Recipe for Disaster: Construction Methods, Materials, and Building Performance in the January 2010 Haiti Earthquake

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The earthquake that shook Hispaniola on 12 January 2010 devastated Haiti. The damage was widespread due to uncontrolled construction, poor material quality, and lack of rigorous engineering design. Post-event reconnaissance has brought to light serious deficiencies in these areas. Residential buildings in Haiti are typically constructed by their owners, who may or may not have the skills or resources to build a structure that is earthquake-safe. Few structures are designed by engineering professionals or are inspected for quality of construction. The two most common construction materials are masonry block and reinforced concrete. Masonry blocks, concrete cylinders, and reinforcing steel were taken from Haiti and tested in the United States. The concrete and masonry were shown to be of low strength and quality. The steel samples show expected strength properties with some specimens having reduced ductility due to bending. Building performance is demonstrated by reconnaissance photographs and case studies of the structures inspected by reconnaissance team members. [DOI: 10.1193/1.3637031]

INTRODUCTION

On the afternoon of 12 January 2010, an earthquake struck the island of Hispaniola. The epicenter was located at 18.443° N latitude, 72.571° W longitude, 15 miles (25 km) southwest of the largest population center in Haiti, Port-au-Prince (USGS 2010). Although there were no recorded motions, the U.S. Geological Survey (USGS) ShakeMap intensity ranged from modified Mercalli intensity (MMI) VIII to X over the area, including Port-au-Prince, Carrefour, Pétion-Ville, Léogâne, and Grand-Goâve (USGS 2010). According to official estimates reported by the USGS, 222,570 people were killed, 300,000 were injured, and 1.3 million were displaced (USGS 2010). The damage to Haiti’s infrastructure was severe. The primary port, the power grid, and the water system were nearly completely disabled. The effects of the initial shock were compounded by the number and magnitude of aftershocks. People were afraid to live in undamaged or lightly damaged homes during the aftershocks because they observed the collapse of similar homes. The Haitian people and their infrastructure were completely unprepared for this event.
Haiti occupies the western portion of Hispaniola. Figure 1 shows satellite imagery of the most affected regions in Haiti. The authors of this paper represent two reconnaissance teams. The areas visited by the authors include Port-au-Prince, Carrefour, Léogâne, Grand-Goâve, Petit-Goâve, Pétion-Ville, and Cité Soleil. Figure 1 also shows the epicenter and the assumed trace of the Enriquillo-Plantain Garden Fault (EPGF; USGS 2010). Hispaniola is at the interface of the Caribbean and North American tectonic plates. The western half of the island has two identified faults capable of MW 7 or greater earthquakes. The southern section of the east–west fault system thought to be responsible for the January 2010 event had not produced a significant event since 1860. Prior to that, Port-au-Prince was destroyed in 1751 and 1770 by earthquakes thought to be generated by the EPGF (USGS 2010). Recent work has identified a previously unknown fault, the Léogâne Fault, which is thought to run roughly parallel to and just north of the EPGF. The faulting mechanism and geological data suggest that the Léogâne Fault, rather than the EPGF, was responsible for the January 2010 event (Calais et al. 2010).

Due to the long period of seismic inactivity, the government and the citizens of Haiti were not cognizant of the possibility of major earthquakes. Haitians are accustomed to natural disasters in the form of hurricanes. Four major storms hit the island in 2008. Given the known hurricane hazard it seems irrational that Haiti has no building code, no inspection process, and limited engineering practice.

The lack of resources applied to civil infrastructure and citizen education for the eventuality of an earthquake was evident. Damage surveys conducted in downtown Port-au-Prince and Léogâne provided an approximation of local collapse percentages. The surveys were accomplished using reconnaissance photos correlated with pre- and post-event satellite imagery to determine the number of structures in existence prior to the event. The route in downtown Port-au-Prince was rectangular with a long side of 1,200 ft (365 m) and a short side of 350 ft (105 m). The total number of structures along the path was estimated to be

