APRIL 16, 2016 - MUISNE, MANABI EARTHQUAKE, ECUADOR

Surveyed May 01-06, 2016

Build Change Post-Disaster Reconnaissance Report

International Headquarters
535 16th St., Suite 605
Denver, CO 80202 USA
www.buildchange.org

Contents

1. BACKGROUND

Ecuador is located in northwestern South America, bordered by Colombia on the north, Peru on the east and south, and the Pacific Ocean to the west. Ecuador also includes the Galápagos Islands in the Pacific, about 1,000 kilometers (620 mi) west of the mainland.

There was an M7.8 earthquake registered on Saturday, April 16th at 18h58m (local time), 2016 in which the hypocenter was located near Pedernales, Ecuador (Manabí), 20 km deep. This earthquake was the result of displacement between two tectonic plates: the Nazca plate (oceanic plate) that plunges under the South American (continental plate), as shown in Figure 1.
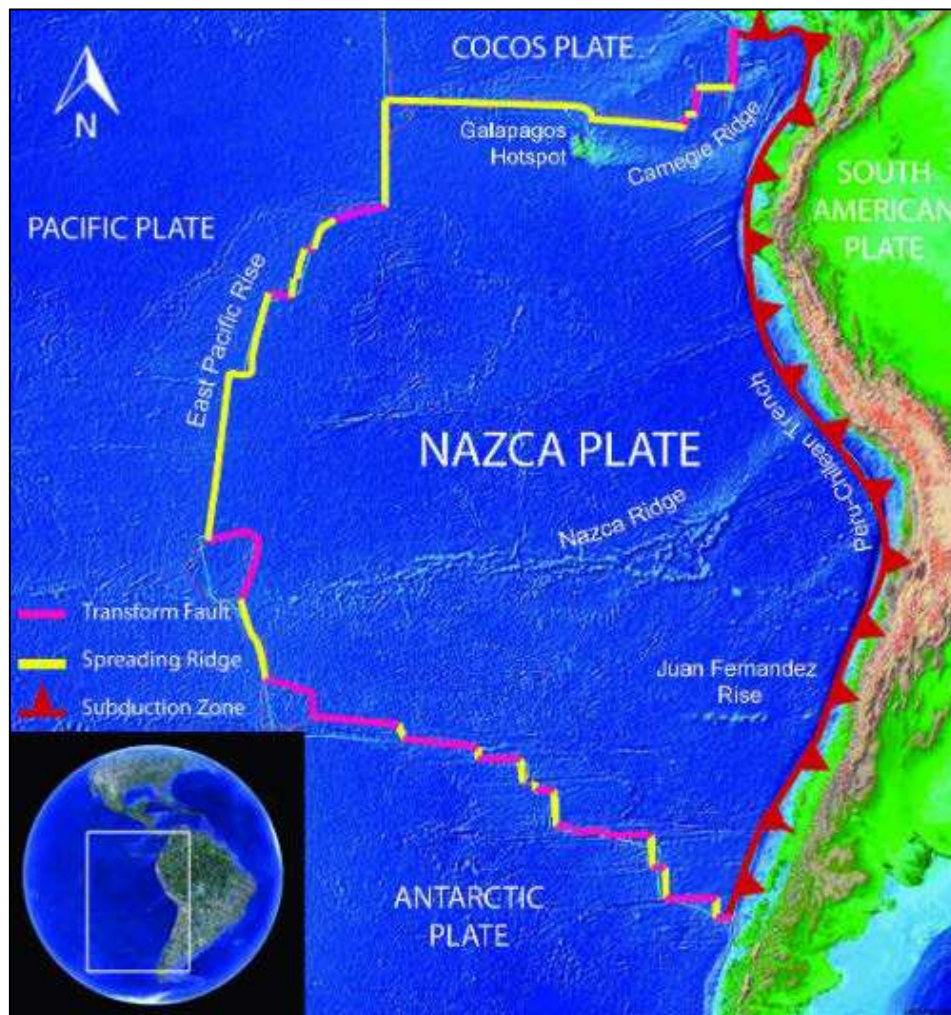

Figure 1. Interaction between Nazca Plate and South American Plate – Subduction Area
Source: http://caribbeantectonics.weebly.com/plates.html

Historically, Ecuador has suffered the effects of several registered large quakes:

- January 31st 1906 (M8.8), which is the largest recorded in Ecuador and the sixth largest globally.
- May 14th, 1942 (M7.8)
- January 19th, 1958 (M7.8)
- December 12th, 1979 (M8.1).

1.1 Visit Objectives

Following the earthquake, Build Change Latin America, based in Bogota, Colombia deployed a small reconnaissance team, with the following objectives:

1. Visit several of the main towns affected by the M7.8 Earthquake along the western coast of Ecuador: Manta, Canoa, Crucita, Jama and, Portoviejo.

2. Identify the main types of existing structures for housing and schools, especially those that suffered damages.

3. Identify the most common damages and failure modes that appeared in the building structures to improve understanding of the seismic vulnerability of the common building types.

4. Gain understanding of the distribution of damages throughout the affected region (concentrated or dispersed).

5. Survey homeowners and builders within the affected areas, in order to identify common construction practices and preferences.

6. Deliver presentations for two universities and surrounding communities regarding Build Change's experiences in housing reconstruction programs and also technical observations from damaged buildings during the recent earthquake.

1.2 Survey Team

Build Change's reconnaissance team included the following members:

- Lizzie Blaisdell (Civil/Structural Engineer): Build Change Director of Engineering, in charge of quality supervision and consistency of Build Change's technical and project solutions.

- Juan Caballero (Architect): Director of Programs and Partnerships in Latin America, in charge of leading the regional office in Bogota, and in charge of projects in Colombia, Guatemala, and other regional ventures.

- Walter Armando Cano Alvarado (Civil Engineer): Project Engineer for preventive retrofitting program in Colombia.

2. THE EARTHQUAKE

From USGS (http://earthquake-report.com/):

Magnitude: 7.8
Local Time (conversion only below land): 2016-04-16 18:58:36
GMT/UTC Time: 2016-04-16 23:58:36
Depth (Hypocenter): 20 km

27.0 km (16.8 mi) SSE of Muisne, Ecuador
52.0 km (32.3 mi) W of Rosa Zarate, Ecuador
68.0 km (42.3 mi) SSW of Propicia, Ecuador
111.0 km (69.0 mi) NW of Santo Domingo de los Colorados, Ecuador
170.0 km (105.6 mi) WNW of Quito, Ecuador
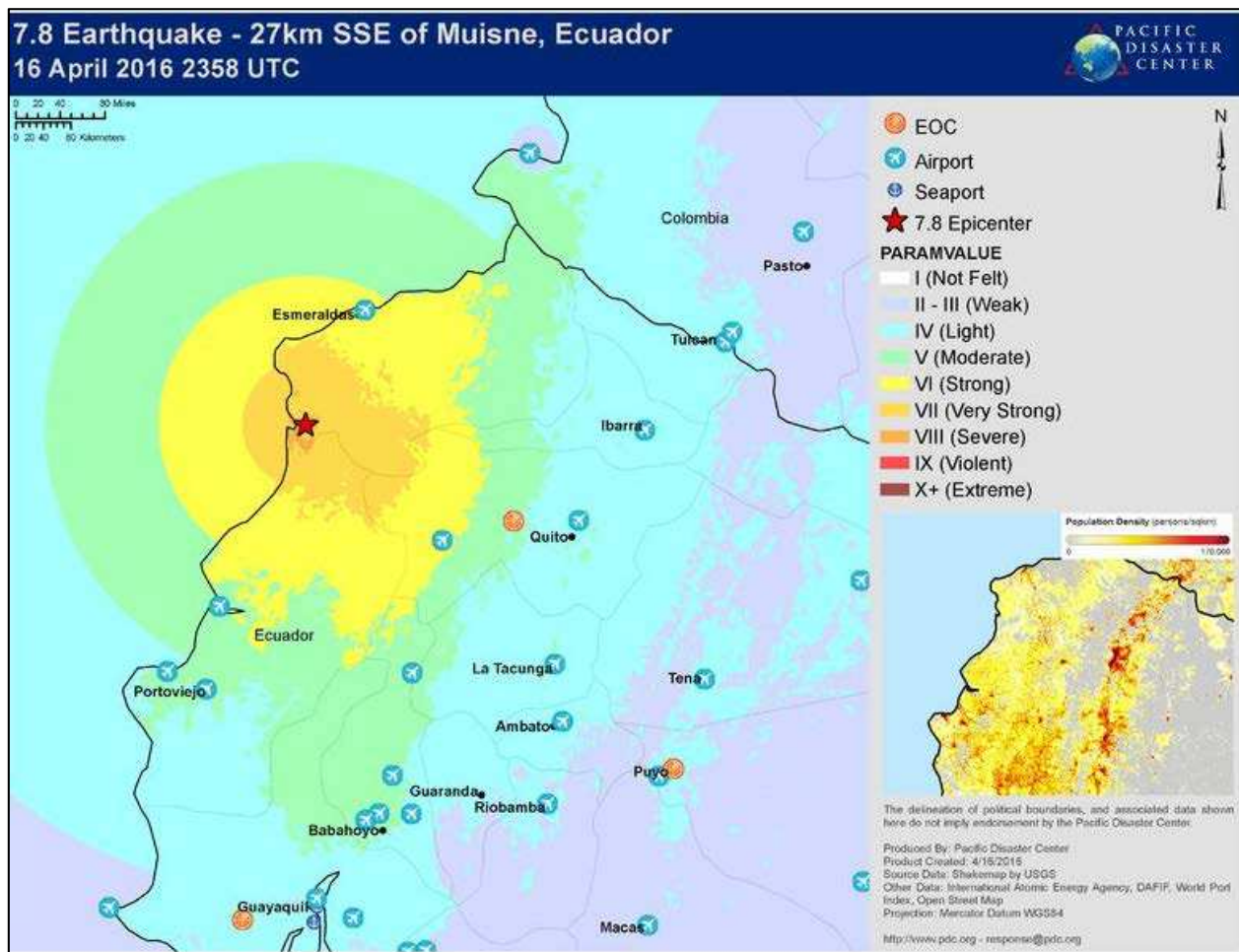
Figure 2. Epicenter of M7.8 Earthquake – April 16th, 2016.
Source: Pacific Disaster Center

