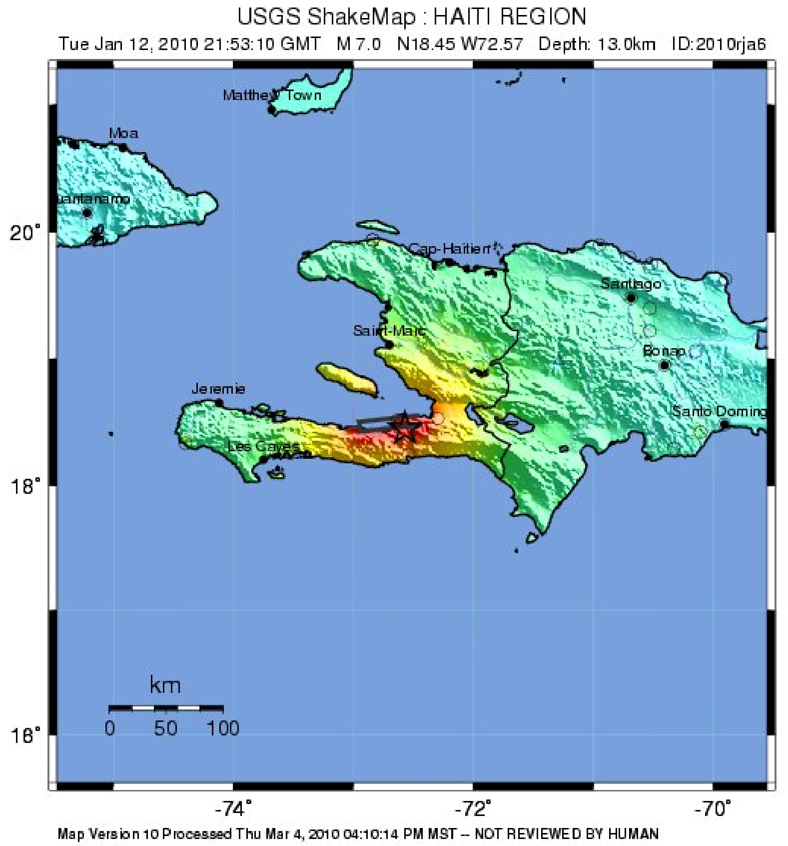
Precast concrete construction is rapid and economical for low-rise facilities requiring large open areas. This fact has resulted in over 650 million square feet of tilt-up building construction in the U.S. annually (Lawson, 2007). It is estimated that there are 20,000 tilt-up or similar types of precast concrete buildings in Southern California (Chou and Anderson, 2007) and 1060 in Shelby County, Tennessee with many more spread throughout Mid-America and other regions of the U.S. (Bai and Hueste, 2006). Many of these buildings house critical industries (Hamburger et al, 1988). Low-rise industrial buildings have a significant impact on local economies. Ensuring that these structures are resilient in the face of earthquakes is critically important.

Multiple major earthquakes around the world have been the norm in recent years. This paper will report on the performance of low-rise industrial structures for two earthquakes with very different ground motions and levels of earthquake preparation. The first is the January 2010 Port-au-Prince, Haiti earthquake which devastated a nation that was almost completely unprepared. The second event is the February 2012 Christchurch, New Zealand earthquake which is the second major event in what is now called the Canterbury Sequence. New Zealand is generally well prepared for earthquakes given the country's history and strong building codes. This paper will briefly describe the two events, the basic types of low-rise structures in the two locales, the performance of low-rise buildings in the events and present possible solutions to the identified problems.

1.1 Port-au-Prince, Haiti Earthquake

In the late afternoon of January 12, 2010 an earthquake struck the eastern portion of the island of Hispaniola. The epicenter was located 15 miles (25 km) southwest of the largest population center in Haiti, Port-au-Prince (PaP). Several other areas of significant population were severely affected. In total, over 3,000,000 people were subjected to significant ground shaking. Official estimates reported