Figure 1. Google Earth image of region affected by the 12 January 2010 earthquake.
Of those structures, 30 (28%) were classified as collapsed or severely damaged. A similar survey was completed in Léogâne, 7 miles (11 km) from the epicenter with a population of approximately 80,000 (CityPopulation 2010). The Léogâne route was L-shaped with a long and short side of approximately 1,200 ft (365 m) and 360 ft (110 m), respectively. An estimated 52 structures were surveyed, although the precise tally was impossible to determine due to excessive damage and poor overhead image quality. Of 52 structures, four were classified as undamaged and 32 (62%) were classified as total or partial collapses. The high collapse rate may be due primarily to epicentral distance, but could also be related to soil conditions. More detailed information on earthquake damage can be found in the various reconnaissance team reports (Eberhard et al. 2010, EERI 2010a, EERI 2010b, Fierro and Perry 2010).

The significant damage to Haiti’s infrastructure was due to a lack of planning for this type of event. Common nonengineered buildings experienced structural failures due to a lack of engineering design considering seismic hazards, substandard and uncontrolled construction, and poor building material quality. Each of these issues will be explored throughout the remainder of this article. Typical construction practices will be detailed, including a discussion of how the typical practice contributed to failures during the earthquake. The results of material tests of samples secured in Haiti and tested in the United States will be presented and discussed. The final section presents case histories, each of which highlights the different elements of Haitian building practice that led to unacceptable structural performance. It is important to bring these aspects of Haiti’s construction industry to light so that they can be dealt with appropriately during reconstruction. If the status quo is not changed, the next significant seismic event will be a repeat of the last.

**TYPICAL CONSTRUCTION PRACTICES**

**BUILDING AND LAND USAGE**

The most prevalent building type in Haiti, particularly in the Port-au-Prince region, consists of nonengineered, lightly reinforced concrete frames with unreinforced concrete masonry block infill. These low-rise buildings are used as single-family dwellings and for small businesses. The familiar mixed-use “soft-story” design common in the developing world, whereby the ground level is dedicated to commercial space and the upper floors are high-density residential apartments, is not prevalent in Haiti. Most people live and work in different areas. Soft stories are still a problem, though. Large window openings and reduced wall area caused numerous floor collapses, both at ground level and at higher floor levels.

Most of these nonengineered buildings are one or two stories, though three- and four-story nonengineered buildings were also observed. Concrete blocks were the prevailing masonry unit utilized. Other types, including fired clay brick, were not seen, except in historic buildings. Floors and roofs were reinforced concrete slabs, typically 6 in to 10 in (15 cm to 25 cm) thick with a single layer of bi-directional reinforcement. In some cases, concrete blocks were cast into the slab to minimize the use of concrete. Corrugated steel spanning over wood or steel trusses was also commonly observed.

Many residences in Haiti are constructed over a long period of time as the homeowner acquires funds or the family’s needs expand. Most homes are designed and constructed by
their owners or a local mason. Residents sometimes squat on land, public or private, to be near family, friends, or employment. These unauthorized developments exist on hillsides surrounding Port-au-Prince and Pétion-Ville, as well as in low-lying coastal areas such as Cité Soleil.

CONSTRUCTION MATERIALS

Type I Portland cement was used for all construction elements, including masonry blocks, foundations, wall mortars, roof and floor slabs, columns, and beams. Concrete mix proportions regularly lacked sufficient cement, had poor-quality aggregates, and were heterogeneously mixed. Often, concrete was mixed on the ground. Figure 2 shows an image that was taken at a construction site near the U.S. Embassy where a masonry wall with vertical and horizontal reinforced concrete elements was being constructed. Even with the availability of a concrete mixer, the materials were combined on the ground. Many other sites showed similar evidence. Water content varied depending on availability. If sufficiently available, the water content tended to be high to ease workability. If not, mixes were overly dry. Conversely, contractors working on engineered projects that required large concrete mixers would transport their mixes with a high water content because of evaporation and uncertain transport delays.

