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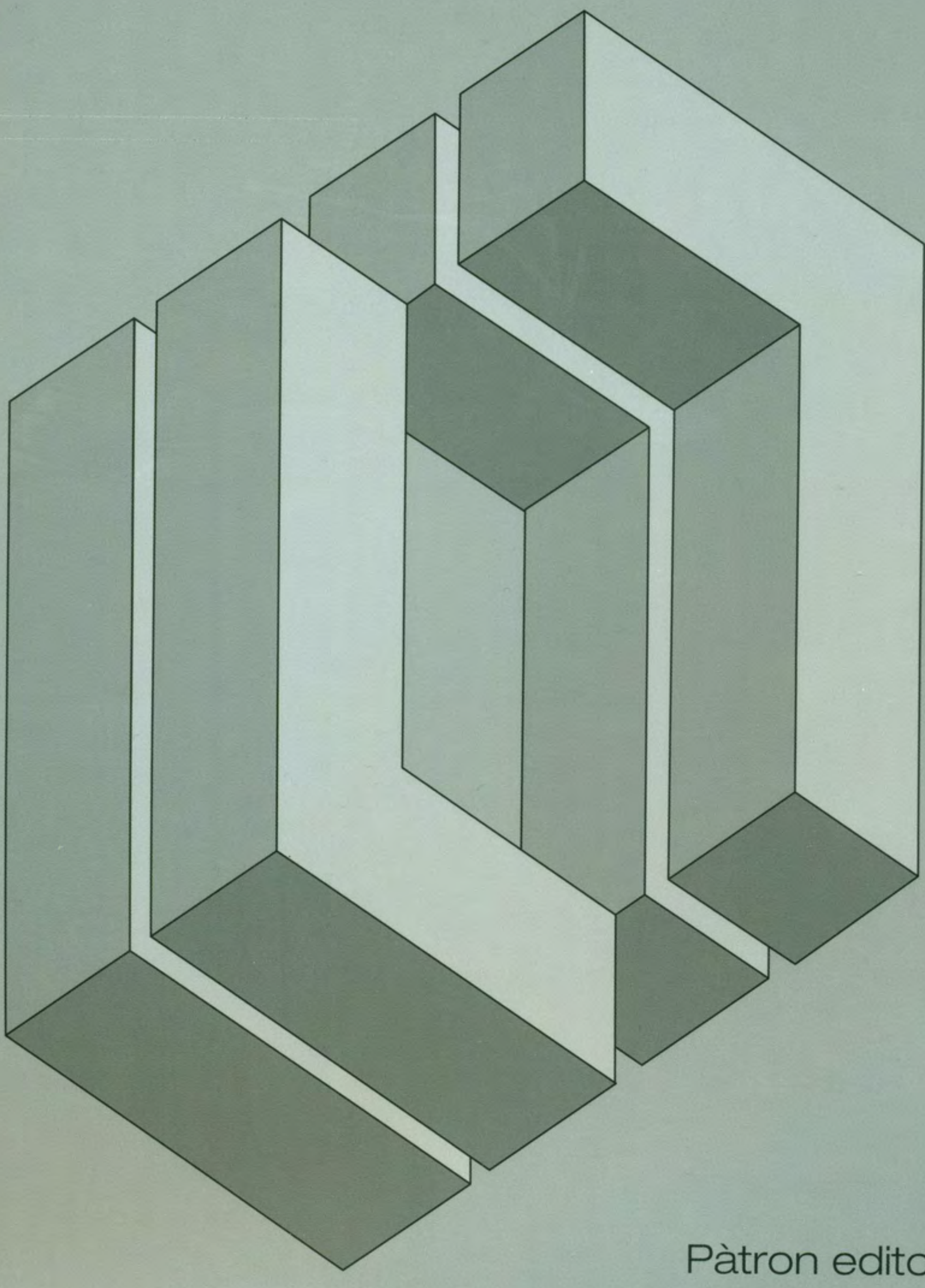

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The Haiti Earthquake Mw = 7.0 of January 12th 2010: structural and geotechnical engineering field observations, near-field ground motion estimation and interpretation of the damage to buildings and infrastructure in the Port-au-Prince area

Panayotis Carydis*, Efthymis Lekkas**

SUMMARY – *The Mw 7.0 destructive earthquake that struck the Republic of Haiti on January 12, 2010 was one of the deadliest of the last century in the region. What has been concluded after a field study during the period from 16th to 22nd of January 2010 is that this event is mostly a man-made natural disaster. This is due to the very adverse social, political and economic conditions that existed before the earthquake that greatly contributed to the lack of any building – construction code and control of land use and the lack of any governmental – public services during the post-earthquake emergency. The primary effects of the earthquake were worsened due to the inability or absence of any preparedness for search and rescue and help the victims. The frozen response of the structures to the seismic event, observed during the field trip, gave valuable information about the ground motion, since instrumental records were unavailable. The response of a fence functioning as a primitive seismoscope was analyzed, resulting to an effective ground horizontal acceleration of about 0.6 to 0.7 g. Similarities between Haitian structures and their response with instrumented cases in other regions, resulted in the acceptance of a 1.0 g vertical acceleration for the site. Finally, the detailed investigation of the whole liquefaction phenomenon at the site, documented that this is starting with ejection of sand and water, before the large ground displacements take place and continues after the strong phase of the ground motion.*

Keywords: Earthquake reconnaissance; historical and traditional buildings; liquefaction and phases of liquefaction; vertical seismic component; disaster management.

1. Introduction

The Mw 7.0 destructive earthquake struck the Republic of Haiti on January 12, 2010 (4.53 p.m. local time), with an epicentre approximately 20 km W-SW of the city centre of Port-au-Prince. The earthquake ruptured initially at a shallow focal depth (12 km) and produced extensive landslides, liquefaction and lateral spreading, Figs. 2 and 3. According to the analysis of the response of a fence, as this is described in the following section, the main event lasted over 20 seconds approximately. It was followed within 20 minutes by two large aftershocks along its western end, an Mw 5.9 at 5.00 p.m. local time and an Mw 5.5 at 5.12 p.m. local time. An aftershock sequence developed during the next few days, including an Mw 6.1 earthquake on January 20, 2010. The authors visited the earthquake stricken area from 16 to 22 January 2010 and studied it from geological, engineering and disaster management points of view. The initial purpose of the visit had a dual objective: first, to check the effects of a strong shallow earthquake to a populated area at a rather small epicentral distance and compare the findings to other similar cases, /5/ and /6/; second, to try to explain the reason why so many victims, structural damages and losses were caused by this earthquake. The authors believe that these two goals have been fulfilled. Nevertheless, during their visit they had a unique opportunity to observe the undisturbed and "fresh" earthquake effects and especially: a) the valuable evidence of the earthquake response that was recorded by a rather free standing fence which was functioning as a simple seismoscope; b) the in time documentation of the evolution of the liquefaction phenomenon; and c) the response of the responsible authorities and the affected population.

The Enriquillo-Plantain Garden Fault Zone (EPGFZ), /14/ and /15/, has been seismically quiescent during the 20th century, although historical seismicity indicates that movement on this fault zone caused the largest events of the last centuries, such as the ones of 1615, 1673, 1684, 1691, 1751, 1761, 1770, 1860 and 1887, /13/. The event in 1770 is referred to as the «Port-au-Prince Earthquake» and is assumed to have occurred on the same fault segment as the event of January 12, 2010.