316,000 fatalities, 300,000 injuries and 1.3 million displaced persons. The damage to Haitian infrastructure was severe including 97,294 houses destroyed and 188,383 damaged (USGS, 2010a). The primary port, the power grid and the availability of clean water were nearly completely disabled. The effect from the initial shock was compounded by the significant number and magnitude of aftershocks. People were scared to live in undamaged or lightly damaged homes due to aftershocks and observed collapses of similar homes. The Haitian people were completely unprepared. The USGS Shakemap of the earthquake is shown in Figure 1. It should be noted that no recording stations were close enough to provide accurate measurements of the shaking.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL.(cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Figure 1. USGS ShakeMap for Port-au-Prince, Haiti Earthquake

While the citizens and the infrastructure of Haiti were not ready for a major earthquake, Hispaniola has a history of significant seismicity. It is on the boundary of the Caribbean and North American tectonic plates. On the western half of the island, two faults capable of producing M_W 7 or greater earthquakes have been identified. Port-au-Prince was destroyed in 1751 and 1770 by significant earthquakes thought to be generated by the same fault (USGS, 2010a). Due to the long period of inactivity, the government and the citizens of Haiti were not prepared for the possibility of significant earthquakes. Haitians are accustomed to major natural disasters in the form of hurricanes. Given the known hurricane hazard, it would be expected that some kind of building code would exist for safety. However, this is not the case. Haiti essentially has no building code, no inspection or quality control process, and limited engineering practice. Many residential homes are constructed by the homeowner.

1.2 Christchurch, New Zealand Earthquake

On February 22, 2011, a Mw 6.2 earthquake shook the city of Christchurch, New Zealand. The February earthquake is considered to be an aftershock of the Mw 7.1 Darfield earthquake of September 4, 2010 which is part of a sequence of earthquakes which began in September 2010 and is still ongoing with the most recent Mw 6.0 occurring in December 2011. The population of Christchurch was approximately 348,400 at the time of the September earthquake (Statistics New Zealand, 2012). Although lower in magnitude and shorter in duration than the earlier quake, the aftershock was centered closer to the city and caused significantly more damage including structural, non-structural and liquefaction, particularly in the Central Business District (CBD) and in industrial areas located east of the CBD. It also resulted in 182 fatalities primarily due to collapsed buildings in the downtown area. One likely cause of the difference in damage levels is due to the proximity of the epicenter. The Darfield event was 25 miles (40 km) from the CBD while the February event (Lyttleton Earthquake) occurred within 6 miles (10 km) of the CBD. Figure 2 shows the location of the epicenter for the Mw 7.1 September 4, 2010 Darfield earthquake and the Mw 6.2 February 22, 2011 earthquake along with the aftershock sequence from both events (GNS, 2012). Figure 3 shows the ratio between vertical and horizontal accelerations at multiple recording stations around Christchurch indicating the severity of the vertical component.