USGS ShakeMap : NEAR THE COAST OF ECUADOR
Apr 16, 2016 23:58:37 UTC M 7.8 N0.37 W79.94 Depth: 19.2km ID:us20005j32

Map Version 5 Processed 2016-04-17 03:05:33 UTC
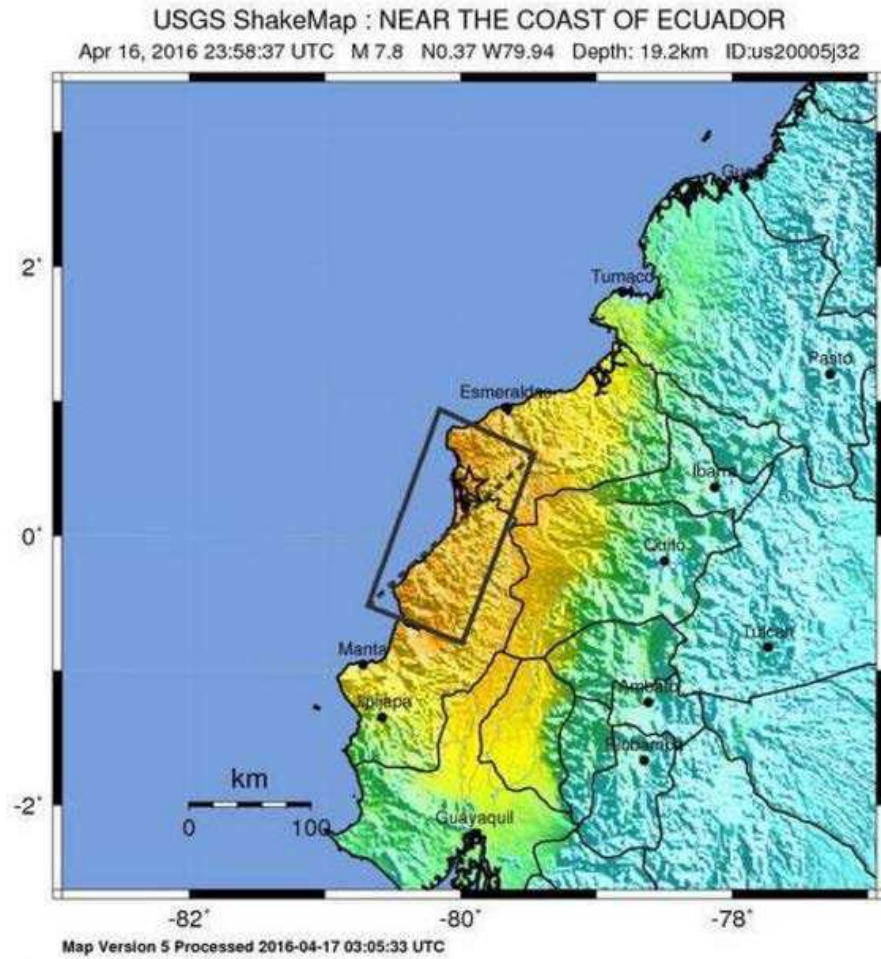
PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	<0.02	0.1	1.4	4.7	9.6	20	41	86	>178
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Worden et al. (2012)

Figure 3. Location and intensity of the Earthquake
Source: USGS, 2016

Since the Earthquake on April 16th, there have been recent shallow aftershocks up to M6.9 magnitude, near the coast of Ecuador.

2.1. Most Affected Areas

The most affected coastal areas were the Provinces of Manabi and Esmeraldas, which reported the most deaths and injuries. In addition there were other areas, such as Guayas, Santo Domingo de los Tsachilas, Pichincha and Chimborazo, which also reported deaths and injuries. See Figures 4 and 5.

Figure 4. Reported deaths - Ecuador
Source: Pacific Disaster Center / wfp.org / http://www.telesurtv.net

Figure 5. Reported injuries - Ecuador
Source: Pacific Disaster Center / http://www.telesurtv.net
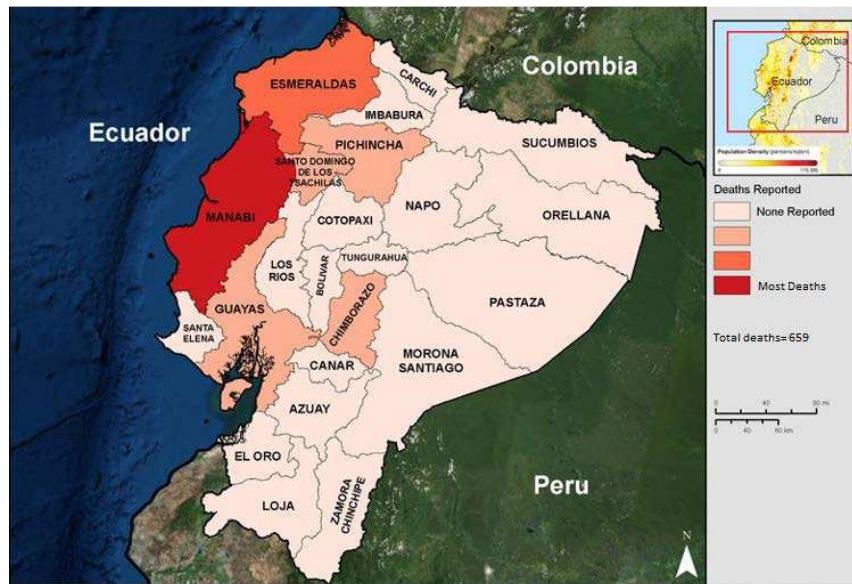
The most affected province was Manabi, which reported the most damaged and collapsed buildings.

Figure 6. Destroyed and damaged buildings - Ecuador
Source: Pacific Disaster Center / OCHA, United Nations

As a result of the earthquake, it was reported that in total 663 people were killed, 12 went missing, and over 4,800 were injured. Additionally, an estimated 80,000 people were displaced due to damaged or collapsed homes. Approximately 30,000 homes were damaged or destroyed, with a nearly 50/50 split between urban (13,962) and rural (15,710) housing. 875 schools were affected by the earthquake.

2.2. Design Ground Acceleration Parameters

The map for seismic design included within the Ecuadorian Building Code (NEC15) shows that the most affected area falls within the very high (VI) seismic hazard zone, with design ground accelerations greater than 0.50g. See Figure 7.