The EPGFZ is located on the southern part of Hispaniola Island. It can be traced along the southern part of the Leogane city and towards the east crosses the Momance and Froide rivers, runs through the northern flanks of Massif de la Selle, enters the Cul-de-Sac plain and Azuel lake, and terminates in Enriquillo lake. The EPGFZ is related with a series of key changes in the drainage network, such as: a) branches piracy; b) systematic changes in streams direction; c) asymmetric

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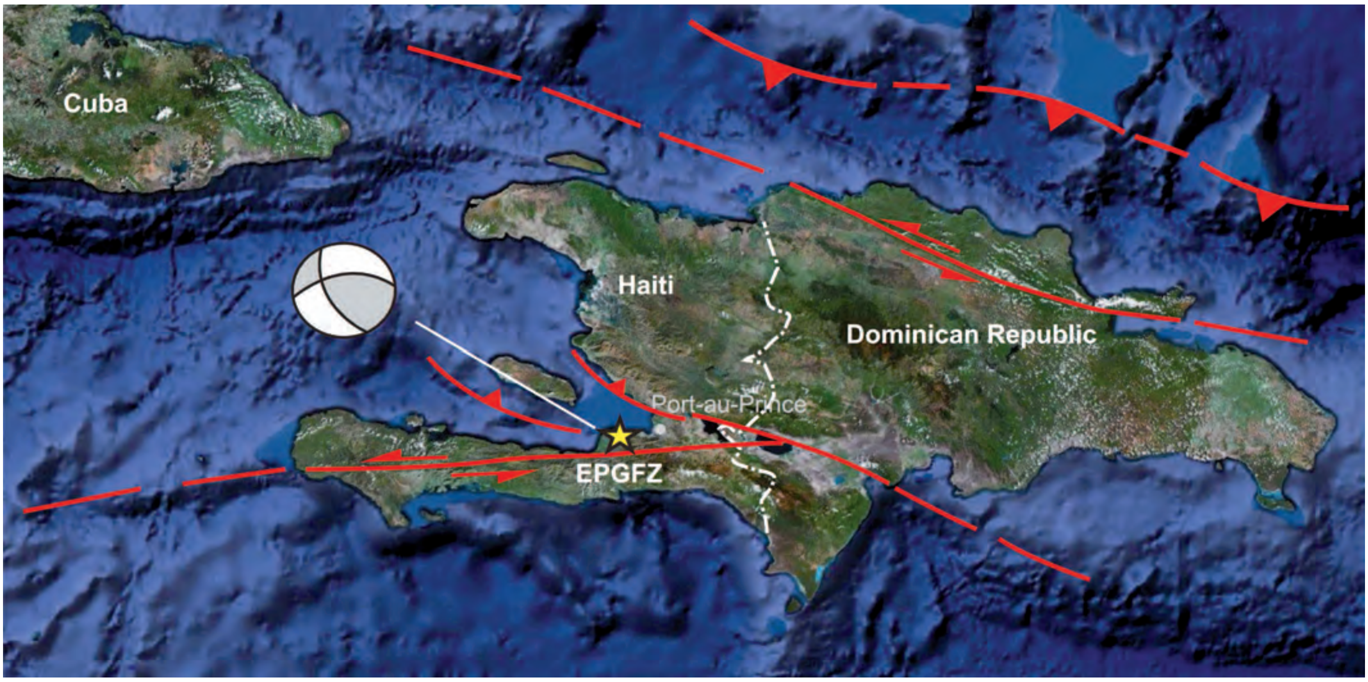


Fig. 1. *Simplified tectonic map of Hispaniola Island. The epicentre of the January 12, 2010 earthquake is indicated by a star. The fault plane solution is also shown.*

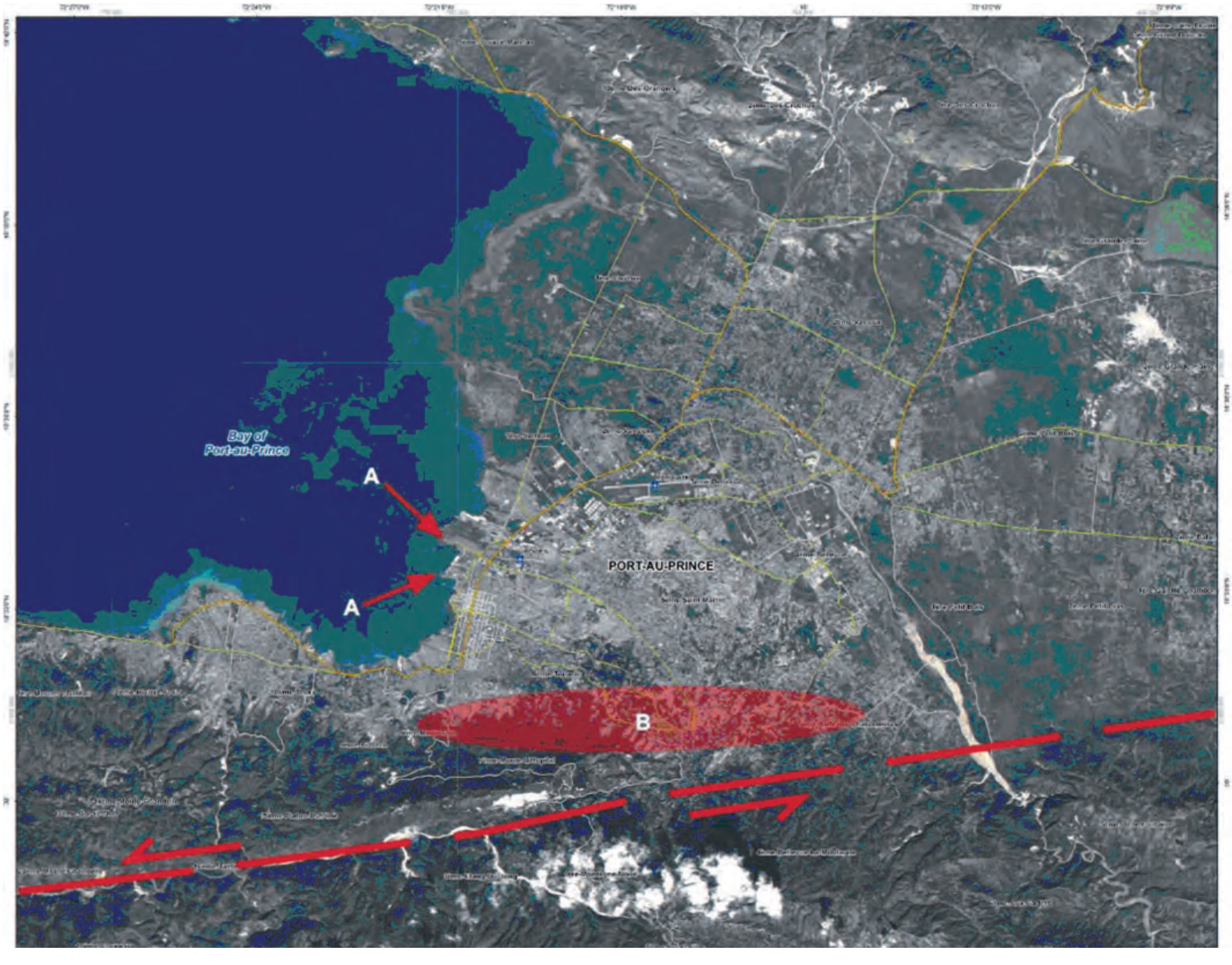


Fig. 2. *Red line indicates part of EPGFZ at the southern part of Port-au-Prince. Arrows show numerous surface manifestations of liquefactions and lateral spreading fissures in the port (A) and hatched ellipse areas of extensive landslides in the southern districts of the city (B).*

Fig. 3a-h. *Extensive liquefactions and lateral spreading phenomena caused significant damages in the port facilities of Port-au-Prince. Marine transportations and shipping were interrupted during the initial period after the main shock.*

distribution of the drainage network in each fault block; and d) major topographic variations.

Recently, /4/, using Global Positioning System and Radar Interferometry measurements of ground motion showed that the earthquake ruptured an unmapped north-dipping thrust fault (Leogane fault) which terminates on the big EPGFZ, and is the southernmost front of the Haitian fold-and-thrust belt.