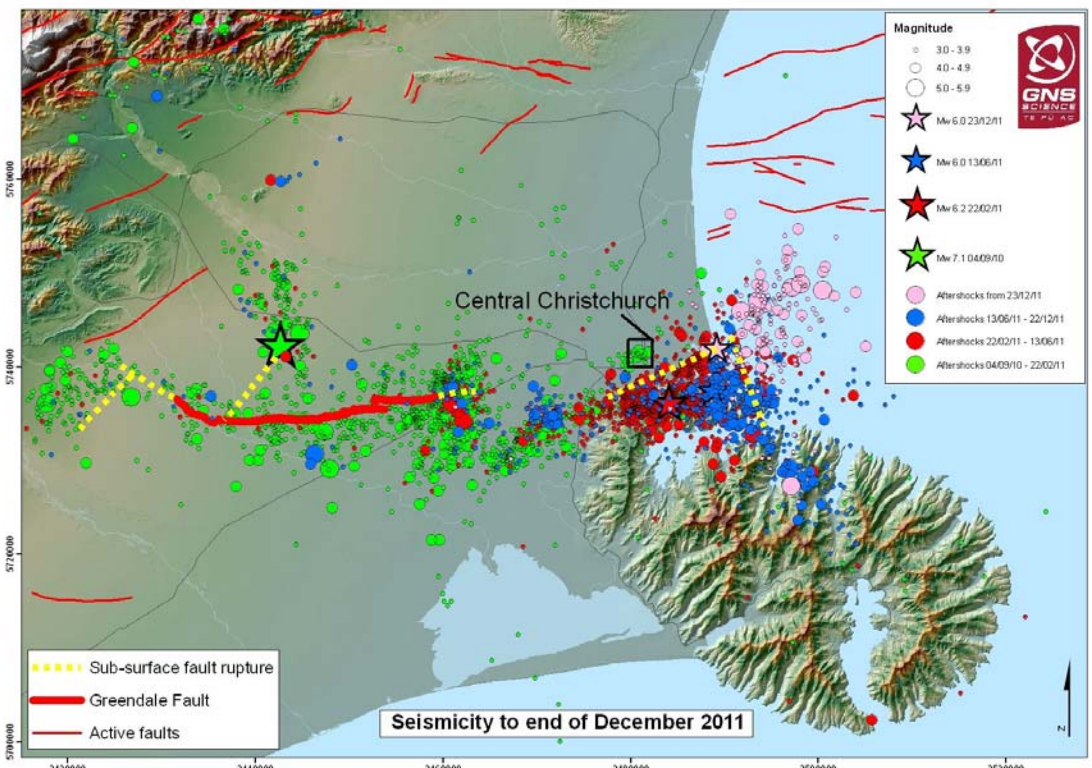


Figure 2. Epicenter of earthquake and aftershocks from the Canterbury Sequence (GNS, 2012)

2. LOW-RISE STRUCTURES IN HAITI

The most predominant building type in Haiti is lightly reinforced concrete frame structures with unreinforced masonry infill and concrete floor and roof diaphragms. Most industrial or warehouse buildings in Haiti are constructed of long span steel trusses supported on masonry-infilled reinforced concrete frame walls. In the United States many of these buildings would be pre-engineered metal buildings (PEMB). Due to the cost and availability of steel in Haiti, metal buildings were the exception. The typical construction of PEMBs in Haiti utilized a masonry façade. Both types of low-rise construction experienced damage although the levels of damage and the ability to reoccupy buildings following the earthquake were different. Numerous low-rise industrial structures were located throughout industrial and port areas of PaP. Due to the wall height, typically low amounts of reinforcing and low compressive loads, wall failures primarily due to out-of-plane effects were common. This type of damage can be seen in Figure 4.

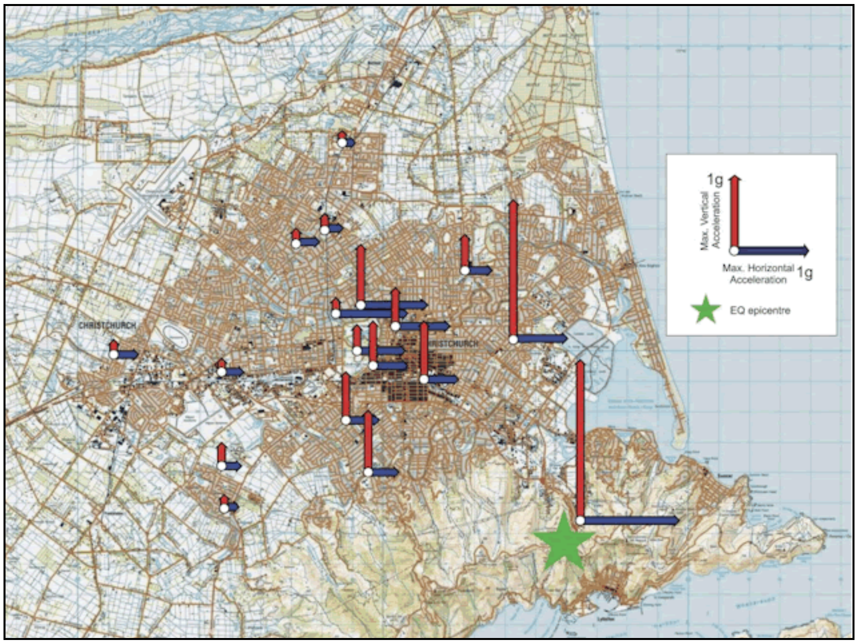


Figure 3. Comparison of horizontal and vertical accelerations from the Canterbury Sequence (GNS, 2012)

Figure 4. Low-rise masonry infill wall failures of industrial buildings in Haiti a) Endwall failure b) Bearing wall failure and temporary truss supports (3b Photo Credit: Steve Baldridge)

During reconnaissance efforts, several PEMBs were encountered and the damage to each of them was similar. The masonry façade failed while the primary structural framework was undamaged. Photos of this failure are shown in Figure 5. Figure 5a is a retail building located across the street from the U.S. Embassy. All of the front sections of the PEMBS observed at this location detached from the framing although some of them had not fallen. The buildings have been repaired and the masonry exterior walls were replaced by insulated metal sheathing. Figure 5b is a warehouse building in PaP with masonry cladding. This out of plane failure of the masonry walls of PEMBs was common on nearly all metal buildings observed during the reconnaissance. PEMBs in Haiti did not experience failure of the gravity load system. This was true even of the warehouse building at the PaP port which failed due approximately 6 ft (2 m) of lateral spread between the foundations.