Seismic zoning
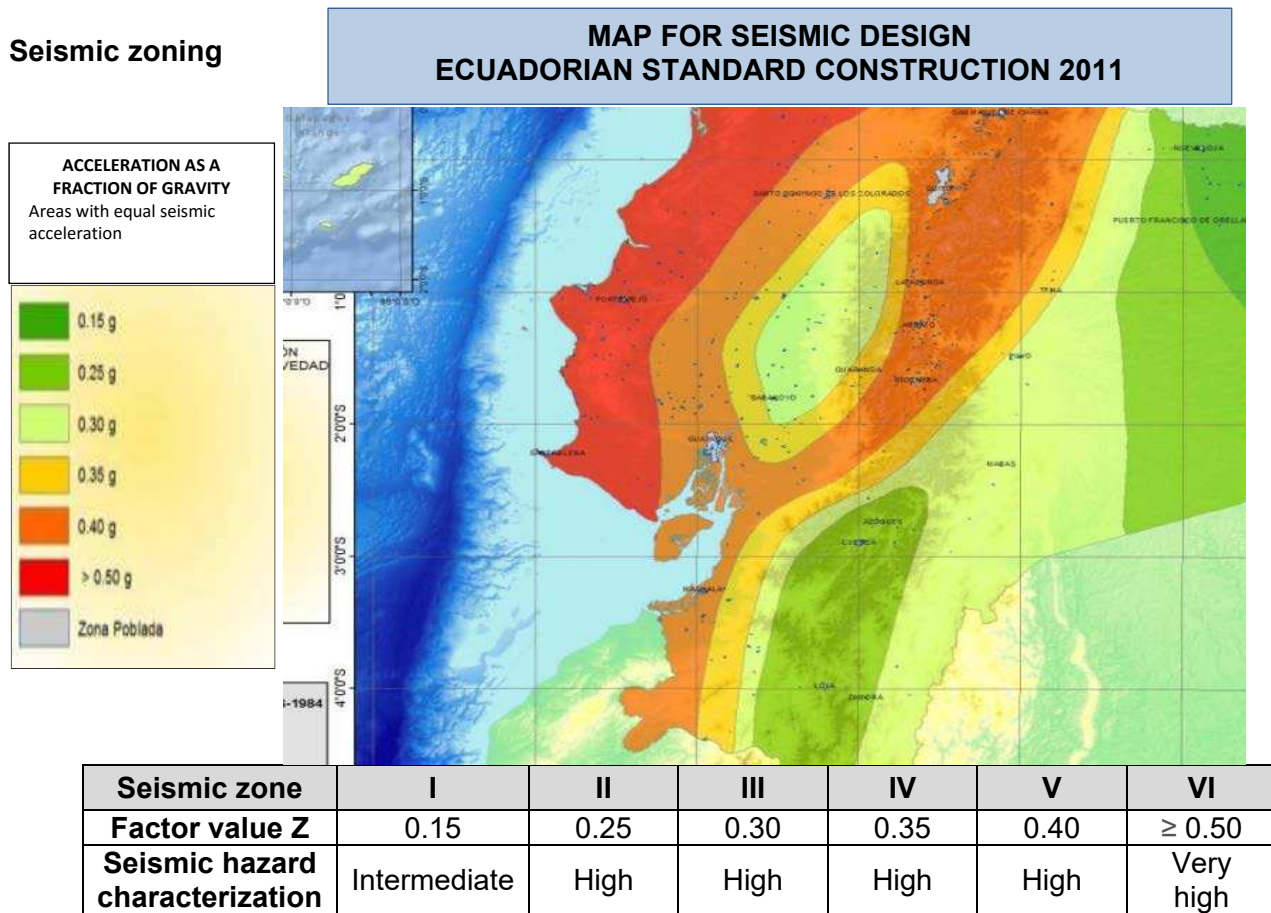
Seismic zone	I	II	III	IV	V	VI
Factor value Z	0.15	0.25	0.30	0.35	0.40	≥ 0.50
Seismic hazard characterization	Intermediate	High	High	High	High	Very high

Z = Expected maximum acceleration in rock for the design earthquake, expressed as a fraction of the acceleration of gravity (g).

Figure 7. Ground Acceleration - Design Map
Source: Ecuadorian Building Code (NEC15)

According to NEC15, the structural design for structures is made for the Design Earthquake that has a 10% probability of being exceeded in 50 years, equivalent to a return period of 475 years. The design spectrum can be determined according to the following in NEC15. See Figure 8.
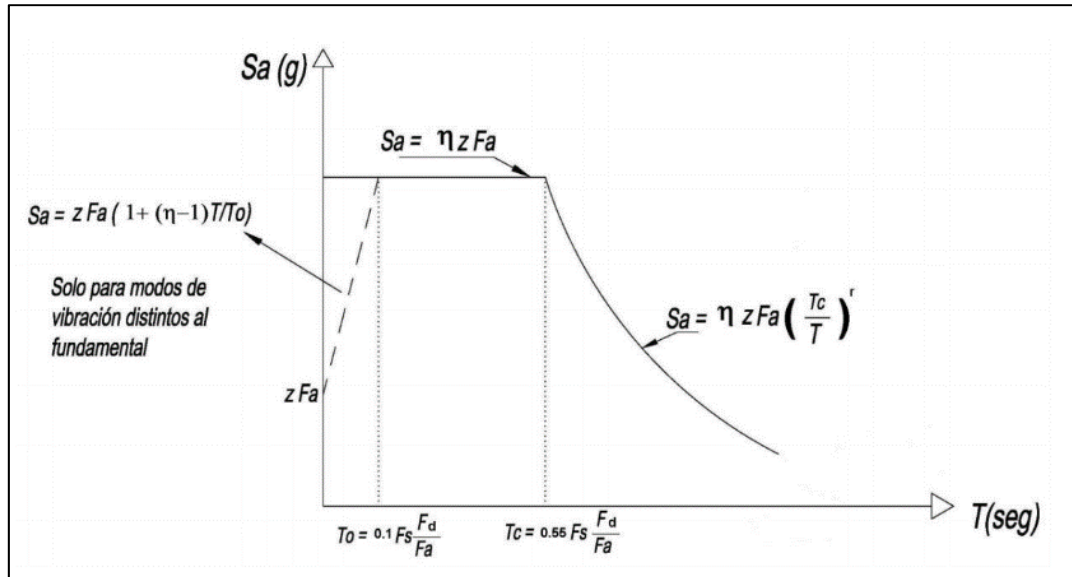
Figure 8. Design Response Spectrum
Source: Ecuadorian Building Code (NEC)

η = Ratio of the spectral acceleration Sa (T = 0.1 s) and the PGA (Value of the expected seismic acceleration rock) for selected return period.

η = 1.80 - Coast provinces (except Esmeraldas).

η = 2.48 - Provinces: Sierra, Esmeraldas and Galapagos Islands.

η = 2.60 - East provinces.

Z = Expected maximum acceleration in rock for the design earthquake, expressed as a fraction of the acceleration of gravity (g).

S_a = Elastic response spectrum of accelerations (expressed as a fraction of the acceleration of gravity g).

r = Factor used in the Elastic Design Spectrum, which values depend on the location of the building.

$r = 1$ - All types of soil, except soil type E.

$r = 1.5$ - Soil E

T = Fundamental period of vibration of the structure.

T_o = Vibration limit period in the elastic seismic acceleration spectrum, representing the design earthquake.

T_c = Vibration limit period in the elastic seismic acceleration spectrum, representing the design earthquake

F_a **=** Soil amplification coefficient in the short period zone. Amplifies the Elastic Acceleration Design Spectrum for design over Rock, considering the site effects.

F_d **=** Soil amplification coefficient. Amplifies the Elastic Design Spectrum for design over Rock, considering the site effects.

F_s **=** Soil amplification coefficient. It considers the nonlinear behavior of soils, degradation of site period that depends on the intensity and frequency of seismic movement and relative displacements of soil.

As an example, the design acceleration, S_a, for short period buildings in the coastal areas affected (excluding Esmeraldas) would be 1.0g, assuming soil type D, and the location in Pedernales – Manabí Province.

2.3. Recorded Accelerations

The Geophysics Institute (*Instituto Geofísico*) from Ecuador has retrieved information from several stations that have no real-time transmission, located across the country, and detailed with circles on the map of Figure 9. These stations are part of the National Accelerographs Network (RENAC) owned by Geophysics Institute. In addition, there are values for the maximum amplitude (Peak Ground Acceleration - PGA) from 7 stations owned by Heavy Crude Oil Pipeline Company (OCP).

Figure 9 shows the locations and available recorded values of PGA from the RENAC Network, for the three main directions. The highest accelerations were recorded in Pedernales, to the south of the epicenter (Black Star). The values in Manta, Portoviejo and Chone are larger than the accelerations recorded in Esmeraldas, even though Esmeraldas is closer to the epicenter.

In Pedernales (APED station) a maximum value of 13.803 m / s^2 (1.407 g) was recorded. In Figure 10, an important difference is observed between the waveforms from the southern stations WTSA, ACHN, APO2 and the stations located in the north LGCB (From ADN Network), AES2. In the north, the peak acceleration values are lower but with longer strong shaking duration, while in the south, higher peak accelerations observed but with a shorter strong shaking duration.

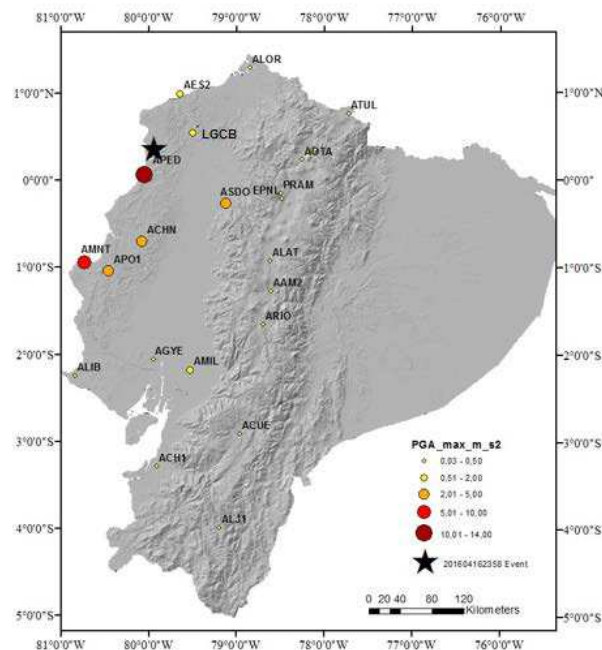

Figure 9. Location of the stations by the National Accelerographs Network and OCP network - Seismic accelerations. *Source: http://www.igepn.edu.ec/*

Location and acceleration data by station are summarized in Table 1. The PGA values are shown for each orthogonal direction.