According to the general structure and focal mechanisms, ENE-WSW compressional force is dominant in the region of the Caribbean tectonic plate, which moves eastwards at around 20 mm/yr approximately, with respect to the much larger North and South American plates, /2/, /3/ and /9/. On its eastern edge, the boundary runs perpendicular to the direction of relative plate motion, so there is concurrent compression and subduction. However, as the boundary curves around to form the northern boundary, it starts to parallel the direction of relative plate motion, making strike-slip faulting along E-W trending faults, like EPGFZ and transpression along the Leogane thrust which dips to the north at an angle of approximately 60^o, /4/.

Taking into account the historical seismicity, the wider tectonic regime and the high population density of Port-au-Prince, the enormous and immediate impacts of the 2010 Haiti Earthquake, from a humanitarian and an economical point of view, can be easily inferred. The initial estimate of affected was approximately 3.0×10^6 people out of 4.0×10^6, while the area with the most severe damages was about 40×15 km^2. As a rough estimate, 3.5×10^6 people that were left homeless were living on the pavements of arterial roads waiting for the few supplies of food and water. In the locations where these necessities were provided into, many injuries and casualties were recorded. There is a considerable uncertainty in economic estimates of the 2010 Haiti Earthquake due to the lack of recent building inventory, personal property and real estate data. Nevertheless, it has been estimated that the total financial loss comes up to about $8 billion, which is about 100% of Haiti's GDP.

2. Estimation of Seismic Motion in Relation to Ground and Foundation Conditions

The ground motion is estimated: (a) from the observed effects on structures as this is explained in the respective section of the present communication; (b) from the relevant calculations based on recorded motion of a simple seismoscope; (c) based on recorded motions of similar or comparable events, as for example, the January 17th, 1995 Kobe, the April 6th, 2009 L'Aquila, the September 3rd, 2010 Darfield and the February 22nd, 2011 Christchurch earthquakes; and (d) based on comparative studies with other earthquakes with similar damages in the near field that the authors have visited and studied, /5/, /6/ and /17/. It is worth mentioning here that careful field observations of the response of structures are very important to get valuable information about the characteristics of the ground motion. This can be achieved, considering each building structure as a kind of a 3-D seismoscope, by a comparative evaluation of their responses. What it is observed is the frozen final response of each seismoscope, id est of each structure. It is worth mentioning here, based on hundreds of shaking table tests, that this frozen response does not always coincide with the maximum response. From the earthquake engineering point of view, the key point is to focus on subjects of major importance that are more frequently encountered, regardless of their earthquake capacity, which in every case proved to be generally poor. This process in the case of the January 12nd, 2010 Haiti Earthquake is of special importance, since there is a lack of any instrumental records in the region.

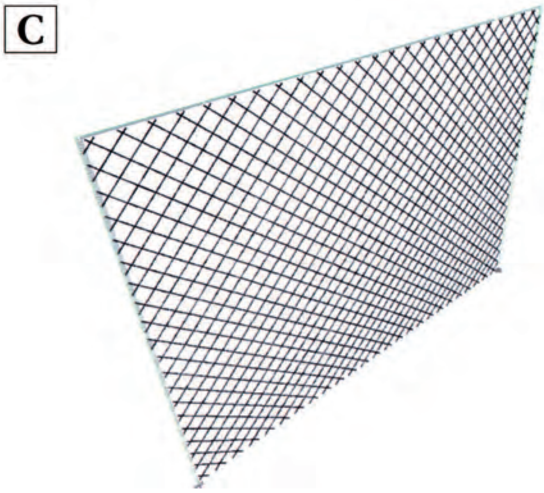
Another fact that has to be mentioned here, which also agrees with the engineering findings, is the existence of rather reliable witnesses. These people have been in different places around the city, they were not influenced by the media that were not functioning and – logically – their evidences are probably unbiased. They claimed that the collapse of the built environment took place within the first few seconds of the ground motion which definitely was along the vertical direction. A woman who was driving her car during the earthquake felt as if the car sunk into a large cavity of the road and, at that very moment, she looked around and saw that the buildings around her were already collapsing.

The evaluation of the response of a simple seismoscope was based on field observations near the Hotel Montana, about 14.0 km to the N-E of the epicentre. A fence made of steel wire net and mounted on steel posts which were fixed on a masonry base-wall, Fig. 4a, underwent some oscillations due to the earthquake. An horizontal concrete slab was constructed on the top of the base-wall. The dimensions of the wall were: 60 cm in thickness, 0.4-1.0 m in height and 30-40 m in length in en echelon geometry of about 5 m per step. The plane of the fence was along the NW-SE direction and the uphill direction was towards the S-E. It was evidenced that the foundation material of the base-wall was rocky and stable. Not a single damage or any sign of relative motion to the ground below the wall was observed. Yet, the top concrete surface of the base-wall had been lightly scratched by the lower ends of the wires of the fence during the earthquake, Fig. 4b. The displacements were ±13 cm along the NE-SW direction, perpendicular to the plane of the fence and quite symmetrical to its central position. Almost equal findings where documented along the other panels of the fence as well. The natural periods along NW-SE and NE-SW directions of the fence were measured, as described below, and the results were almost identical, at 0.6-0.7 sec.

Pulling by hand and suddenly releasing the centre of mass of one of the fence's panels it underwent free damped oscillations. The overall measured, damping was a combination of the internal damping and of the friction of the lower ends of the wires of the fence on the top concrete surface of the wall. The initial displacement that was applied was about 8.5 cm relative to the central axis on the concrete base and the ampli-

(a) General view. The plane of the fence is along NW-SE direction.

(b) Details A of the observed imprints – light scrtches, by the lower wires on the steel net on the concrete base. The direction of the motion is NE-SW.



(c) A print out of the analyzed model usign ETABS computer code, in which a symmetry of the boundary conditions is maintained along the two vertical posts of the panel.

Fig. 4. *A fence was functioning during the earthquake as a primitive seismoscope.*

tude after the lapse of the second period of oscillation at the same point was 3.0 cm. The response of the wire net was perpendicular to its vertical plane and all in phase with the first modal shape. The wires of the net were functioning as springs, just sliding and lightly scratching the underlying concrete surface.

The equivalent overall viscous damping ratio ζ may be defined according to the relation:

$$\zeta = \frac{-1}{2 \cdot \pi \cdot n} \ln \frac{Dn}{Do} = 8.3\% \quad (1)$$

Where:

Dn = 3.0 cm, Dn being the amplitude after the nth period

D_0 *= 8.5 cm,* D_0 being the amplitude at the beginning of the free oscillation and

n = 2

Taking into account all the geometrical parameters of the fence, the respective analytical dynamic model was composed using ETABS computer code as it is shown in Fig. 4c. In the composition of the model, special effort had been paid, in order to simulate all mechanical members that were measured and drawn at the spot with the highest possible accuracy. Symmetry of the boundary conditions along the height of the posts has been maintained. The accuracy of the model was checked by comparing its natural periods, along each direction, with the measured ones. A linear time history analysis was conducted by generating various artificial time histories as excitations compatible to EC-8 1998 elastic response spectrum Type 1 (strong earthquakes) and for ground type B (S = 1.2, T_B = 0.15 sec, T_C = 0.5 sec, T_D = 2.0 sec). By using a damping ratio ζ = 0.083, the maximum relative displacement to the moving ground D = 0.13 m was set as a target of the said analytical approach. Once the time history was set, its amplitudes were linearly normalized in order to achieve the target. Since several excitation time histories were used, a scattering was found with a rather small variation of the maximum ground acceleration. Its mean value was fluctuating between 6.0 and 7.0 msec^{-2}. Similar results were found by using some recorded accelerograms during the earthquakes of January 17th, 1995 Kobe and the April 6th, 2009 L'Aquila. The strongest records along the two horizontal directions in these two events were used one at a time (four in total). These earthquakes were selected since their seismotectonic characteristics are closer to those of Haiti.