Figure 5. Low-rise masonry infill wall failures of industrial buildings in Haiti a) Gable wall failure of commercial buildings b) Gable wall failure of warehouse building (4b Photo Credit: Steve Baldridge)

3. LOW-RISE STRUCTURES IN CHRISTCHURCH

Industrial buildings in Christchurch and the surrounding areas can be classified as one of three types. The first structure type consists of load bearing tilt-up precast concrete panels with steel roof framing. The second type includes pre-engineered steel frames with pre-cast concrete cladding panels. The third type includes pre-engineered steel frames with a light gauge metal or insulated metal panel cladding. The first two types of structures were the most common and also experienced the most damage. For buildings with load-bearing tilt-up panels, damage was observed in the panels as well as the connection between the panels and roof framing members. Buildings with steel frames supporting gravity loads and tilt-up panels for lateral resistance experienced a different mode of failure. In several cases, the connections between the steel frames and the precast walls were insufficient and the wall panels fell away from the building. An improved precast-to-steel connection is needed to allow for the incompatibility between the flexible steel frame and the rigid panels. The buildings discussed in the following sections are primarily in the Bromley area which is east of downtown Christchurch and was within 5 km of the epicenter of the February event.

3.1 Load-bearing Concrete Tilt-up Buildings

Typical damage to load-bearing concrete tilt-up panels is shown in Figure 6. Several structures of this type seen throughout the region had to be stabilized with temporary shoring and deadman anchors. In one case (Figure 6a) the building had been temporarily stabilized but was due to be demolished because of multiple failures of structural elements. Several of the beam-to-wall connections suffered significant damage, as shown in Figure 6a. The facility was evaluated by structural engineers retained by the Owner who concluded that the structure could not be economically repaired and as such, it was scheduled for demolition. Figure 6b shows damage which occurred at the corner connection which had already been repaired by the owner to allow the building to be reoccupied. While the damage to the connections at the facility shown in Figure 6b did not result in a collapse of the structure, the level of damage suggest that there was some loss of vertical load-bearing capacity of the beam-wall connection. The structure in Figure 6b is similar and close to the structure in Figure 6a, both of which experienced similar damage. In the case of Figure 6b, an overhead crane served as a strut to maintain stability of the building. Connection repairs were required but the building was able to remain and was operating at the time the photos were taken. At one light industrial facility diagonal shear cracks were observed in the load-bearing concrete tilt-up wall panels. In some cases, the shear cracks appeared to emanate from the connection of the beam to the concrete wall (Figure 7a). Damage was also seen in the flexible diaphragm in the form of fractured tension-only bracing (Figure 7b).

Design drawings were not available for these industrial facilities; however through numerous discussions with several different owners, it appears that the concrete tilt-up panels were typically lightly reinforced due in part to the perception that there was not a significant earthquake risk in the area relative to other parts of New Zealand. It should also be noted that a much older building with more complex geometry across the street which consisted of steel frames with light gauge metal sheathing experienced minimal damage although it did experience some equipment damage.