RED	Estación	Latitud	Longitud	Altitud (m)	R_{epi} (km)*	PGA E (m/s²)	PGA N (m/s²)	PGA Z (m/s²)
RENAC	AAM2	-1,269	-78,611	2664	235	0,2548	0,3465	0,1465
	ACH1	-3,287	-79,910	13	407	0,2501	0,2365	0,0805
	ACHN	-0,698	-80,084	18	120	3,2349	3,6266	1,7287
	ACUE	-2,910	-78,959	2578	381	0,3419	0,2896	0,1748
	AES2	0,991	-79,646	4	76	1,5120	1,0831	0,4314
	AGYE	-2,054	-79,952	30	270	0,1832	0,2304	0,1462
	AIB1	0,347	-78,125	2208	202	0,4839	0,5682	0,1223
	AIB2	0,349	-78,106	2298	204	0,2080	0,3246	0,0839
	ALAT	-0,926	-78,618	2777	206	0,3104	0,2738	0,1191
	ALIB	-2,243	-80,846	40	308	0,4113	0,3895	0,2066
	ALJ1	-3,987	-79,197	2147	492	0,1490	0,1595	0,0882
	ALOR	1,293	-78,847	22	159	0,2564	0,2623	0,1512
	AMIL	-2,181	-79,529	20	288	0,5103	0,4504	0,1836
	AMNT	-0,941	-80,735	38	171	3,9720	5,1421	1,6213
	AOTA	0,240	-78,256	2529	188	0,4203	0,3441	0,1822
	APED	0,068	-80,057	15	36	13,8034	8,3187	7,2734
	APO1	-1,038	-80,460	47	167	3,1150	3,7303	1,0184
	ASDO	-0,263	-79,124	615	115	2,0223	1,0884	0,4988
	ATUL	0,772	-77,723	3097	251	0,1598	0,2065	0,0664
	EPNL	-0,212	-78,492	2813	174	0,2632	0,1987	0,1307
	PRAM	-0,145	-78,495	2840	171	0,2458	0,2290	0,1318
ADN**	LGCB	0,382	-79,575	160	41	0,8338	0,6888	0,4279
	PDNS	0,111	-79,991	442	29	10,4085	9,5497	5,6183
OCP**	AMA1	0,935	-79,725	234	67	4,1746	1,9870	1,3195
	APR1	-0,046	-78,614	2981	155	0,0050	0,0036	0,0021
	APR2	0,077	-78,968	804	113	0,9911	1,0767	0,5544
	APS4	-0,371	-78,106	2887	220	0,0691	0,0633	0,0236
	AV11	-0,073	-78,371	2058	181	0,2930	0,3839	0,1961
	AV18	0,313	-79,478	107	52	1,4142	1,2913	0,6758
	AV21	0,661	-79,547	62	54	1,8957	1,4723	0,8941

'* Repi= Distancia Epicentral/
'** Proyectos de cooperación.

**Repi = Distance to Epicenter

***Cooperation Projects

Table 1. PGA Values (m / s²) for each direction from recorded stations.
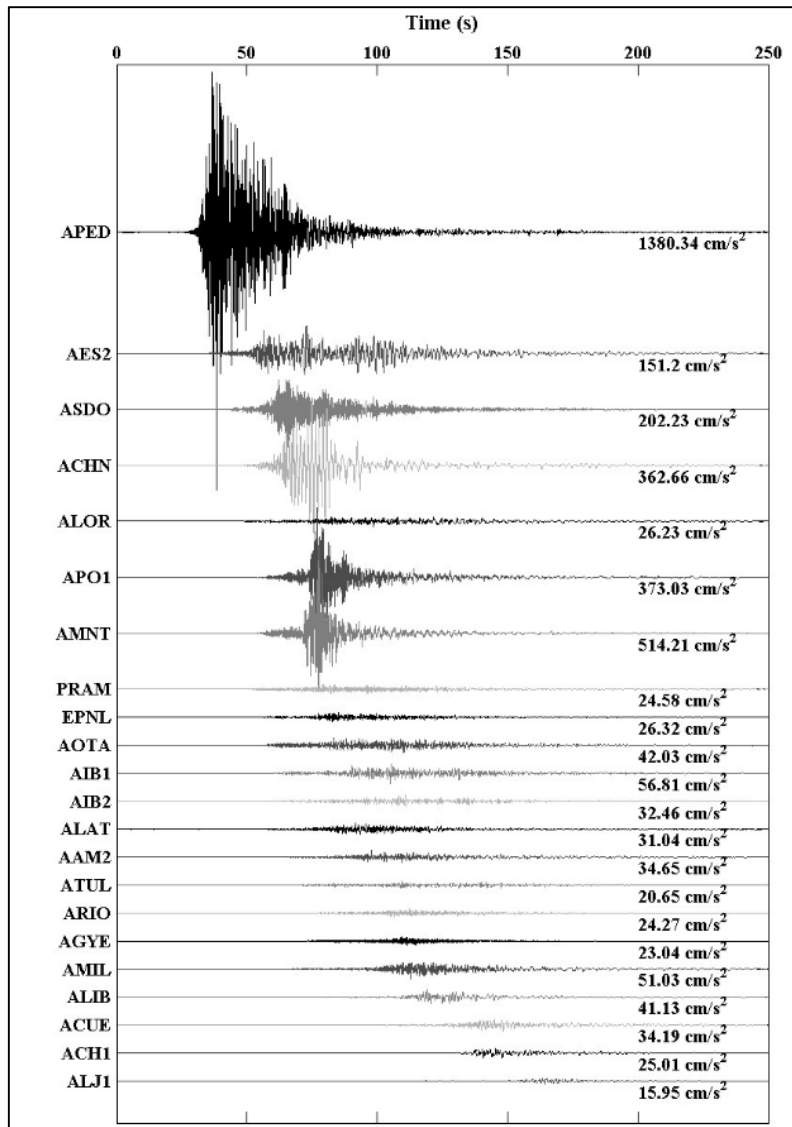*Source: http://www.igepn.edu.ec/*

Figure 10. M7.8 Earthquake - 16th April 2016, ordered according to the distance to Epicenter
Source: http://www.igepn.edu.ec/

Figures 11 and 12 show the records of two stations in Manabi Province. The recorded accelerations for two horizontal directions are compared with the Response Spectrum from the Ecuadorian Building Code (NEC15). For Manta and Portoviejo stations, the recorded response spectra are typically lower than those of the NEC15 design spectra for a Soil Type D. Also, the E-W and N-S records are similar in strength and do not indicate significant directionality effects.
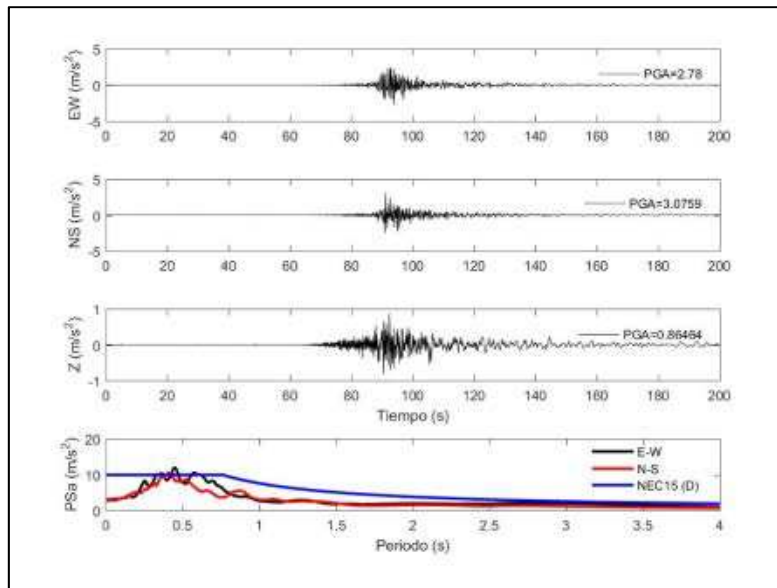
Figure 11. Record of acceleration and response spectrum for Portoviejo station.
Source: http://www.igepn.edu.ec/
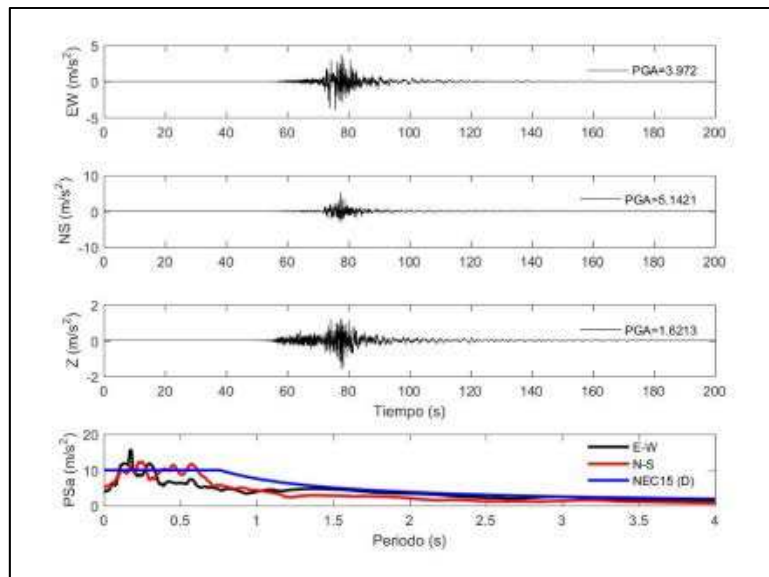
Figure 12. Record of acceleration and response spectrum for Manta station.
Source: http://www.igepn.edu.ec/