By using the methods described at the beginning of the present section and the above mentioned analysis of the record of the simple seismoscope, the following characteristics are proposed, /7/, as best estimates:

The characteristics of the vertical ground motion are:

– the *acceleration* in hard soil conditions is of the order of $|a_v| \approx 1.0$ g;

– the *duration* of the strong phase is from about 4 to 9 sec, with an abrupt amplitude in the beginning and gradual decrease. (Zero (0) time is assumed to be the instant moment of the generation of the strong phase motion at the hypocentre);

– the *frequency* content due to the impact type of the motion is quite broad, $f_v \approx 5\text{-}25$ Hz in hard soil conditions.

– the *displacements* are rather small, of the order of no more than $|d_v| \approx 4$ cm in hard soil conditions;

– *amplitude versus distance*. A basic characteristic of the vertical seismic component is that it is damped out quickly as the hypocentral distance $R = \sqrt{H^2 + L^2}$ is increasing, were H is the focal depth, and L is the horizontal distance from the epicentre. For quite shallow depths ($H \leq 15$ km) and medium to small size earthquakes, the effects of the vertical component become insignificant at distances $L > H$. As the focal depth is increasing, the affected area by the vertical component is decreasing. On the other hand, with greater sizes of earthquakes ($M \geq 6.5$), the affected area is increasing.

The characteristics of the horizontal ground motion are:

– the *acceleration* in hard soil conditions is $|a_h| \approx 0.6$ to 0.7 g. In softer soil conditions this may be reduced;

– the *duration* of the strong phase is from about the 7th to the 25th seconds. It's amplitude gradually increases from about the 7th to the 9th, then it holds up to the 15th second and then further decreases;

– the *frequency* content is $f_h \approx 1.5\text{-}5$ Hz in hard soil conditions;

– the *displacements* are $|d_h| \approx 5\text{-}20$ cm in hard soil conditions. As the surface soil becomes softer, the displacements respectively enlarge;

– *direction of the motion*. The NE-SW direction, at least at this particular site, is dominant. On the contrary, no sign of significant motion on the NW-SE direction was found, although the respective natural periods of the fence were almost identical as already mentioned.

Due to the concurrence of the two main ground motions, vertical and NE-SW horizontal, although they function incoherently, they may have produced a combined stronger motion within the coinciding time window from 7 to 9 sec. During this time window though, the vertical acceleration vanishes while the horizontal one increases. Therefore, any arithmetic values in the expression $a_g \approx \sqrt{a_v^2 + a_h^2}$ would be quite disputable to set, and the effects quite dubious.

Furthermore, it has been concluded that the bedrock under the city must have a bowl shape with the open side towards the west. As a logical consequence, reflections of the seismic waves magnifying the horizontal seismic motions could have taken place only along the N-S direction. Still, the N-S magnification was not verified. This could be attributed to the fact that: a) the source of the main seismic energy release may be located outside the aforementioned bowl shaped bedrock formation; and b) the focal mechanism is a left lateral strike – slip along the E-W direction. On the other hand, convolution of the P waves may have taken place between the ones that came direct up to the surface and the reflected ones on the massive bedrock formation of the city, towards the east, south and north. For these reasons the Port–au–Prince region could be called as a macroseismic epicentral region possessing all those characteristics of a real microseismic epicentral region.

Buildings founded on, or close to the retaining walls in the hilly part of the city were subjected to amplified seismic motions, especially when the plane of the retaining wall had an N-S direction. What is more, the retaining walls had a height from one up to six or eight meters and they were rather slender, constructed out of rubble stone masonry, as it was proved after their collapse. The seismic hazard (or intensity) of the area was also increased by the fact that the ground (over the bed rock) along the slopes was instable and the flat part of the city was covered with fluvial deposits. It is worth mentioning here that near the upland region of EPGFZ, on the southern part of the city, many deforested areas and steep north trending slopes contributed to extensive landslides, Fig. 2. Many of these occurred in densely populated areas causing an additional loss of life and material, limiting the possibility of providing help where it was needed.

The water table in the alluvial plain is quite high throughout the whole flat part of the city. This produced an additional movability prone to the shaking of the ground particles. Another effect of the observed high ground water table level related to the transmission of the vertically emerging P waves, is the reduction of the absorption of the transmitted seismic energy due to the high incompressibility of the water compared to that of the dry ground. This fact plays a catalytic role in the present case to the transmission of high frequency and impact type P waves.

It has been observed that there is a large variation on the seismic intensity from one part of the city to the other, even within small horizontal distances where the soil conditions are nearly identical. These observations coincide with many other similar observations in epicentral regions of shallow earthquakes, where the vertical component of the seismic ground motion is dominant. It is interesting to mention here that the vertical seismic motion by itself is a function with effects on structures of an abrupt discontinuity: for $|\alpha_v| < 1.0$ g the results on structures are almost trivial, while for $|\alpha_v| \geq 1.0$ g the results can be extremely detrimental. In epicentral regions the $|\alpha_v|$ is about 1.0 g, but due to the various wave convolutions this quantity fluctuates above or below the critical limit of $|1.0\ g|$.

This fluctuation is quite sensitive to various parameters, as for example: a) the non – linearities that are developed along the transmission path due to rather high values of the accelerations; b) the quick reduction of the high frequency amplitudes according to the distance; c) the geological underground relief and the associated convolution of the emerging P waves due to various reflections and refractions; and d) the quality of the transmission path materials, combined with the ground water table. On the contrary, when the horizontal seismic component dominates the motion, the response at the ground surface is more or less uniform all over the area, depending only on soil and subsoil boundary conditions, a process which is much less sensitive to various parameters than that for the vertical component.

It has also been observed that in some regions over stiffer ground, the seismic intensities were higher than in regions over softer ground formations, something that is common in epicentral regions of several shallow earthquakes. On the other hand this observation comes to an agreement with the high water table level effect. Actually, the presence of the water increased the apparent stiffness of the ground since the first is a rather incompressible liquid, as already mentioned, along the vertical direction which is the direction of the emerging P waves. Therefore, the observed increase of the seismic intensity in harder grounds in the near-field of shallow earthquakes, is related to the dominance of the vertical seismic component of the ground motion, in such areas.

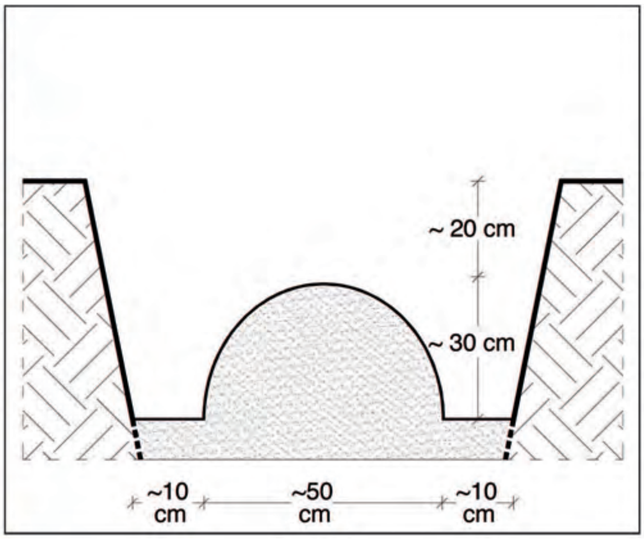
3. **Liquefaction and Lateral Spreading**

Liquefaction and lateral spreading were significant factors contributing to the extensive damage at the naturally or artificially illuviated areas of Port-au-Prince. Field studies carried out at the port of the city, showed that it suffered numerous surface manifestations of liquefactions and lateral spreading fissures that caused the collapse of the north wharf and the partial immersion of cranes, Fig. 3. The incapacitation of the port contributed largely to the difficulty of aid supplies and relief personnel to reach the region.