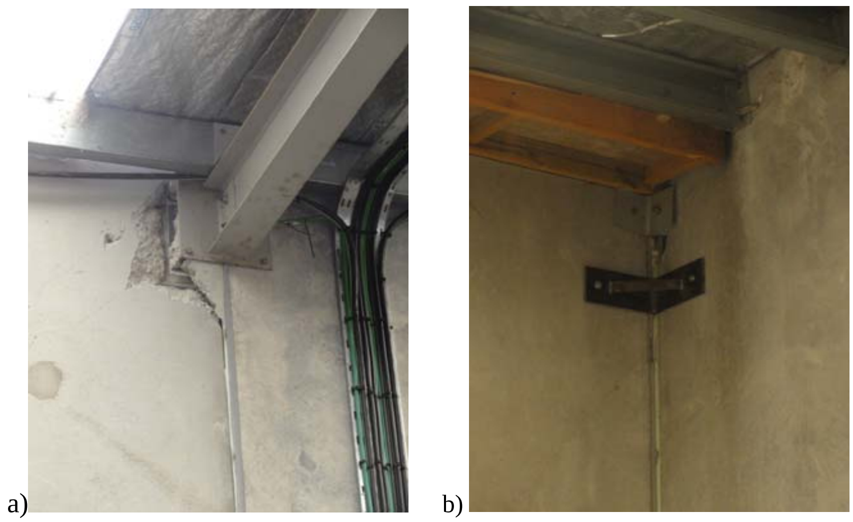


Figure 6. Damaged connections in tilt-up industrial structures in New Zealand a) Beam-to-panel connection damage b) Corner connection damage with post-event repair.

Figure 7. Damaged structural elements in tilt-up industrial structures in New Zealand a) Shear cracking in load-bearing panels b) Fractured tension-only roof diaphragm bracing.

3.2 Pre-engineered Steel Frame Buildings with Precast Concrete Cladding

Buildings with pre-engineered steel frames supporting gravity loads and tilt-up panels for lateral resistance experienced a different failure mode. In multiple cases, the connection between the steel frame and the precast walls was insufficient and the wall panels fell away from the building. Figures 8a and 8b illustrate a typical failure of a wall panel that separated from the steel framing and fell to the ground. Even though the remaining panels did not fall to the ground, the connections were damaged to an extent that shoring was required as illustrated in Figure 8b. This failure mechanism was evident on other buildings of similar construction in throughout Christchurch.

The connections between the steel frame and concrete panels were similar in many of the structures seen during the reconnaissance effort. It was apparent from the framing of the structures that in many cases, the panels were the lateral load system for the longitudinal direction of the building. These connections are therefore a demand critical link in the load path. In some cases tension-only bracing and half height panels were utilized as the lateral system. Other damage was seen to panels of this type of buildings. Figure 9a illustrates damage to a panel with a reduced section due to window penetrations. The damage includes shear failure of the reduced section and out-of-plane flexure of the panel. Figure 9b shows cracking of the end panel which is typically not used to resist loads but did so because of the panel stiffness compared to the steel frame. This type of damage was seen in multiple structures in the area.

a) b)

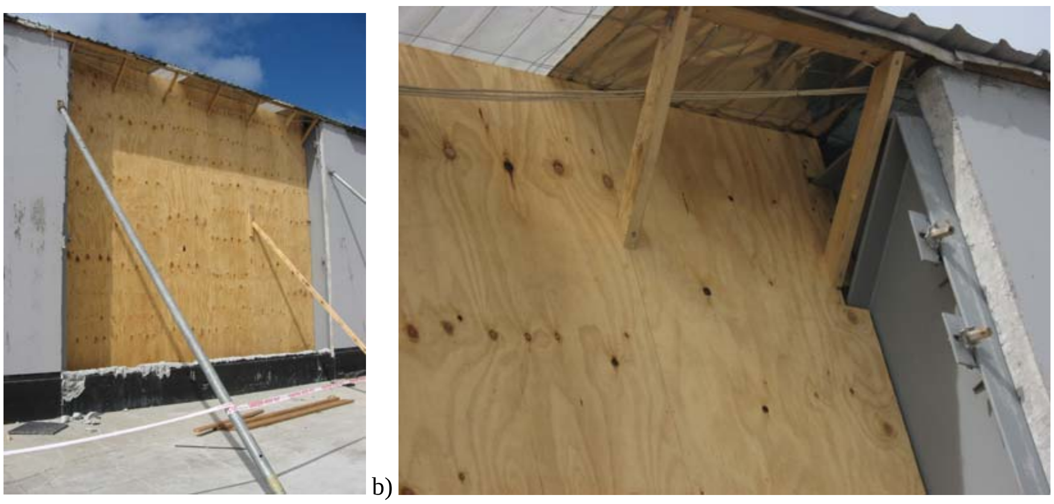


Figure 8. a) Loss of panel and temporary shoring of adjacent panel b) Concrete anchor connection failure.

a) b)

Figure 9. a) Damaged panel due to reduced section b) Cracking of end panel.