In the past, unwashed beach sand was widely used in concrete mixes. It was used in the construction of most of the buildings subjected to the January 2010 earthquake. However, a campaign in past years to discourage its use for construction appears to have been successful; no evidence of its current use was observed and residents were aware that it was objectionable. Aggregate was obtained from nearby limestone quarries or river beds. La Boule, the largest supplier of sand and aggregate for construction in and around Port-au-Prince, produces a light-colored, weak limestone material. A month after the earthquake, Haiti’s

Figure 2. Mixing concrete on the ground for a masonry wall near the U.S. Embassy.
public works ministry issued a ban on using La Boule material and instead recommended using higher-quality material from river beds. It is unclear whether residents have abandoned La Boule material for rebuilding. The river rock sometimes used as aggregate is strong and smooth. The smooth surface reduces the bond strength between the aggregate and cement matrix. Cast-in-place concrete is not typically consolidated, resulting in large air pockets and a poor bond with the reinforcement. Further, the lack of sufficient cement in concrete mixes results in reduced bond strength. Commonly, the strength of the cement bond to the aggregate was insufficient to fracture through the aggregate. Figure 3 illustrates the problems with air voids and poor-quality aggregate.

Concrete masonry blocks are commonly manufactured at or near the construction site. Concrete is typically mixed on-site, poured into formwork, extracted, and set aside to “dry.” Proper curing techniques were not observed. Typical block dimensions are 15.75 in × 7.25 in × 5.75 in (40 cm × 18 cm × 15 cm). Mortar for the block walls is mixed on the ground with the same small aggregate components used for the blocks. Horizontal bed joints are commonly 0.75 in (2 cm) thick; vertical bed joints vary from 0.0 in to 0.75 in (2 cm).

**BUILDING CONSTRUCTION**

The most common construction type in Haiti resembles infill masonry. A typical structure’s vertical-load-resisting system consists of reinforced concrete columns, beams, and slabs. Unreinforced concrete masonry blocks are used to infill the frame. No mechanical connection is made between the masonry wall panel and the columns, floor, or roof slabs. Figure 4 shows examples of typical buildings in Haiti; reinforced concrete elements are infilled with unreinforced masonry wall panels.

The construction sequence of the reinforced concrete frame and masonry walls can vary. Roughly half of these nonengineered structures are assembled in a traditional “infill masonry” manner: The reinforced concrete frame, floor, and roof slabs are assembled and finished first, and then the unreinforced masonry wall panels are infilled within these elements. This construction type is identifiable by clean, distinct lines between wall panels and
Figure 4. Typical reinforced concrete frame with masonry infill construction in Haiti: (a) Single story building with light roof and large openings, (b) multistory frame building with large walls and slender columns.
columns. Conversely, builders also execute a sequence that resembles traditional Latin American “confined masonry,” whereby the masonry wall panels are erected prior to the frame. This permits use of the wall ends as formwork for the reinforced concrete columns, resulting in a modest bond between the two components. Regardless of the construction sequence, the masonry wall panels are not typically assembled to the full story height. After the frame and walls are assembled, rock or masonry debris is added to fill in the gap between the wall and the bottom of the slab. Figure 5 shows an example of this construction.

**Figure 5.** Infill construction with rubble-filled gap between the masonry wall and reinforced concrete frame.
practice, which results in masonry walls that are not load-bearing. The gravity load is only carried by slender concrete columns.

Masonry walls are typically 8 ft to 10 ft (2.5 m to 3.0 m) high with a single-wythe staggered block arrangement. Walls are constructed directly on top of a finished foundation or floor slab; no mechanical connection is made between the two elements. Walls vary in length from 6 ft to 13 ft (2 m to 4 m) and are commonly bounded by lightly reinforced concrete columns. Wall aspect ratios did not appear to be problematic; most have a height-to-width ratio of less than 1.0.

The slender reinforced concrete columns that bookend the masonry walls are typically 8 in to 12 in (20 cm to 30 cm) wide. Column depth is no less than the masonry unit width. Longitudinal reinforcement usually consists of four #3 or #4 bars. Transverse reinforcement is typically #2 bars, spaced between 6 in and 12 in (15 cm to 30 cm), with no decrease in spacing at column ends. Transverse ties are not bent beyond 90 degrees. Both smooth and deformed reinforcement are used, sometimes in the same structure. The use of smooth bars in new construction was largely abandoned after the year 2000. For future construction of additional levels, longitudinal reinforcement of the columns commonly extends through the slab thickness, but without additional connection detailing.