The liquefaction phenomenon is caused by the increase of the ground pore water pressure that penetrates through the interfaces between the grains or the soil particles of the ground. In this way, the friction among the particles is highly reduced. As a result, the water under pressure acts as a lubricant among the solid particles, the shear – mainly – strength of the ground is almost lost and the ground behaves as a liquid of high viscosity. The increase of the pore water pressure is due to the “primae” (P) waves which arrive first. The phenomenon of liquefaction, under the above mentioned definition, starts at the very beginning and well before the establishment of surface displacements. The water in the ground, as already mentioned, when compared to the fill ground material, has a rather high incompressibility and stiffness in transmitting the P waves with very small absorbsion of energy. The time lag between the

Fig. 5. *Port–au–Prince port area. The ejected material could form a continuous surface if the broken pieces are assembled together like in a puzzle. This shows that sand and water ejections started before the actual liquefaction and the lateral spreading phenomenon took place.*

Fig. 6. *Another documentation that the sand and water spouts in the port started before the strong ground shaking (please note the two parallel horizontal white signs on the tires which are due to bounding of the lorry platform on the ground during the strong phase of the ground motion. The ejected material was already there).*



(a) General view of the trenche with ridge of liquefied material inside it.

(b) Cross section AA

Fig. 7a, b. *Ridges of liquefied material was observed inside the trenches. Their crest was almost in the center and parallel to the trenches axis. The visible thickness of the ridge was about 30 cm almost uniform all along its length. The internal visible sides of the trenches were found clean, justifying in this way the spouted material continued taking place during the third phase too.*

initiation of the whole liquefaction phenomenon and the structural damages might not be very small. The whole liquefaction phenomenon that has been observed in the case under consideration has three distinct phases.

During the first phase, sand mixed with water is spouted from the ground forming craters, sand ridges, sand dunes, or larger areas of small thickness of sand. The first phase is expressed in larger areas of sand with a thickness of few mm up to 20 – 30 cm. It is supposed that the pressure of the ejected fine ground particles must be relatively small. Larger ground deformations do not take place at this phase since the larger particles of the fill ground material are still stable. In Fig. 5 it is shown that, despite of the large deformations of the ground surfaces, observed afterwords, one could assemble the broken pieces, like in a puzzle, and produce continuous surface with the material as it was before the braking. Therefore, according to the above given definition of liquefaction, this phenomenon did not occur during its first phase. Only sand and water spouts take place.

The second phase is the phenomenon during which large ground deformations, lateral spreading and translations vertically and horizontally take place, and the ground is broken into pieces along with the adjacent quay walls. The observed ejection of ground materials that started in the first phase continues. In Fig. 6, the white signs on the tires of the lorry platform are due to its bounces on the ground during the strong phase of the motion, where the ejected material had already been spread. This picture is another evidence that the sand and water spouts started before the actual liquefaction and lateral spreading phenomena. During this phase the actual liquefaction phenomenon takes place.

During the third phase, the ejection of ground material continues taking place, but gradually decreases until its completion. Not any significant ground deformation primary connected to the actual liquefaction phenomenon takes place. This phase is documented, among others, in Fig. 7 where fine liquefied material continuous to be ejected, well after the completion of the large ground deformations – main liquefaction phenomenon. According to this evidence, the ejected material – that possesses a high viscosity – formed inside the trenches symmetrical ridges along the central axis of the trench. This may mean that the material was still gushing up even after the integration of the destructive strong motion and the creation of the large ground displacements. If the liquefied material was inside the trench and after that, the trench became wider, the material should have come down having its concaves turned upwards. This is absolutely opposite to what it has been observed all over the many trenced created, Fig. 7.

This means that the ridge was created after the final formation of the trenches. Thus, it is more likely that the pore water overpressure that created the ridges inside the trenches was still active, perhaps under smaller pressure, even after the final integration and complete ceasing of the dynamic ground motion. This is evidenced by the slopes at the ridges that are still standing intact, Fig. 7b. These ridges should have become at least flat under the action of any dynamic vibration such as the seismic ground motion. The high water content of the liquefied material, decreasing its high viscosity, contributes also to the above mentioned argument. A logical scenario for the explanation of this phase, from the engineering point of view, is that

(a) Typical collapse of a contemporary building with reinforced concrete framework.

(b) A detail of (a), notice the inadequate reinforcement

(c) Jet components from different places around the building. Glazing is intact!

(d) Typical collapse of a multi-storied buildings with reinforced concrete framework.

(e) The arch-like lintels over the windows survived.

(f) The Presidential Palace. An almost symmetrical vertical collapse of the upper level.

(g) Typical street view after the earthquake.

(h) Informal housing destroyed on sloping ground,

Fig. 8a-h. *Representative pictures of the damaged buildings and structures. Most of the collapses occur within the ground plan of the building. To be continued by Fig. 8i-q.*

(a) Aerial view from S-E. The roof collapsed within the ground plan of the Port-au-Prince Cathedral.

(b) Aerial view from N-W side. All main walls of the Cathedral remain intact, vertically.

(c) Kalamata, Greece earthquake, M=6.0 R, 13 September 1986. The dome caved in the ground plan of the church.

(d) L'Aquila, Italy earthquake, M=6.3 R, 6 April 2009, Collapse of the Cathedral's dome.

(e) Lisbon, Portugal earthquake, M=8.5 R, 1 November 1755. A painting showes that the roof collapsed within the plan.

(f) Internal view of the Cathedral of Port-au-Prince. The roof collapsed within the plan of the church.

Fig. 9. *A study case of the damaged Cathedral Notre-Dame of Port-au-Prince. Comparison with other similar cases, in which the roof or dome collapsed inside the ground plan of the church while the walls and bell towers remained in their position.*

the opening of the trenches during the second phase took place quite suddenly and the pore water pressure dropped accordingly, but enough for the fine material to be ejected.

Finally, comparing the size of the material ejected during the third phase to that of the previous phases, one could conclude that the maximum discharged quantity of ejected material was produced after the integration of the above mentioned first and second phases of the whole liquefaction phenomenon.

4. **Basic Building Characteristics and Respective Damage**

4.1. *Historical and Traditional – Monumental Buildings*

Churches, Fig. 9a, the Presidential palace, Fig. 8f, buildings of various governmental organizations and ministries (as for example the ministry of Justice and Finance) and several other private buildings possess structural and architectural characteristics which are influenced by the western European architecture of the French – Spanish period. Especially the Cathedral of Port-au-Prince is quite similar to the Cathedral of Lisbon of the period of 17^{th} and 18^{th} centuries. The construction materials are rubble masonry with carved stones in the corners, brick walls, wooden roofs and floors. In some cases, a mixture of bricks and rubbles was observed for the construction of main load bearing masonry walls. Steel beams and other steel elements had been used for reinforcement. For bridging windows and larger openings, some structures based on dome or arch function had been used. Nevertheless, in some of these buildings, reinforced concrete elements were also found, like columns, slabs and beams.

The majority of these monumental buildings suffered from heavy damages to partial or total collapse. The collapse in the majority of the cases occurred within the perimeter of the buildings, Fig. 8f, while the most of the other walls remained standing in their position vertically. In accordance to this fact is the observation of unbroken glasses of the windows, even in cases that the building had suffered extensive structural damages. Furthermore, the arched lintels remained in their position, in spite of the fact that in many cases the rest of the building collapsed, Fig. 8e. This type of collapse could be attributed only to a strong vertical component of the ground motion which exceeds 1.0 g. The starting moment of the collapse should have been from 4 sec up to 7 sec, when the horizontal component of the emerging waves had not arrived yet. This was based on the fact that not any sign of significant horizontal motion had been noticed. The vertical collapse occurred abruptly, allowing no chance of reaction for escape or protection to the building occupants.