4. NON-STRUCTURAL DAMAGE IN CHRISTCHURCH

Non-structural damage to buildings can result in many significant issues. Non-structural damage may include damage to architectural finishes, mechanical or electrical components, or equipment needed for operations. In many cases these types of damage can lead to losses which may also result in downtime and business interruption losses which may exceed which the cost to repair building damage. Considering that the demand for contractor services is very high following an event, multiple effects may contribute to a significant cost increase to get back to an operational status. In the industrial areas east of the CBD, several manufacturing facilities were observed with substantial non-structural damage to critical equipment and supporting infrastructure. At one location in Bromley, high vertical and horizontal accelerations in the area combined to translate machining equipment weighing 25,000 pounds (110 kN) approximately 4 ft (1.2 m) from their original position, as shown in the Figure 10a where the workers right food indicates the original location. Workers at the facility described seeing the large equipment literally “bounce” across the floor. Multiple pieces of equipment within the same facility as well as in other surrounding facilities moved in excess of 2 ft (0.6 m). In some cases, the non-structural damage is not directly shaking related. A textile factory in the Bromley area experienced a significant amount of both structural and non-structural damage. Much of the damage was reparable; however one particular effect rendered the factory inoperable. A combination of strong ground motions and liquefaction induced slab damage resulting in displacements in critical equipment that significantly exceeded levelness tolerances. This equipment damage resulted in facility shutdown and the loss of hundreds of jobs to the facility. Figure 10b shows the equipment damaged by the combination of strong ground motion and the slab displacement resulting from liquefaction.

Another interesting aspect of non-structural damage in New Zealand occurred in low-rise commercial buildings. The first is a commercial mall east of downtown. The structure was composed of a pre-engineered steel frame with precast concrete cladding panels connected to the frame with a concrete pour strip around the steel columns. The pour strip utilized rebars extending out from the precast panels. The problem with the structure was a long cantilevered parapet, approximately 6ft (1.8 m), above the top of the column. This led to significant cracking of the panels which necessitated removal of the panels. This operation is shown in Figure 11a. A similar condition existed with a commercial tilt-up building just south of the central business district. In addition to removal of the long cantilevers, the tilt-up structure was temporarily supported by jack posts and deadman blocks. This structure can be seen in Figure 11b with the removed cantilevers in the foreground.

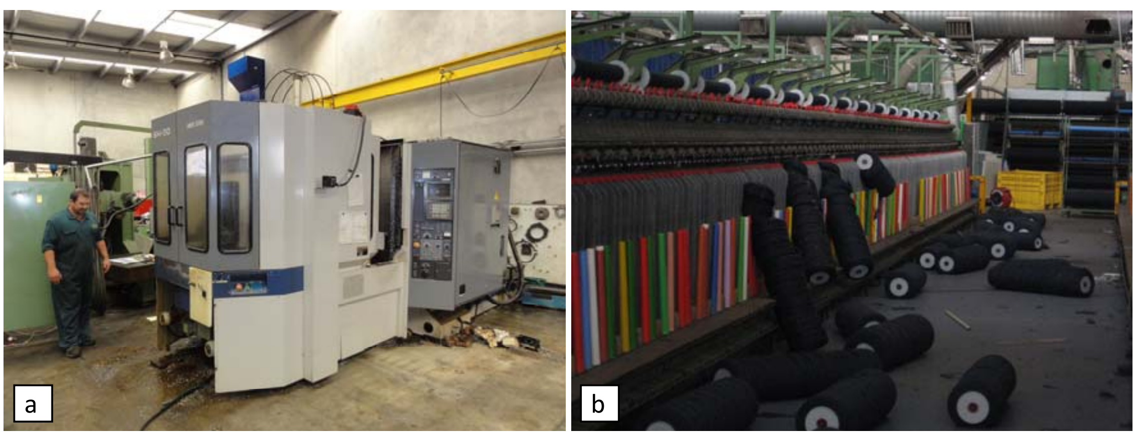


Figure 10. Non-structural damage in industrial facilities a) Heavy equipment moving 4ft (1.2m) from initial position b) Liquefaction damage to slab supporting low tolerance manufacturing equipment.