3. VISITS TO AFFECTED AREAS

The survey team visited several towns within the Manabi province, in order to observe buildings, particularly houses and schools. The towns visited were Manta, Montecristi, Canoa, Portoviejo, Crucita and Jama. See Figure 13.
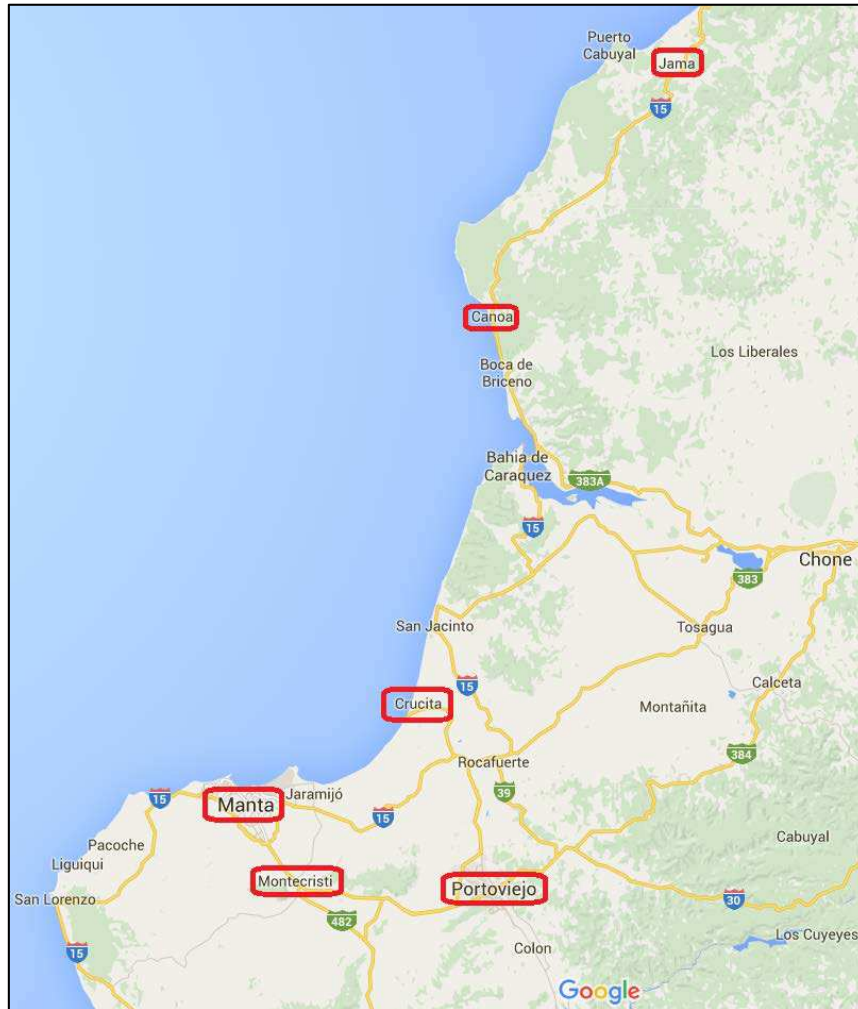

Figure 13. Visited Towns across Pacific Coast - Ecuador
Source: Google Maps

3.1 Guayaquil to Manta - May 1st, 2016

The team traveled from Guayaquil, through Montecristi to Manta, performing quick "windshield" surveys of the towns. More detailed information about the building types and damages observed can be found in the corresponding blog post for the day at http://www.buildchange.org/2016/05/01/responding-to-the-earthquake-in-ecuador-day-1/.

3.2 Canoa and Jama - May 2nd, 2016

The team traveled with local engineer, Mr. Hermel Flores, north up the coast, stopping for more detailed surveys in the towns of Canoa and Jama. Houses and schools were visited. Additional information about the activities and observations of the day can be found in the two corresponding blog posts for the day at http://www.buildchange.org/2016/05/03/day-2-canoa-and-pedernales-a-first-look-at-damaged-schools/ and http://www.buildchange.org/2016/05/03/day-2-part-2-canoa-and-jama-analysis-of-damaged-homes-and-schools/.

3.3 Tarqui and Manta - May 3rd, 2016

The team was escorted around the barricaded area of Manta, the neighborhood of Tarqui, which had concentrations of damaged buildings higher than surrounding areas. In the evening, the team with Mr. Flores presented a seminar at Secular University "Eloy Alfaro" of Manabi, in Manta, about experiences in housing reconstruction following other earthquakes and the Construction Code in Ecuador (Fig. 14).

Figure 14. Seminar at Secular University "Eloy Alfaro" of Manabi, ULEAM

Additional information on the observations in Manta and Tarqui as well as the seminar, can be found in the corresponding blog post for the day at http://www.buildchange.org/2016/05/05/day-3-4-manta-crucita-and-portoviejo-a-look-at-mixed-use-buildings-and-sharing-knowledge/

3.4 Crucita and Portoviejo - May 4th, 2016

The team visited Crucita, a coastal town and the area of Portoviejo with the higher concentration of damages. In the afternoon, the team presented a similar seminar as the previous night with Eng. Flores, but at the Technical University of Manabi in Portoviejo (Fig. 15). Additional information on the day can be found in the blog post at http://www.buildchange.org/2016/05/05/day-3-4-manta-crucita-and-portoviejo-a-look-at-mixed-use-buildings-and-sharing-knowledge/

Figure 15. Seminar at Technical University of Manabi

3.5 Los Esteros, Manta - May 5th, 2016

Visits by the team concluded in the neighborhood of Los Esteros in Manta. It is near the hard-hit area of Tarqui, but is largely residential. The damaged buildings seemed to primarily be those that were deteriorated, mixed wood and masonry houses, or buildings that were four or more stories.

4. BUILDING TYPES AND CONSTRUCTION OBSERVED

4.1 Housing

For housing there were several types of construction systems observed:

- Concrete frames, typically with masonry infill
- Timber frames, with masonry infill
- Timber frames, with timber or lath panels (lightweight)
- Metal frames, with masonry infill
- Combined systems

Below, each system is described and examples are presented.

- **Concrete Frames:** Typical reinforced concrete frames are present in residential and commercial areas, consisting of reinforced concrete beams and columns to resist both lateral and vertical loads. The floor systems are usually solid or ribbed concrete slabs and the infill walls are made of CMU or brick or both.

Figure 16. Concrete frames with masonry infill, Manta

- **Timber frames with masonry infill walls:** A common type of housing structure is timber frame with infill masonry walls (Fig. 17 and 18). In some case, the end bays also included timber diagonal bracing within the infill. Timber frame elements were typically approximately 4"x4" or 6"x6". Brick turned on its short side was the most common infill material. As observed in damaged houses where collapsed infill left these portions exposed, the connections between masonry infill walls and timber members were commonly composed of nails partially driven into the columns so that the undriven length would embed into the mortar joints of the infill. Wood joist framing was used as the floor system and this type of housing typically had lightweight roofs (CGI).

Figure 17. Timber frame with masonry infill walls and lightweight roofs.

Figure 18. Timber frame with masonry infill walls and lightweight roofs, outside Guayaquil

- **Timber frames with lightweight panels**: These houses were typically one or two stories tall and constructed with heavy timber frames. The timber framing was a mix of dressed or sawn lumber and natural uncut timber sections. The walls were built from light materials, including wood planks or bamboo lath (Fig. 19). In some cases, the bamboo lath was covered on the exterior with a cementitious plaster for finishing. Lightweight floor and roof systems were constructed with wood framing.

Figure 19. Timber frame and lath walls (Exterior and Interior walls), Canoa

- **Light metal framing with infill**: This building type was less common. It consisted of welded steel framing (square or rectangular tube shapes) with infill concrete block walls (Fig. 20).

Figure 20. Metal framing with infill concrete blocks wall, outside Guayaquil

- **Combined Systems**: Combined systems were very common and included several different combinations of materials, such as steel framing or timber framing, masonry walls and concrete columns (Fig. 21). In some cases, houses had a masonry and concrete ground floor, with a timber upper level (Fig. 22). In other cases, timber framed houses had a mix of timber wall panels and masonry infill panels. There were also some houses that had appeared to originally be timber

frame with masonry infill which were then modified later to include reinforced concrete columns and beams. It was most common to see this addition of concrete frame elements to the front exterior columns of timber and masonry houses – supporting a balcony or overhang above.