It is worth mentioning here that the two bell towers of the Cathedral remained almost in their vertical position, Fig. 9a. The Cathedral, up to the level of the base of the roof, presents a rather high and uniform density along the height. From this level and above, the construction members (roof and top part of the bell towers) present a significant mass discontinuity along the height, compared to the lower part of the church. The collapse of these members occurred along this horizontal level, as it is shown in Fig. 9b. After the collapse of the top of the structure within its lower part perimeter, the emerged compound system is a rigid body rather than a flexible dynamic system of a conventional structural character. Therefore, according to the observations, only in a few cases the horizontal seismic component completes the structural damages, since most of them had already taken place in despite of the fact that the horizontal component was quite strong. Unfortunately, there was no building of this category that could be identified to survive the strong vertical earthquake component and damaged by the horizontal one. It is worth mentioning here that this type of collapse (vertically and inside the ground plan) is a rather common finding in shallow earthquakes close to the building stock. This is shown in Fig. 9c, and Fig. 9d. Nevertheless, based on this fact, very important conclusions may be drawn related to famous historical earthquakes. One of those is the Lisbon earthquake M = 8.5 R, of 1755. In Fig. 9e a painting of that period represents the remainings of the Lisbon Cathedral after the said earthquake as well as some other structures to the right hand side of the picture. In Fig. 9f the remainings of the Port-au-Prince Cathedral show a quite similar response: the bell towers and the walls are still standing vertically, the collapse of the roof occurred inside the ground plan. A logical conclusion is that the Lisbon 1755 earthquake hit the city with a strong vertical component. This might mean that it should be very close to the city, in a small focal depth and perhaps of not so great magnitude. Probably, due to the other evidences, immediately after, a second earthquake located in the ocean (to the W of Lisbon) might be the one with that magnitude of 8.5 R.

4.2. *Contemporary Buildings and Structures*

The observation of the glass panels that remained intact in the aforementioned cases of buildings, also applies to the category under discussion. What is worth mentioning, is that there is not any official building code in Haiti. The erection of this type of buildings, in most cases, was carried out without any earthquake resistance design principles, just empirically. If some structures were constructed according to a certain code, this had happened due to the decision of the owner and not mandatory by law. And, of course, even in these cases the quality of the materials used and workmanship are questionable. In the majority of these structures, instead of the usual baked clay bricks, have been used hollow concrete bricks.

The contemporary buildings and structures in the region can be classified in the following four basic categories.

4.2.1. *Shanty – Makeshift Houses*

There are common in the area, one, or at most, two-storey houses made of cheap materials of low quality, old and in most cases of second hand. The erection is usually carried out offhand by local untrained builders and in a clumsy way, or by the owners of the house and its neighbors who are also not necessarily building technicians.

The building materials are hollow concrete bricks, wood beams or planks, sheet–iron and steel beams. These materials can be put at any part of the structure. For example, in the positions of the walls, or for roofing a space one could see wooden planks or sheet–iron. Each one of these shanty units is structurally separated

Fig. 10. *Typical view of the way of self constructing of low income masonry buildings.*

from the adjacent one, leaving a small space between them. In this way a rather dense network of internal and very narrow streets is formed. The pavements are generally formed out of picked out rubbish.

These shanty houses are concentrated in large numbers and high densities, where the poorest people live. In each of these houses (of a plan of about 5-7 m^2), a family of many members may live.

The water supply is provided by plenty of shallow wells located inside or in the fringes of these agglomerated settlements, since there is not any water supply system available. What is more, there is neither draining nor any power supply system in the wider neighboring area.

In many cases, these shanty houses present structural problems even without the occurrence of an earthquake. The most common are the deviation from the vertical axis, deformations of the walls perpendicular to their plane, strange deformities of the roof, erosion and general problems in stability and safety. In the lowest parts of these houses, it is often observed that they suffer from deterioration of the materials due to rising humidity from the underlying water table level. In order to eliminate this problem and to strengthen their houses, the owners used to plaster externally the walls with a lean layer in cement mortar – and at best – reinforced with a thin wire net for coops without repairing, before the application, the existing geometric defects. A similar, but often better method of construction can be observed in rural areas.

Generally these buildings did not suffer any major damage from the earthquake, except for some further inclinations from the vertical axis, some partial and localized collapses of roofs or walls.

4.2.2. *Load – Bearing Masonry Buildings*

The most common material used in this case, is the hollow concrete brick for the load bearing walls and slabs made of reinforced concrete. There is a further classification, of this category of buildings, into rural and urban buildings, according to the architectural style and the respective needs of residents.

The main structural characteristic of the urban buildings is that in the ground floors, in order to achieve large openings for the street-front shops in the facade, reinforced concrete columns had been used instead of masonry. The rural houses have one or two floors, while the urban ones may be higher, especially in the hilly areas, in the fringes of the city. It has been observed that in these areas where the slopes are very steep, at the uphill side, the house might be two storied while downhill, the same house could be even four storied.

In many cases, the corners of the building and the masonry walls had concrete columns at distances of about 3.0 m, enhanced by some fine steel bars longitudinally and transversely. The dimensions of these columns are quadratic and do not exceed the thickness of the wall (about 0.20 m). The concrete is usually molded at layers following successively the erection of the walls. The mixing of the concrete, as well as its placement is carried out by its owners or by unskilled labour. In Fig. 10, a representative case of this fact is shown. The majority of buildings in Haiti are of this type, where people of low to medium income live. Some single family dwellings, inhabited by people of higher income, could also fall in this category.

Generally, the class of structures under discussion suffered extensive damages with the following key features: the vertical collapse of floors and in most cases, within the floor plan of the building. A large contribution to the destruction must be attributed to the loose and unstable ground of foundation, as well as to the weak retaining walls as mentioned above. The collapse of the buildings was abrupt, directly downwards and the occupants had no time to respond to protect themselves or even escape. Many structures of this category were covered by various materials due to landsliding or due to collapse of other neighboring buildings. A typical picture is presented in Fig. 8h. The sliding buildings looked almost without any earthquake damage that could be attributed to the effects of direct earthquake ground motion. It seems likely, therefore, that the very loose and landsliding ground totally absorbed the direct and dynamic ground motion. The same could be stated for buildings founded over liquefied ground.

As mentioned above, there were some parts of the city where the damage and hence the inferred seismic intensity was extremely low or insignificant. However, a large number of similar buildings in this category either suffered extensive damage or collapsed while in other parts of the city they exhibited extremely slight damages or were completely undamaged. It must be added here that this observation is also valid for apparently similar soil conditions. Of course, it can't be accepted that the heavily damaged buildings are not earthquake resistant while neighboring and similar to them are seismically invulnerable. A large number of buildings suffered stronger damages on rather stiff ground conditions, rather than on softer ones. This is in agreement with observations derived from epicentral regions of other earthquakes of shallow depth.

4.2.3. *Buildings with a Reinforced Concrete Load Bearing System*

Since there is no building code legislation in the urban area, the resistance of high-rise buildings in earthquakes is questionable. The steel used in the same building could be of various types and qualities. The cross sections of the columns primarily and of the beams secondarily are found, at first glance as quite insufficient. The longitudinal as well as the transverse reinforcements are small in diameter and in large distances (see Figs. 11a, b and 12). There was no use of shear walls or wide columns. Columns of small quadratic cross sections were commonly used instead. From an architectural point of view, some of these buildings are quite interesting, with eminent structural demands, as for example large openings, long cantilevers, flat slabs etc. But the load bearing system does not follow as it is quite old-fashioned: the empty spaces, between the reinforcement, in the slabs of large openings are formed with hollow concrete blocks; there are not any rigid beams to column joints and the construction detailing is extremely poor. The use of soft stories is widespread at the ground floors along the main roads, mainly for commercial purposes.