The two primary roof systems observed were flat slabs and trusses. Slab roofs were sometimes solid reinforced concrete, and in other cases were voided. The lighter roof trusses were constructed of either timber or steel with a light-gauge corrugated metal sheathing. Roof and floor slabs are commonly poured after the wall panels are constructed. Concrete masonry blocks are commonly cast into the slab to minimize the use of concrete (see Figure 6).

Figure 6. Voided slab roof construction with concrete block. Photo courtesy of A. Irfanoglu.
PERFORMANCE OF NONENGINEERED STRUCTURES

When Haiti’s nonengineered buildings were excited by the earthquake, their concrete floor and roof slabs generated large inertial loads. These loads were transferred through the column-slab or beam-column joints, which typically lacked sufficient detailing and transverse reinforcement. Columns lacking in-plane lateral support from adjacent infill walls due to wall damage or openings regularly experienced damage to the column-beam joint, which often resulted in the formation of a plastic hinge without ductile detailing. The moment resistance of the frames was reduced and a racking failure mode ensued, resulting in collapse or residual drift. Figures 7a and 7b illustrate residual drift and structural instability as a result of this failure mode. Lighter roof systems also experienced damage, but much of it was repairable and fell within the life-safety damage measure.

Figure 7. Typical failures of infill masonry construction: (a) Infill wall failure initiated at openings resulting in large rotational demands at beam-column joint, (b) damage to reinforced concrete beam-column joints following failure of masonry infill, (c) common brittle in-plane shear failure (X-cracking) of masonry wall, (d) complete failure of infill panels followed by partial collapse due to poorly confined columns.
Because the masonry walls are typically not load bearing, as previously described, their lateral capacity is reduced due to the lack of friction development between the unreinforced masonry blocks. This frequently resulted in brittle X-cracking of the walls, often initiating at the corner of openings (see Figure 7c). Subsequently, wall panels did not significantly contribute to the overall lateral capacity of structures once they were damaged. As described above, however, the walls reduced the unsupported length of slender columns, improving the in-plane moment resistance of the frames. Localized damage was often observed as a result of in-plane interactions between wall panels and columns. Column shear failures were common. In some cases the column failure resulted in partial or total collapse, as shown in Figure 7d. Further, toppling and out-of-plane wall failures were frequently observed and caused the majority of the complete structural collapses. Even when they didn’t contribute to building collapse, these out-of-plane wall failures caused innumerable injuries and deaths.

There were examples of many of Haiti’s nonengineered structures performed well during the 2010 earthquake. This was not necessarily a result of improved materials, design, construction details, or ground conditions. As discussed above, the construction sequence commonly used resembles Latin American confined masonry, in which the walls are assembled prior to the concrete columns, and results in an improved wall-column bond. The authors observed that structures with little or no damage frequently employed this assembly method. This construction technique is identifiable by the presence of concrete on the faces of the walls and within the crevasses of the masonry wall ends, as shown in Figure 8. Infill masonry, on the other hand, produces a finished, smooth column face with little capacity to bond with adjacent elements. The authors conjecture that the modest bond between the walls and columns of the Latin American-style buildings resulted in increased in-plane shear transfer to the walls, reducing demand on the columns. Moreover, this improved bond permitted the development of an out-of-plane arching action across the walls for increased out-of-plane capacity relative to the capacity of infill masonry.

MATERIAL TESTING

Members of the Earthquake Engineering Research Institute’s (EERI) first and second reconnaissance teams collected construction material specimens for strength testing. This information can provide researchers, designers, and decision makers with a realistic starting point for assessing Haiti’s reconstruction efforts. Justin Marshall (Auburn University) and Steve Baldridge (Baldridge & Associates, Honolulu, Hawaii) secured eight concrete blocks from a collapsed property wall with the assistance of U.S. Southern Command Joint Task Force Haiti Engineers. These blocks were generously tested by the Kaderabek Company of Miami, Florida. The average compressive strength was 1,638 psi (11.3 MPa) with a standard deviation of 365 psi (2.5 MPa). Typical concrete block strength in the United States is 1,900 psi (13.1 MPa). These specimens are likely representative of higher-quality materials available in Haiti. Conversely, a colleague noted that at a rural location outside Port-au-Prince, concrete blocks would fall apart if they were picked up from one end (Holliday 2010). Similarly, Degenkolb engineers noted that some concrete blocks broke apart and shattered when dropped from 4 ft (1.2 m; Rookstool 2010).