Figure 21. Reinforced Concrete Frames, concrete block masonry and steel elements in the gable wall, outside Guayaquil

Figure 22. 1st floor: Concrete frames with infill walls. 2nd Floor: Timber frames with lateral bracing (In some cases) and infill brick walls. Manta

4.2 Schools

For schools, there were two main types of construction systems observed:

- Concrete frames, typically with masonry infill
- One-story light frame with infill

- **Reinforced Concrete frame with infill walls**: The most common type of structure for schools observed was reinforced concrete frame with infill walls. These were typically two or three story buildings. Usually cantilevers at the front support the upper level corridors and the guardrails are constructed of masonry (Fig. 23). Masonry parapets were present at the roof level. The floor system was usually a ribbed slab.

Figure 23. Reinforced concrete frames with infill walls for schools, Jama

- **One-story light frame with infill**: Some school buildings were one-story structures built out of light frames, such as steel tubes, with masonry infill walls (Fig. 24). These typically had lightweight roofs with metal or timber framing and CGI cover.

Figure 24. One-story light frame with infill classroom, Canoa

4.3 Construction Materials

There are several material types identified for common construction of housing and schools.

Masonry units for walls

- Brick: Solid craft brick with different dimensions: 6.5x17x24 cm, 8x12x25 cm, 6x12x25 cm, 6x10x18 cm.

- Hollow Clay block: There are several types of clay blocks with horizontal perforations, with different dimensions, such as: 8cmx20cmx40cm, 7cmx20cmx40cm, 10cmx20cmx40cm.

- Concrete Masonry Units, CMU: Typically 14cmx19cmx39cm in dimension. According to the websites of several block manufacturers, the average compressive strength of the block is 8.0MPa, although sampling and testing was not performed during the visits.

Mortar and concrete

For concrete and mortar mixtures, homeowners of informal housing reported using either sand from nearby quarries or river sand, and Portland cement. A builder in Crucita thought that the relatively better performance of buildings in that area was in part attributed to the use of sand from a nearby quarry, versus beach sand, which they alleged was used in concrete construction in other surrounding areas. Aggregates observed in stockpiles at construction sites appeared to be angular and good quality (Fig. 25).

Figure 25. Sand and coarse aggregate stockpiles for informal construction, Crucita

Steel reinforcing

In reinforced concrete frame elements, the rebar for longitudinal reinforcing was usually ½" diameter ribbed bars, and for transverse reinforcing, like stirrups and ties, it was usually ¼" diameter smooth bars. (Fig. 26)

Figure 26. Typical reinforcing exposed in damaged column, Tarqui, Manta

Timber / Timber Lath

Timber frames are very common along west coast of Ecuador, with brick or clay block infill. For columns and beams the most common sizes were 4"x4" and 6"x6". For floor joists and roof rafters the common sizes were 2"x4" and 2"x6".

For perimeter and interior walls, wood panels, planks or bamboo lath are used. Such walls in some cases are covered with non-structural plaster. The bamboo is sliced longitudinally and rolled flat to create the lath (Fig. 27).

Figure 27. Preparing bamboo lath, Canoa

5. DAMAGES OBSERVED AND DISCUSSION

5.1 Concrete Frames with Masonry Infill – Houses

Overall, reinforced concrete frame buildings with infill that were 1 and 2 stories tall fared better and had less severe damages than buildings 3 stories and taller. In these taller buildings, which also had increased mass and flexibility, the deficiencies were more likely to cause damages and in some cases, collapse of the structures, although the deficiencies were not unique to the taller buildings.

Failure of frame columns

- Description:
  Concrete frame column damage was observed in many concrete frame buildings – from single story to multi-story (Fig. 28). Typically, damage was localized at the base or top of the column due to flexural demands. Column damage was much more common than frame beam damage. The damage observed varied from cracking and spalling of the concrete cover in the column, to complete deterioration of the concrete core or fracture of the reinforcing bars.

Figure 28. Non-ductile column failure in multi-story building, Manta (left) and single-story building, Jama (right).

- Possible Causes:
  The failure of the columns can be attributed to several possible causes that may apply to different cases observed. These frames may be undersized for the demands required. Additionally, it was common for the slab to be integrated with the beam, increasing the capacity of the beam significantly and thus forcing failure in the column, which would be relatively weaker than the beam (Fig. 29). It was also observed that many of the failed columns did not have what would be considered "ductile" reinforcing details – such as closely spaced stirrups (many were spaced 12" more apart instead) or 135° hooks at the stirrups (many had short 90° hooks instead) – so that the resulting non-ductile frame columns were more susceptible to failure in the earthquake and unable to resist the drift demands. Additionally, in some cases, the reinforcing at damaged columns was severely corroded, which may have caused microcracking or weakening of the surrounding concrete, early fracture of the longitudinal bars, or breakage of the stirrups leaving the concrete core unconfined and weaker. The placement of infill walls may have unintentionally

increased the demand on same elements by creating torsional irregularities in the buildings in some cases.

- Possible measures for improvement:
  For new construction, concrete frame buildings with infill should use reinforced concrete frame elements that are adequately proportioned for the required demands and use ductile detailing as required by current building codes. The use of corroded reinforcing steel or aggregates and water that may increase corrosion of the steel should be avoided. Proper concrete cover should be provided, and increased in corrosive environments, such as locations close to the ocean. Additionally, the placement and use of masonry infill panels should be considered to contribute to the rigidity of the structure as appropriate and be accounted for accordingly.

Figure 29. Failure of vertical elements near the column-beam joints, Jama.

Soft-/weak-story failure

- Description:
  Soft-/weak-story failures were observed as collapse of a story, typically the ground floor, of multi-story concrete frame with infill buildings (Fig. 30). In most cases, the collapsed level was where the damages were concentrated in the structure and the remaining levels were left with relatively less damage (Fig. 31).

Figure 30. Before (left, source: Google Street View) and after (right) earthquake, Tarqui, Manta.

Figure 31. Ground floor collapsed due to soft/weak story failure, Jama.

- Possible Causes:
  Soft-story and weak-story failures can occur when one level is significantly less stiff or less strong, respectively, than the adjacent levels. Because of the degree of damage, it is difficult to discern from a collapsed building whether the stiffness or strength or a combination of both contributed to the failure. Soft-stories can be caused by taller ground floor heights, more open ground floors such as in mixed use buildings, set-back ground floors such as in overhang conditions, and other configurations causing the lower level to be more flexible and less stiff than upper levels. Weak stories can be caused by similar configurations or also by construction detailing, such as inadequate splicing at a level, or a change in construction materials quality at a certain level.

- Possible measures for improvement:
  Avoid or correct soft and weak story problems in new and existing construction. This can be done by addressing the cause of the soft or weak story, such as
  - Providing rigid elements at the soft story to reduce its flexibility to be more compatible with the other levels. This could include infilling openings, stiffening frames, adding infill panels in strategic locations, or removing overhanging portions of the building.
  - Strengthening the lateral force resisting system in locations where inadequate detailing or where a change in material quality may have created a weak point. This strengthening would need to be performed without creating new large discontinuities in stiffness.

In-plane cracking of infills

- Description:
  Diagonal cracking of masonry infill due to in-plane shear loading was observed (Fig. 32 and Fig. 33). Cracks also tended to initiate at the corners of windows and doors in panels with openings. This types of damage rarely appeared to lead to the collapse of the wall, unless combined with out-of-plane failure or failure of the adjacent frame elements.

Figure 32. In-plane masonry infill wall cracking, Tarqui, Manta

Figure 33. In-plane cracking at end wall, Canoa

- Possible Causes:
  Diagonal cracking in masonry wall due to in-plane shear loading is not unusual and can be a more ductile damage condition than others. In-plane failure can be attributed to the low resistance of masonry units and joint mortar, large openings in the wall without confinement, or a lack of sufficient shear walls for lateral resistance.

- Possible measures for improvement:
  To reduce the incidence of in-plane shear cracking and failure, several approaches can be taken, such as: reduce the size of openings to increase the length of shear walls, provide confinement (reinforcing) around openings in each wall panel, increase the amount of shear walls in each direction by infilling more bays, ensure good quality mortar and bricks are used. When increasing

the length of infills is not possible, strengthening the shear wall panels can be an options, such as by integrating reinforced concrete overlays or walls to the structure.

Out-of-plane failure of masonry infill walls

- Description:
  In concrete frame buildings, out-of-plane failure was less common than in other building types described below. Typically, out-of-plane masonry failure in concrete frame buildings was accompanied by one of the following conditions: severe in-plane cracking of the panel, poor connection to the frame, particularly at the top (Fig. 34), or lack of a frame beam such as at the top of a building where a temporary light-weight roof had been built prior to completion of the concrete frame (Fig. 35).

Figure 34. Out-of-plane failure of masonry walls. Perforated block/vent at wall panel connection to slab. Manta

Figure 35. Out-of-plane failure of masonry walls. Lack of ring beam or diaphragm at roof. Manta

- Possible Causes:
  Out-of-plane wall failure seemed to be caused by the following conditions: slender walls (thin side-stacked brick spanning full wall height) infilling large panels, large spans between perpendicular bracing elements such as walls or frames, poor connection of the infill to the underside of slab due to window or perforated block at interface, lack of bracing at top of walls such as not diaphragm or ring beam, failure of rebar dowels between the wall and column (in some cases due to corrosion), and out-of-plane failure after excessive in-plane cracking.