Many multistoried buildings suffered extensive damage or collapsed. The type of collapse is of the pancake-type, where the upper floors fall over the lower ones without any horizontal translation. This is attributed to the strong vertical component at the very first phase of the seismic motion, due to the emerging waves (P). The buildings that were not damaged in the first phase of the earthquake were destroyed by the second one, as this can be justified from their horizontal displacements. The second phase is dominated by the horizontal seismic motion (S), which is more harmonic with longer predominant periods, larger displacements and longer duration.

Finally, in the same category belong some buildings which have not suffered any damage. This may be attributed to favorable soil and subsoil conditions, together with the convolution of different seismic waves, improved structural form and design and better dimensioning, detailing and construction.

4.2.4. *Steel Structures*

Most of the steel structures in the region behaved rather satisfactorily during this earthquake. Nevertheless, there was a large crane in the port that became unsafe for use after the earthquake. Around this structure, the foundation ground liquefied. There were also some gasoline station shelters that collapsed, but these must have been of an offhanded and rudimentary construction quality.

The old French shed behaved quite well in spite of the fact that its long side, which was close and parallel to the waterfront, subsided due to large ground deformations and liquefaction. This shed is externally isostatic and can be restored fully, since it is an historical landmark of the area. The simplest method is to uplift simultaneously the bases of all the columns that subsided.

The battery of silos, close to the port, did not suffer any visible damage, except an external ladder that was toppled.

The waterfront line of the harbor suffered extensive damages with large displacements, subsidence and liquefaction. It is logical that any structure above the ground fill in this area, the various cranes, piled containers, lorries and other cars were overturned and some of them sunk in the sea water.

The mast, in the port area, of cylindrical cross section, with a stepwise reduction of its diameter along the height, withstood quite well the earthquake with not any visible damage.

(a) Flat slab construction of the third floor. Inadequate reinforcement of small diameters and at large distances. Small dimensions of columns. Absence of any shear wall.

(b) A detail of a column to beam joint of the framing system shown in (a).

Fig. 11. *A typical case of contemporary building with reinforcement concrete framing system ready for concreting before the earthquake.*

5. Disaster Management and the Involvement of International Community

Management of the disaster caused by the 2010 Haiti Earthquake required differentiation and readjustment of the response actions that usually take place in situations of complex emergencies worldwide. When management of disasters requires the assistance of international community, the coordination of actions is the responsibility of the affected country's government, in cooperation with the UN. The 2010 Haiti Earthquake is the only case where the coordination of response actions was

Fig. 12. *A reinforced concrete framing system was ready for concreting before the earthquake as found after it. Total lack of any stiffening element. Note the beam at the center of ground floor with the triangle strut was added after the concreting of the floor.*

undertaken by another country's military forces, despite the existing humanitarian and peacekeeping activities of the UN in the area.

Social, political and economic conditions that existed before the earthquake and contributed to the development of complex emergency situation are:

– Lack of effective and reliable government services, as a result of many years of political instability, social unrest and corruption in the administration.

– Very poor country's economy with a GDP of around 8 billion U.S. dollars annually. That places Haiti in poor countries with nonexistent funds for spending on disaster prevention, on infrastructure, transport, health and education.

– Lack of basic spatial and urban planning, of land use plans, of construction plans and of regulations for safe reconstruction. As a result, there is chaotic and dense urban fabric in areas with high risk in natural hazards.

– Lack of planning of disaster management activities at all stages of pre-, co- and post-disaster. As a consequence, there was no appropriate machinery available such as cranes, electricity generators, mechanical jacks, lifts, e.tc. In parallel, there was complete lack of any know how for search and rescue. What's more, there was neither public informing nor any psychological support to those affected (3 million people approximately).

– Adverse economic and social conditions, low literacy population, lack of emergency goods, increased crime and high population concentration in certain areas.

– Weather conditions during the immediate post-disaster period were good for the homeless, but they were also favorable for the spread of disease, proliferation of reptiles, etc.

Pre-existing of the earthquake conditions formed a complex emergency situation with the following characteristics:

– Functional breakdown of the administrative system in general and especially that of emergency management. This may be partially attributed to the fact that most of the governmental buildings collapsed or became dangerous for re-occupation after the earthquake and many governmental officials died or were wounded.

– The U.N. building collapsed and many U.N. officials were killed, making the immediate participation of international assistance to the country difficult.

– Destruction of transport infrastructure and of the border posts made the access to the country during the first critical days impossible. In particular, the international airport was shut down due to significant damage to the main building and control tower. U.S. forces took over the operational control of the airport on 15th of January, allowing use only to US military forces.

– The port suffered considerable damage due to liquefaction and lateral spreading, making it dangerous for vessels with supplies and humanitarian assistance to approach. Limited use of the port was restored after 22nd of January, 10 days after the seismic event.

– The road linking Haiti with San Domingo (in the Dominican Republic) was out of use during the first two days after the earthquake, due to severe damage of buildings in the border posts. Additionally, the road in the border area was incapacitated to a distance of about 2 km, since it had been flooded by the water of the surrounding lakes.

– The increasing needs of medical supplies and the huge number of casualties overloaded the few operational health units that had managed to get to the area on time.

– The increased demands for potable water, food and camps caused starvation (or famine) in most parts of the city.

– There was a failure in management of cadavers, since there was no process of identifying them and they remained unburied for several days.

– There was complete lack of hygiene and health infrastructure in most parts of the city.

– Failure to coordinate actions led to the accumulation of the humanitarian aid at the airport of Port-au-Prince for several days.

– Some phenomena of crime, were noticed such as looting, robberies, shootings, assaults, arsons, kidnappings and attacks to rescue teams.

Relative improvement of the general situation and coordination of required actions were achieved only after the 27th of January, when members of the U.N and U.S military forces were finally able to distribute successfully humanitarian aid.

6. **Proposals for the Reconstruction**

The composition of the following proposals is based on the conclusion that the 2010 Haiti earthquake is mostly a man made natural disaster in a rapidly growing urban region of a country already extremely weakened by the adverse effects of long-term political instability which led to the collapse of its social, administrative and economic structures. Every disaster is unique in its circumstances, but commonalities also exist that apply to other parts of the World that had in the past or will face in the future similar emergencies.

Additionally, in this particular under consideration case it cannot be applied the generally accepted and applied goal of «return to pre-earthquake condition» since this situation has proven to be already dangerous.

Therefore, a completely new design should be implemented to minimize the primary and secondary effects of any natural disaster affecting this region such as earthquakes, floods, hurricanes, etc. However, it must be taken into account that the precautionary measures for one type of destruction are complementary and not incompatible with the other.

– Geotechnical and geological investigation of the quality and the characteristics of the ground, as well as an adequate microzonation study of the whole area. What will come out of these studies are the basic characteristics for the reconstruction of structures and the proper land use.

– A seismic hazard analysis as well as hazard maps for the most probable natural disasters threatening the city (earthquake, wind, rainfall etc.).

– A city planning study must be carried out in order to define the proper area for the installation of the critical facilities, the main and secondary streets, the administration and commercial centers, the industrial zone, the green areas etc.

– Definition and codification of standard design and construction of buildings.

– For the time being, any reconstructional activity must be prohibited, except those for simple repairs in order to provide immediate housing. Unfortunately, what had been observed even from the very first days after the earthquake was an activity of illegal housing, using materials detached from the collapsed or damaged buildings, which is quite understandable as the authorities were not able to provide alternative shelter for the homeless population.

– Construction of the necessary infrastructures (sewerage, drainage, water and power supply systems) and civil engineering works (streets, bridges, railways or subways), various governmental buildings (hospitals, fire brigade, police stations etc.), communal areas, parking areas etc.

– Specific measures for funding should be taken made to strengthen the legal construction industry development. For this goal the necessary building codes from countries of similar socio-economic conditions could be utilized. Among the first measures that must be taken is the re-allotment of the land, based on the new city planning study.