Two cylinders of freshly poured concrete were tested in unconfined 28-day compression, in accordance with the American Society for Testing and Materials International’s
ASTM C39, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens (ASTM 2010a). The first concrete cylinder was taken from concrete used in the foundation of a security wall being constructed near the U.S. Embassy. The foundation was sized to accommodate a roughly 10-ft (3-m) high concrete masonry unit (CMU) wall. This work was being performed by a local Haitian contractor for the U.S. Military, with limited supervision. The concrete was cast into a section of PVC pipe in accordance with ASTM standards for concrete strength testing. The sample was field cured for two days before being transported in carry-on baggage to the United States. The sample was then placed in standard wet cure storage for 28 days. The test revealed a peak strength of 1,260 psi (8.7 MPa). The cylinder break was a cone and split fracture type, as defined by ASTM C39. Figure 9 shows the cylinder before and after the testing.

The second fresh concrete sample was taken from concrete used to level the top of a rock rubble foundation for a property separation wall. The sample was gathered in a standard 6 in × 12 in (150 mm × 300 mm) sealed cylinder and transported back to the United States where it was placed in wet cure storage. The sample was tested 28 days after collection. Its peak strength was 410 psi (2.8 MPa). The fracture type was columnar, as defined by ASTM C39 (see Figure 10). The minimum concrete strength allowed for construction in the United States is 3,000 psi (20.7 MPa) in seismic regions.

Figure 8. Concrete on the face and within cavities of the masonry units suggest that the wall was assembled prior to the column, creating a modest bond and improved overall lateral performance.
A forensic concrete specimen was taken from the collapsed Hotel Montana, cut into rectangular prisms, and tested in accordance with ASTM C39, compression testing of cubes. Because of an insufficient aggregate bond, the sample was not cut to the specified 2-in (5-cm) cube. Instead, the specimen dimensions were 2.2 in × 2.3 in × 4.4 in (5.6 cm × 5.8 cm × 11.2 cm). Peak strength was 1,754 psi (12.1 MPa). The result was classified as a combination columnar and shear failure. The test results for concrete and masonry samples are presented in Table 1.

Various sizes of smooth and deformed reinforcing bars were collected and tested in accordance with ASTM A370, Standard Test Methods and Definitions for Mechanical Testing of Steel Products (ASTM 2010b). The largest specimen was extracted from a collapsed Jesuit dormitory in Turgeau. This large three-story structure was considered to be “engineered.” The steel sample was approximately a #5 bar with a peak tensile strength of 66 ksi (455 MPa). Most reinforcing used in nonengineered structures is smooth, small diameter bars, typically #3 and #4. Smaller #2 bars are used transversely. Five small diameter bars were collected from the collapsed Hotel Montana in Pétion-Ville and tested (see Figure 11). Though the Hotel Montana was a large “engineered” structure, the collected steel samples represent those commonly used in residential and nonengineered buildings. The test results shown in Table 2 suggest that the steel reinforcement was Grade 40 (275 MPa) or higher.

Steel reinforcement is imported into Haiti in large, tightly bound coils. Before use, the steel may undergo up to three large deformations. A machine first straightens 30-ft sections from the coils. These pieces are then bent in half for transportation. Once on-site, the reinforcement is cut and bent again into workable geometries. Dr. Andre Filiatrault, Multidisciplinary Center for Earthquake Engineering Research (MCEER) Director and professor at the University at Buffalo, the State University of New York, secured three deformed steel reinforcement samples with a nominal diameter of 0.5 in (13 mm) from a construction site in Haiti (Filiatrault 2011). Two of the three specimens had been bent for transportation purposes and were re-straightened at the construction site. The specimens were tested in accordance with ASTM A370; the results are shown in Table 3. It appears that the steel was
Grade 60. The approximate yield and ultimate tensile strength values of the two bent specimens were not significantly different to the unbent specimen. However, it should be noted that ultimate strain values were reduced by 50% and 82% as a result of bending.