- Possible measures for improvement:
  When light weight roofs are used, reinforced concrete ring beams should be placed at the top of the masonry panels to connect the top of the walls to perpendicular cross walls, which should have a close, regular spacing to provide out-of-plane bracing to the heavy wall panels. Infill should be installed tight to the underside of the beam or slab above and the connection can be improved by pouring the beam or slab directly on top of the wall after the wall has been built. Infill should be well connected to the adjacent columns, with good quality, sufficiently spaced dowels, or by toothing in the wall when the columns are poured after the wall is constructed. Thin, slender walls can be effectively thickened by using wider masonry units or with overlaying existing masonry with structural elements such as reinforced concrete overlay or structural plaster on one or both sides or intermediate beams can be used to help reduce the span of the infill.

Overhang walls failure

- Description:
  Wall failure at overhangs, diagonal cracking (Fig. 36).

Figure 36. Failure at the overhangs, Manta

- Possible Causes:
  Discontinuity of vertical load caused stress concentrations at the overhangs and increase of flexibility, leading on cracking of overhanging walls and slabs.

- Possible retrofitting measures:
  Make walls continuous from foundation to roof, design walls as non-structural elements by providing the proper confinement and tying directly to the slab.

Short column failure

- Description:
  Significant damage to the portion of a column left unrestrained by adjacent structural elements, such as masonry infill, which restrain other portions of the column during an earthquake. This was only observed in a limited number of locations (Fig. 37).

Figure 37. Short Column failure. Outside Canoa

- Possible Causes:
  It's caused by lateral restriction of partial height of the column or the construction of short columns within the building, leading to shear concentration along such short effective length. A short column has large stiffness and during earthquakes attracts higher value of earthquake induced forces.

- Possible measures for improvement:
  This can be fixed by avoiding columns which are restrained over part of the height, either by providing lateral restriction of the column's free length (over the full height), or eliminating the lateral restriction of the partial height of the column.

Punching shear

- Description:
  This type of damage was mainly observed in locations where partial or full story collapse occurred and was observed in very limited numbers. The column "punched" through the slab above where the reinforced concrete slabs was subjected to high localized forces, such as at column support points.

Figure 38. Punching Shear failure / Red Tagged house, Manta.

- Possible Causes:
  Collapsing slabs due to column failure would impact the lower portion of the column and produce localized concentration of forces that may have caused this type of damage. Additionally, the concrete frames were often built with shallow beams that fit within the depth of the slab (flat slab) which reduces the shear capacity of the beam in comparison to deeper beams.

- Possible measures for improvement:
  Column failures and thus story collapses should be prevented as noted in the section above on concrete frame failures. Providing deeper frame beams with adequate shear reinforcing along column lies could also help reduce this damage condition.

5.2 Concrete Frames with Masonry Infill – Schools

Overall, the performance of the concrete frame schools observed was fair. None of those observed had collapsed and only one appeared to have permanent residual drift. However, some of the damage to non-structural elements observed may pose a risk to the safety of students due to falling hazards and obstruction to egress. The following are the primary damages observed during the site visits.

Concrete frame column damage

- Description:
  The damage to the frames observed included cracking of the core concrete and spalling of the cover in the ground floor columns at the base and at the top below the floor beam. In one building, the frame displayed a permanent drift. Damage to the frame beams was not observed.
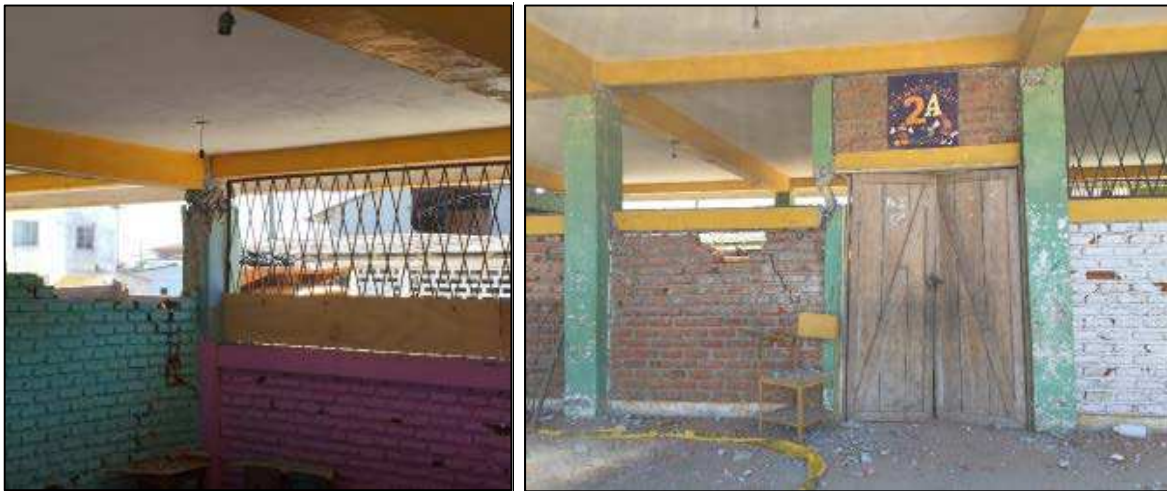

Figure 39. Column damage in classroom building, Canoa

- Possible Causes:
  It is hard to determine the cause of the cracking in the frames, potentially there was transverse reinforcing deficiencies, non-ductile joint detailing, or a consequence of construction sequencing (in one frame, cracking appeared to be concentrated at a horizontal cold joint at the top of all columns).

- Possible measures for improvement:
  Use of modern ductile frame detailing.

In-plane infill cracking

- Description:
  Diagonal cracking of infill walls (exterior walls and interior partitions) within the reinforced concrete frame.

Figure 40. Cracking and failure of non-structural elements at a school, Jama

- Possible Causes:
  Interaction between concrete elements and infill walls, such that the infill walls are active in resisting lateral forces although they may not have been designed to do so.

- Possible measures for improvement:
  Separate infill walls from lateral columns and frame beams to isolate the panel in-plane, or account for the infill in the design and construction, including providing adequate connections between the frame elements and infill.

Out-of-plane infill collapse

- Description: Out-of-plane failure of masonry infill was observed in a limited number of concrete frame schools. In one case, the reinforced bracing elements for the infill openings also failed, separating from the frame columns at the joint – leaving the masonry unbraced (Fig. 41, left).

Figure 41. Out-of-plane failure of a classroom walls, Canoa

- Possible Causes:

Out-of-plane damages in concrete frame schools appeared to be caused by two main reasons: slender infill walls in large panels that were cracked due to in-plane engagement, and failure of the intermediate confining/bracing elements, either at the connection to the structure (Fig. 42) or due to high deformation demands.

Figure 42. Connection failure of wall panel bracing element, Canoa

- Possible measures for improvement:
  Separate infill walls from lateral columns and frame beams to isolate the panel in-plane while maintaining sufficient out-of-plane support to the panel. Add intermediate beams or columns to improve bracing of the panels. Where bracing elements are used, such as in panel openings, ensure the elements are well connected to the structural frame and detailed for ductility.

Parapet collapse

- Description:
  Cracking and collapse of masonry roof parapet (Fig. 43). While this does not affect the structure of the building, it could be hazardous to students or others below.

Figure 43. Parapet collapse at a school building, Canoa

- Possible Causes:
  Lack of confining or reinforcing elements along the parapet.

- Possible measures for improvement:
  Provide confining elements or buttresses in order to reduce displacements and stiffen the parapet wall or avoid the use of unreinforced masonry parapets.

5.3 Light Frames with Masonry Infill – Houses

Out-of-plane failure of infill panels or walls

- Description:
  One of the most common damages observed for this type of building was the out-of-plane collapse of infill masonry – either individual panels separating form the framing, or full panels, including the framing (Fig. 44).

Figure 44. Failure of infill walls. (Manta, top and Jama, bottom)

- Possible Causes:
  Where panels separated or collapsed, this may be due to there being too large of a span for the infill (large spacing of timber elements), or where spans were limited, a lack of good connections between the infill and timber elements. Where panels, including framing collapsed, this may be due to insufficient or deteriorated connections to the cross walls, or insufficient quantity and spacing of cross walls.

- Possible measures for improvement:
  Ensure that timber elements are closely spaced to provide adequate bracing and the masonry infill is positively connected to the timber. Wall panels should be braced to perpendicular cross walls of the same construction type, with a positive connection between the panels designed to transfer the loads. The timber should be in good condition, with no signs of decay or infestation and treated to be protected from future deterioration.

Out-of-plane failure of gables

- Description:
  Out-of-plane failure of masonry gables at roof level occurred even when the remaining portions of the infill wall were not damages (Fig. 45).

Figure 45. Collapse of infill gable walls, Canoa, left and Manta, right

- Possible Causes:
  The causes are similar to those described above for wall panels. However, gable wall can be more difficult to adequately brace because they extend above the top of the walls.