5. DISCUSSION OF DAMAGE

The amount of damage to structures in Haiti was an extreme example. Many factors contributed to this problem including lack of rigorous engineering design and poor material quality which are systemic problems in Haiti due to the lack of a building code, no construction regulation and severe economic and governmental challenges. The industrial structures composed of masonry-infilled reinforced concrete frame walls performed poorly which often reported in partial collapse and loss of the gravity load system. Many of these structures were essential to the recovery process for storage of commodities coming into the port. While few in number and still experiencing some damage, pre-engineered steel frame buildings demonstrated better performance. The loss of masonry cladding at the gabled ends of the structures showed that one critical detail for these structures was missing. However, these facilities still had a complete gravity load path so use of the buildings during recovery was possible. This detail should be a critical note for structures in the United States as the metal building manufacturers and the masons or cladding panel producers must ensure that these connections are designed, detailed and constructed to support the required lateral loading as falling panels or blocks present a significant potential for damage to other elements or building occupants.

a) b)

Figure 11. Precast panels with vertical cantilevers greater than 6 ft (1.8m) a) Commercial mall wall panels attached to pre-engineered steel frame b) Commercial tilt-up building.

The primary problem with the performance of these low-rise industrial buildings with precast panels in Christchurch during strong ground motion appears to be the connections between the wall panels and the steel framing. For structures with load-bearing precast panels, the damage was observed at the roof beam-to-panel connection. This is likely occurring because all the out-of-plane wall forces are transferred to the diaphragm through the beams which are typically widely spaced. Significant normal forces are introduced into a thin wall panel. Essentially, the connection does not have a ductile fuse mechanism. One solution to this problem is to provide a continuous ledger with a positive connection to both the diaphragm and wall panels. This allows the out-of-plane forces to transfer along the length of the wall and reduces the concentrated forces at the beam-to-panel connections. It would also help to tie the panels together to provide additional capacity against panel out-of-plane failure.

For non load-bearing panels the problem is the deformation incompatibility between the stiff wall panels and the flexible steel frame. Two options are presented here which would likely alleviate the problem. The first is to provide a header element along the top of the panel with all the connections provided along this element. The inertial load could be transferred from the diaphragm at this point. The column and panel could move relative to each other without affecting the tie between the steel frame and the panel. An alternative would be to modify the connection between the column flange and wall panel. This includes increasing the number of connections and/or the strength of the connection. Current research on metal buildings with concrete cladding panels has used a welded connection between the panel and the frame which performed well in shaking table testing at the University of California San Diego.

End panels on manufactured buildings experienced cracking due to acting as a part of the lateral system due to their stiffness. While this is a feasible load path if the connection is designed to transfer the lateral loads, another option, which is to utilize the steel frame may be better. The panels will often crack due to relatively small lateral loads as the panels are typically lightly reinforced. The steel frame has the flexibility to deform without damage. In order to get the load into the steel a connection is required which allows the load to transfer into the frame. Providing a slot in the connections which allows the panels and frame to slide without degrading the tensile capacity of the concrete anchor would meet these requirements. It is also possible that the connection would dissipate energy through friction without damage to the structure. This would require careful design to determine the length of the slot and the appropriate direction of the slotted hole.

The approach for design of the long parapet would be to ensure that either the panel has the capacity to resist the loads or to provide bracing to achieve the desired behavior. These issues must be considered as part of rigorous design to ensure that buildings and businesses they house, do not experience significant downtime and sever economic impacts due to seemingly insignificant construction details.

6. CONCLUSIONS

The January 2010 Haiti earthquake and the Canterbury Sequence of events around Christchurch, New Zealand resulted in considerable structural and non-structural damage at numerous industrial facilities. The two events were different in many ways but both highlighted possible problems with low-rise industrial structures. Many modern industrial structures, comprised of load-bearing concrete panels, or steel frames with non load-bearing concrete cladding or unreinforced masonry cladding, suffered significant structural and non-structural damage. Some of the damage was due to failed connections which lacked the strength and ductility to accommodate the high accelerations and differential displacements at the interface of the steel framing and concrete panels. Other damage was due to lacking or non-existent non-structural detailing. The recommended detailing and connection changes to improve performance require further research to verify that they would provide sufficient load transfer and deformation capacity.

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