The data shown in Tables 2 and 3 suggest that the strength of steel reinforcement in Haiti was not a contributing factor to failures. Instead, insufficient steel ratios, smooth bars, and improper detailing contributed to collapses.

Figure 10. Fractured concrete specimen taken from concrete used to level the top of a foundation.
Table 1. Summary of specimen testing results of concrete and concrete masonry blocks

<table>
<thead>
<tr>
<th>Specimen Type</th>
<th>Date Obtained</th>
<th>Specimen Description</th>
<th>Test Type</th>
<th>Peak Value (psi) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Block</td>
<td>Feb. 2010</td>
<td>Eight concrete masonry blocks for property wall</td>
<td>Compression</td>
<td>1,638 ± 365[^a^] [11.3 ± 2.5]</td>
</tr>
<tr>
<td>Fresh Concrete</td>
<td>March 2010</td>
<td>Leveling concrete for foundation top</td>
<td>28-day Cylinder</td>
<td>410 [2.8]</td>
</tr>
<tr>
<td>Fresh Concrete</td>
<td>Feb. 2010</td>
<td>Foundation of U.S. Military facility</td>
<td>28-day Cylinder</td>
<td>1,260 [8.7]</td>
</tr>
<tr>
<td>Extracted Concrete</td>
<td>Feb. 2010</td>
<td>Collapsed Hotel Montana, Pétion-Ville, Haiti</td>
<td>ASTM C109 Cube</td>
<td>1,754 [12.1]</td>
</tr>
</tbody>
</table>

[^a^]Average value of eight specimens with standard deviation.

Figure 11. Steel samples collected from the Hotel Montana.
CASE STUDIES

A series of case studies is presented to highlight the causes and effects of Haiti’s weak infrastructure. Each case study emphasizes a different flaw in the materials, planning, construction, or engineering that resulted in failure.

UNIVERSITÉ INTERNATIONALE D’HAIŢI BUILDING

The Université Internationale d’Haiti (UNIH) building sat alone along Route Nationale No. 2 about 3.75 miles (6 km) east of Léogâne. The building had not been completed at the time of the earthquake. Prior to the event it was three stories tall with column reinforcing steel protruding from the top deck in preparation for a fourth floor. There were no signs of recent construction activity at the site, indicating that the project may have slowed down or stalled. While there were concrete frames in each direction, the building still suffered catastrophic total collapse. The building was relatively simple and repetitive in plan, measuring approximately 61 ft × 158 ft (18.5 m × 48 m). The short direction had three 16.5-ft (5-m) bays with a 5.75-ft (1.75-m) cantilever at each end. The long direction had nine 16.25-ft

Table 2. Summary of specimen testing results of steel reinforcement in tension

<table>
<thead>
<tr>
<th>Type</th>
<th>Diameter (in)</th>
<th>Nearest Size</th>
<th>Specimen Description</th>
<th>Approx Yield (ksi)</th>
<th>Peak Tensile Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformed</td>
<td>0.630 [16.0]</td>
<td>#5 Imperial 16 Metric</td>
<td>Extracted from collapsed Jesuit residence, Turgeau, Haiti</td>
<td>43 [297]</td>
<td>66 [455]</td>
</tr>
<tr>
<td>Deformed</td>
<td>0.307 [7.8]</td>
<td>#2–#3 Imperial 8 Metric</td>
<td>Extracted from nonengineered building site</td>
<td>53 [365]</td>
<td>81.7 [563]</td>
</tr>
<tr>
<td>Smooth</td>
<td>0.326 [8.3]</td>
<td>#2–#3 Imperial 8 Metric</td>
<td>Extracted from nonengineered building site</td>
<td>57 [393]</td>
<td>76.8 [530]</td>
</tr>
<tr>
<td>Smooth⁴</td>
<td>0.462 ± 0.003</td>
<td>#3–#4 Imperial 12 Metric</td>
<td>Extracted from nonengineered building site</td>
<td>52 ± 4</td>
<td>72.0 ± 1.1</td>
</tr>
</tbody>
</table>

⁴Average of three values displayed with standard deviations.