- Possible measures for improvement:
  Use lightweight materials in the gable. Do not use masonry infill in the gable.

Failure of post connections, loss of vertical support

- Description:
  In several cases, following the collapse of infill, the posts became detached from the floor framing which led to the consequent loss of vertical support and collapse hazard.

Figure 46. Wood frame connections failure, loss of support, Canoa

- Possible Causes:
  This can occur because of the lack of good connections among timber elements and the large displacements of the timber frame due to large frame flexibility after loss of infill.

- Possible measures for improvement:
  Provide proper, positive connections between timber posts and floor framing elements. Also, ensure that the walls will maintain their lateral strength through avoiding loss of infill or providing an alternative bracing mechanism, such as wood panel shear walls or bracing.

Collapse of deteriorated/decayed elements

- Description:
  Partial or total collapse of the structure due to the presence of weak, deteriorated elements. Figure 47 shows a house with little earthquake damage, except for a wall that collapsed out-of-plane. It appears that the wall was sitting on a severely decayed floor beam, which may have weakened the support it could provide.

Figure 47. Collapse of wall above decayed floor beam, Los Esteros, Manta

- Possible Causes:
  Lack of maintenance of structural elements, poor constructions procedures, use of recycled and non-suitable materials, water intrusion, insect infestation.

- Possible retrofitting measures:
  Timber elements are susceptible to deterioration from water intrusion, dry rot and insect infestation. They should be treated to prevent these sources of deterioration and protected from the elements. Periodic inspection and maintenance should be provided and elements showing signs of deterioration should be replaced.

5.4 Light Frames with Masonry Infill – Schools

Out-of-plane failure

- Description:
  Many of the classrooms constructed out of lighter frames with infills had very thin, but heavy masonry walls, which were highly susceptible to out-of-plane failure (Fig. 48), particularly because classrooms tend to have large, open spans, with limited cross walls for bracing. The frames were usually not independently rigid, and were brought down with the collapse of the infill. This often led to the collapse of the roof structure above as well, which sat on the frames and walls for vertical support (Fig. 49).

Figure 48. Out-of-plane failure of masonry infill wall in light frame. Jama (left) and Canoa (right)

Figure 49. Out-of-plane failure of masonry wall and roof collapse, Jama

- Possible Causes:
  Inadequate out-of-plane capacity and lack of sufficient cross walls.

- Possible measures for improvement:
  Using less slender masonry walls that are adequately braced out-of-plane at the top by an adequately sized ring beam or diaphragm, which can deliver the loads to the perpendicular cross walls and reduce the out-of-plane deformations, can eliminate the likelihood of out-of-plane wall failure. This will maintain the vertical capacity of the wall to support the roof.

5.5 Timber Frames with Light-weight Panels – Houses

The damages in light, timber buildings were typically less severe than those in heavier buildings with masonry. Some collapses were observed as noted below, possibly due to the presence of deteriorated structural elements or lack of sufficient lateral system.

Story collapse

- Description:
  Collapse or partial collapse of timber buildings, typically where overhangs were present and the frames were covered with vertical siding (Fig. 50).

Figure 50. Partial story collapse in timber buildings, Jama

- Possible Causes:
  Several causes likely contributed to the failure of these buildings, including potential deterioration of the frame elements, poor connections between posts and framing elements, and a lack of sufficient lateral strength. Without infill, the frames require additional elements for rigidity and strength, and the vertical siding likely did not provide enough strength and stiffness to the frames to resist the earthquake demands, particularly at the lower level where forces are higher.

- Possible retrofitting measures:
  Refer to the section above regarding preventing deterioration of timber. Frames can be covered with horizontal planking, wood structural panels, or plaster on metal lath, or braced to increase the lateral strength and stiffness. Positive connections between the frame elements should be provided.

Failure at connections

- Description:
  Movement of the floor relative to the timber post was observed in several buildings (Fig. 51), including movement significant enough to allow the post to detach from the floor (Fig. 52). This can cause a loss of support and collapse hazard for the building.

Figure 51. Displacement of posts relative to floor framing, Jama

Figure 52. Collapse of timber post, shoring in place, Jama

- Possible Causes:
  This can occur because of the lack of good connections among timber elements and the large displacements of the timber frame in an earthquake due its flexibility.

- Possible measures for improvement:
  Provide proper, positive connections between timber posts and floor framing elements. Also, decrease the flexibility of the frame by adding elements, such as those noted above, to increase the strength and stiffness of the building.

5.6 Combined Systems – Houses

Separation between dissimilar materials

- Description:
  Separation between different parts of the building, made of dissimilar materials. Partial collapse of the buildings.

Figure 53. Separation of front façade and overturning, Manta

- Possible Causes:
  Lack of proper tie elements and connections among different parts of the building.

- Possible retrofitting measures:
  Tie properly all portions of the building, improve connections. Avoid using dissimilar materials or different systems within one structure.

Failure of light-weight walls to support heavier perpendicular walls

- Description:
  Failure of light-weight walls supporting heavier perpendicular walls, leading to partial building collapse.

Figure 54. Failure of light-weight walls due to heavier perpendicular walls (Manta, left and Jama, right)

- Possible Causes:
  Combination of wall systems in both directions. Different stiffness for both directions leading to large deflections. Lack of good connections between two different types of wall.

- Possible retrofitting measures:
  Make the bearing wall system as uniform as possible, tie light weight walls to the diaphragms and design them as non-structural elements. Provide masonry walls along weak direction.

5.7 General Damages

Adjacent buildings - impact

- Description:
  Damages in buildings due to the adjacent building impacting it either repeatedly (pounding) (Fig. 55) or during collapse (Fig. 56). The damages were both on the exterior (cracking of frames and infill) and on the interior (in-plane cracking of walls resisting impact loads).

- Possible Causes:
  Lack of seismic joint and diaphragms of adjacent buildings at different levels. Collapse of adjacent building.

- Possible measures for improvement:
  Provide a seismic joint between adjacent buildings according to the expected drifts. Match the diaphragm levels between adjacent buildings where possible.

Figure 55. Pounding in adjacent building, Portoviejo

Before

After
Figure 56. Adjacent building collapsed and affected this building in the mid-height of exterior walls, Canoa

Failure of site walls

- Description:
  Out-of-plane collapse of masonry site walls (Fig. 57). These were more common at schools versus houses.

Figure 57. Failure of site walls at schools (Canoa, left and Jama, right)

- Possible Causes:
  Lack of sufficient reinforced beams and columns to resist overturning moment due to out-of-plane loads and provide confinement to the unreinforced masonry panels.

- Possible measures for improvement:
  Bracing, such as reinforced concrete columns could be added at more frequent intervals and top of wall bracing, provided by a top beam could also improve the performance of these walls. The foundations must also be designed to resist the overturning loads form the bracing elements.

6. CONCLUSIONS

Ecuador is located in an area where seismic activity is high due to the adjacent subduction zone along the pacific coast. Today, aftershocks continue to affect the area and earthquakes will occur again in the future.

There were many buildings heavily damaged or collapsed, not only hotels and commercial buildings, but also housing and schools. The main building types and damages observed for houses and schools were:

Concrete Frames with Masonry Infill – Houses
- Failure of frame columns
- Soft-/weak-story failure
- In-plane cracking of infills
- Out-of-plane failure of masonry infill walls
- Overhang walls failure
- Short column failure
- Punching shear

Concrete Frames with Masonry Infill – Schools
- Concrete frame column damage
- In-plane infill cracking
- Out-of-plane infill collapse
- Parapet collapse

Light Frames with Masonry Infill – Houses
- Out-of-plane failure of infill panels or walls
- Out-of-plane failure of gables
- Failure of framing connections, loss of vertical support
- Collapse of deteriorated/decayed elements

Light Frames with Masonry Infill – Schools
- Out-of-plane failure

Timber Frames with Light-weight Panels – Houses
- Story collapse
- Failure at connections

Combined Systems – Houses
- Separation between dissimilar materials
- Failure of light-weight walls to support heavier perpendicular walls

General Damages
- Adjacent building impacts
- Failure of site walls

There is a need to improve the common construction practices used for housing in the Manabi Province, which resulted in many collapsed or heavily damaged buildings. There is also the need to improve non-structural components of school buildings and to eliminate the use of light frame with masonry infill systems for schools.

Timber structures generally fared better than masonry and concrete structures. Many homeowners were interested to rebuild in timber, particularly outside of major cities, and small changes to timber building practices can make them more earthquake resistant. Although many homeowners the team interviewed were hesitant to rebuild with concrete systems due to the deadly impact failures of these building types had, in the urban areas it will eventually be necessary to build in concrete due to the density. Timber buildings may not be able to reach the heights or have the fire resistance necessary for a dense urban setting. This means that it will also be important to focus on education and training in the reasons why concrete and masonry buildings failed, and the important aspects for making them safe and earthquake resistant in the future.

Changing the construction practice can be achieved by providing clear, simple and accessible technical resources, using local materials, and training to homeowners, builders and local authorities, in the construction of seismic-resistant structures. Reconstruction efforts can take advantage of the recently published NEC15, construction code, which provides guidance on the earthquake hazard and seismic design of structures in Ecuador.

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