Table 3. Summary of specimen testing results of steel reinforcement in tension

<table>
<thead>
<tr>
<th>Type</th>
<th>Min Diameter (in)</th>
<th>Nearest Standard Size</th>
<th>Approx Yield Strength (ksi) [MPa]</th>
<th>Ultimate Tensile Strength (ksi) [MPa]</th>
<th>Ultimate Tensile Strain (%)</th>
</tr>
</thead>
</table>

RECIPE FOR DISASTER
(4.9-m) bays with a 5.75-ft (1.75-m) cantilever at each end. Measurements were taken of both the concrete and reinforcing steel elements. The damage was severe enough to expose both the column and beam steel at the lowest level.

The structural floor framing included a grid of 10-in × 14-in (25-cm × 35-cm) beams in each direction supporting a 5-in (13-cm) thick slab. The beams were reinforced with three #5 bars top and bottom and #2 stirrups at 9 in (23 cm) on center. The #5 bars were hooked at the ends of the beam with 90 degree bends and only a 3-in (7.5-cm) hook length, substantially shorter than required by the American Concrete Institute (ACI) standards. The typical columns were 10 in × 10 in (25 cm × 25 cm) in plan and approximately 12 ft (3.7 m) tall. Typical interior column reinforcement included four #5 vertical bars and #2 column ties at 9 in (23 cm) on center. Typical corner column reinforcement included four #4 vertical bars and #2 column ties at 8 in (20 cm) on center. In some columns, the vertical and tie reinforcement consisted of smooth bars. Typical column ties had nonseismic 90-degree hooks in lieu of the 135-degree hooks required by the ACI standards.

An investigation of this building design was completed to compare the as-built conditions to a rigorous design. The seismic hazard was determined based on spectral acceleration values from the Unified Facilities Criteria 3-310-04 Seismic Design for Buildings (U.S. Department of Defense 2010). The short period and 1-second spectral accelerations for the site were 0.8 g and 0.32 g, respectively. Site Class C was used in the design, based on field observations. Since the hazard would be classified as Seismic Design Category D and special detailing would be required, a response modification factor (R) of 8.0 was used to determine the seismic demand. A model of the building frame made using the building design software ETABS indicated that the frame, without any contribution from the masonry infill, would work for gravity loads, but would be highly overstressed for seismic loads. The demand-to-capacity ratios, assuming a 3,000 psi (20.7 MPa) concrete-under-gravity load, only range from 0.64 at the corners to 1.03 at the interior columns. Under lateral loads, the ratios range from 4.5 at the corners to 7.3 at the exterior columns on the short side. Considering that the actual reinforcement does not meet ordinary detailing requirements, the demand-to-capacity ratios with an R of 3.0 would be significantly higher. Figure 12 shows the collapsed elements and the exposed construction details. This case study highlights the fact that for a structure as important as a university, rigorous design procedures accounting for lateral deformation and force demands were not utilized.

“ENGINEERED” SCHOOL BUILDING

Several examples of buildings engineered to U.S. codes were investigated. A three-story reinforced concrete frame school building was inspected which had construction drawings indicating design in accordance with ACI Standard 318-89, Building Code Requirements for Structural Concrete and Commentary (ACI 1989). The drawings included frame elevations with seismic detailing. The details included tight tie spacing in hinge zones, continuity of beam reinforcing (top and bottom), and code-compliant column vertical reinforcement. The drawings also indicated details for isolating the masonry partition walls from the concrete frames to ensure ductile behavior and limit masonry cracking. There was, however, damage to the columns, typically in the middle third, where flexural demands were low, and throughout many of the partition walls. A majority of the observed damage
appeared to be the result of a lack of quality control for materials and a lack of the oversight needed to ensure conformity to the construction document details. Two notable discrepancies were column ties that did not match the seismic ties indicated in the drawings (90-degree versus 135-degree hooks) and partition walls that were not isolated from the frames. Figure 13 shows the damage that occurred, highlighting the lack of attention to critical details during construction. This study illustrates the need for building officials to ensure that the as-built structure conforms to the well-detailed design documents. Looking at the other side of the coin, the U.S. Embassy was designed and constructed to the most current U.S. seismic requirements when it was completed in 2008. Because construction was controlled, the only damages to the Embassy were displaced ceiling tiles and popped rubber seals on a few windows.

HANDS TOGETHER SCHOOLS

Many buildings in Haiti are combinations of multiple structures of different shapes, roof heights, and number of stories. This results from the economic necessity of using existing structures as components for new buildings. This method can be effective if appropriate
construction details and structural designs are implemented to account for additional forces on the existing structure and dynamic differences between the building sections. This can be accomplished through strengthening or providing seismic expansion joints to allow individual response. A typical practice in Haiti is to use an existing wall for a new structure, thereby reducing construction costs. This practice was commonly observed during reconnaissance missions. The post-event detailed structural inspection of eight school sites and one medical clinic run by Hands Together, a nonprofit organization based in the United States, provided multiple examples of this situation. All the Hands Together sites are in Cite Soleil.

The St. Francois compound, located at 18.5871° N 72.3276° W, is a high school. It consists of six buildings, many of which are connected to one or more adjacent structures. A high-resolution Google Earth image of the compound is shown in Figure 14. The letters shown in the figure are used to reference the different buildings. All of the campus buildings are constructed of reinforced concrete frames with masonry infill. Building A is a two-story classroom structure. It experienced very limited damage during the earthquake in the form of masonry wall cracking and damage to the second story stairway. It is the only structure not connected to another building. The first two stories of Building B are enclosed, while the third floor is a covered patio supported by reinforced concrete columns and a half-height masonry wall. Building B experienced some wall cracking. The most significant damage occurred in the stairwell that connects Buildings B and D at the second and third stories. The damage to the second floor slab is visible in Figure 14. Due to the difference in orientation of Buildings B and D, the stairwell became the fuse element between the two structures and was severely damaged.

Building C, the newest in the complex, is a two-story classroom building. Future upward expansion is expected as reinforcing cages above the roof slab were visible. The building was minimally damaged in the form of cracked masonry around the windows and doors. The only significant damage was at the interface with Building D, the largest building in the complex. The bottom two stories of Building D are enclosed classrooms. The top

![Figure 13](image1.jpg)

Figure 13. “Engineered” three-story reinforced concrete frame school building: (a) Column damage showing the lack of 135 degree hooks in column ties, (b) masonry wall damage indicating extremely poor mortar material properties.
**Figure 14.** Google Earth image of St. Francois School complex.

**Figure 15.** St. Francois School complex: (a) Damage to stairwell between Buildings B and D, (b) third floor slab damage between Buildings D and E.
floor is a covered patio with reinforced concrete columns. It is attached at both ends to the adjacent structures. This was the most seriously damaged building in the complex. The significant difference in plan dimensions resulted in damage at the interface between Buildings D and E, as shown in Figure 15b. Building D also experienced significant cracking of masonry walls and serious damage to reinforced concrete columns and beams supporting the third floor timber trussed roof. Some of the damage was due to the omission of every other block just below the slab to allow for lighting in the classrooms. The early fracture of single blocks transferred the load to other internal walls, which also had large voids. Buildings E and F experienced minimal damage, with the exception of the previously mentioned damage at the joint between D and E. Overall, the compound performed well. Had the structures not been linked together, the most significant damage could have been avoided.

This case study highlights a high-risk building practice that is prevalent in Haiti. The practice of building onto existing structures without considering transfer forces or providing a movement joint resulted in a significant amount of damage. In this case, it was not fatal, however the repair costs are significant and there were surely cases where this type of practice resulted in earthquake fatalities.

CONCLUSION

Haiti’s infrastructure was completely unprepared for the earthquake that struck on 12 January 2010. With rare exceptions, buildings were not planned, designed, or constructed with the necessary detailing to survive seismic events. While some buildings may have been engineered, the necessary channels were not in place to verify compliance during construction. The material testing results and case studies presented here highlight many of these deficiencies. In order to avert a disaster of this magnitude in the future, the people of Haiti must improve construction material quality, planning, engineering design, and construction of all infrastructure. This lesson is critically important during the recovery and rebuilding phase.

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