7. **Discussion and Conclusions**

The existing historical and traditional structures in Haiti possess basic similarities with the historical and traditional structures of western Europe, due to the Spanish and later French colonization of Haiti in the past (Haiti proclaimed its independence in 1804). For similar reasons, western European architecture has influenced many other countries around the world. Therefore, logically, the above mentioned category of structures of Haiti, possess similarities with other historical and traditional structures around the world in countries that have been influenced by some means from the western European Architecture. Valuable lessons could be drawn from the earthquake response of this category of structures after the Haiti earthquake, which are also applicable to many other countries of the world of similar seismotectonic conditions. On this point a very interesting example is the presented case of the two damaged Cathedrals, that of Port-au-Prince after the present earthquake, and that of Lisbon after the 1755 earthquake.

Most of the direct damages on building structures as it is documented, are attributed to the vertical component of the ground shaking in the near field. This has been observed also in many other shallow earthquakes, /1/, /7/ and /8/. In these cases, the seismic intensity is almost at the maximum and is almost independent of the earthquake magnitude. In larger magnitude events, the area of maximum intensity, influenced by very strong vertical ground motion, becomes larger, in relation to the fault dimensions and fault plane solution.

Another fact worth mentioning is that the building collapses occurred most commonly along the vertical direction (buildings collapsing within their own footprint) and within the very first moments of the ground shaking and occurred by taking place abruptly in a brittle manner. The inhabitants had no time to react by escaping or protecting themselves and therefore the lethality rate was really high.

As long as the vertical component of the ground motion remains at a level below 1.0 g, the damage potential is trivial. Once the vertical accelerations exceed this limit, the results start to be disastrous. And this is actually the case under consideration, especially in areas nearest to the fault rupture.

What has been also proved in this earthquake is that the damages have no causative relation with the dynamic characteristics of the structures. Furthermore, it has been observed that some structures with a rather good quality of reinforced concrete load bearing system suffered almost equal damages to other structures of not so good quality. This finding needs further explanation about the above mentioned quality. Actually, this term refers to the capacity of the structure to resist, mainly, earthquake loads along the horizontal direction and not along the vertical one. Of course, a well designed and constructed structure according to the state of the art methods has little to fear from the vertical seismic component, since it can resist it with the surplus of the provisions taken for the horizontal component.

It has also been observed that some structures have not been damaged almost at all, although they were very close and on the same soil conditions (almost adjacent) to other similar structures that suffered extensive damages or collapse. This difference can not be attributed to the inferior quality of construction of the damaged structures compared to the non damaged ones, since in both cases building were of poor quality. This can be attributed to the input motion differentiation. Further, this difference can be attributed only to the convolution of body (P and S, direct and reflected ones) and surface waves. The bowl shaped subsurface relief of the basin may have played a basic role in this mechanism. Similar observations have also yielded for many other cases of destructive shallow earthquakes /1/, /5/, /6/, /7/ and /17/.

Opposing to the general opinion, a rather surprising finding is that in some locations systematic heavier damages had been observed in harder soil conditions and not in softer ones. In other similar cases, indications are confirming this finding, /7/. This may be attributed to the special dynamic characteristics of the vertical component of the ground motion.

By analyzing the observed response of structures, one could draw valuable conclusions about the ground motion (direction, magnitude and even predominant frequencies). The simplest the structures, the better and safer is the reliability of the results. This principle is applied in the present case with the following example of the traces on the base of a fence, helped in estimating the direction and magnitude of the ground motion, considering these traces as a record of a simple seismoscope. Its dynamic characteristics were measured at the spot. This finding is quite interesting since instrumental records are absent in the case of the Haiti earthquake.

The derived conclusions about the liquefaction phenomenon that started before and continued after the integration of the strong phase of the ground shaking, might not be valid for strong distant earthquakes under which the ground is excited and dynamically settles. In this case the liquefaction phenomenon follows the starting of the strong phase of the shaking and does not precede it.

The January 12, 2010 Haiti earthquake, is undoubtedly a natural disaster. Several important secondary effects though, resulted from the disorganized way of dealing with it and the lack of adequate infrastructure. Therefore, it could be categorized as a man-made natural disaster and if the appropriate precautions are not going to be under serious consideration, the same or worse damage will be repeated in the future.

This particular earthquake taught us that preparedness, existing infrastructure and know how, are the key factors in reducing the impact of such a disastrous natural event.

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RIASSUNTO ESTESO

Il terremoto di Haiti del 12 gennaio 2010: osservazioni sul comportamento delle costruzioni ed interpretazione dei danni della zona di Port-au-Prince

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Il terremoto distruttivo di magnitudo M_w 7.0 che ha colpito la Repubblica di Haiti il 12 gennaio 2010 (alle 4.32 pomeridiane) è stato uno dei più distruttivi e fatali tra quelli che hanno infierito nell'area il secolo scorso. L'epicentro fu a circa 20 Km dal centro della capitale, Port-au-Prince, dove gli autori di questa nota si sono recati dal 16 al 22 dello stesso mese al fine di raccogliere informazioni e dati sicuri. Inizialmente il fenomeno di rottura si è sviluppato ad una profondità focale piuttosto modesta ed è stato poi seguito, nello spazio di 20 minuti, da due ulteriori grandi scosse. Inizialmente si è pensato che l'ipocentro fosse situato lungo la faglia denominata Enriquillo-Plantain Garden Fault Zone (EPGFZ), ma successivamente a seguito dell'impiego di sistemi GPS e di misure radar-interferometriche, /4/, è emerso che la posizione della zona di rottura si trovava lungo una faglia non mappata (faglia Legane) che si diparte dalla faglia maggiore EPGFZ e si trova a sud del fronte di spinta di Haiti.

È importante rilevare la mancanza, in tutta l'area, di segnali registrati durante il terremoto. A causa di questa mancanza, gli autori si sono proposti, durante la loro visita, di raccogliere dettagliate osservazioni macrosismiche, considerando la risposta di alcune strutture come grossolani strumenti 3-D, capaci di fornire informazioni 'congelate' sul terremoto. Ad esempio, gli autori hanno osservato strappi simmetrici dei fili metallici alla base di una rete di recinzione. Il recinto consiste di diversi pannelli di rete metallica, uguali tra loro, la cui risposta è stata prevalentemente nella direzione ortogonale ai loro piani. Il massimo spostamento in corrispondenza del cordolo in cls. di base, in c.a., è stato di circa ±13cm. Il recinto si trova a circa 14 Km a NE dell'epicentro, vicino all'Hotel Montana, crollato con molte vittime. Gli autori hanno misurato le dimensioni di un pannello e stimato le sue caratteristiche dinamiche (smorzamento e periodi propri nelle due direzioni principali) in modo assai grossolano: spostando a mano il pannello in corrispondenza del suo baricentro e poi rilasciandolo bruscamente. Le oscillazioni smorzate conseguenti, gli spostamenti, il numero di cicli ed il tempo intercorso hanno costituito la base per la stima grossolana di cui si è detto, necessaria stante la assoluta mancanza di strumentazione. Tali dati sono poi stati impiegati come confronto nel corso di simulazioni numeriche con ETABS condotte al ritorno ad Atene.

Si è intrapresa l'analisi inversa (cioè determinare le caratteristiche dell'eccitazione di base dalle risposte) impiegando un metodo di identificazione, detto di "trial and error" (prova e correggi), con l'obiettivo di ottenere il valore dello spostamento detto (circa ±13cm) considerando diversi segnali di input di base, coerenti con lo spettro elastico di tipo 1 dell'EC-8 e per il terreno B. In questo modo si sono stimate le caratteristiche fondamentali dell'eccitazione come valor medio dei segnali di input che conducevano al detto valore di spostamento. Le caratteristiche principali del moto del terreno durante la fase principale del moto di base sono: accelerazione di picco da 0.6 g a 0.7g, spostamenti del terreno da 5 a 20 cm, durata dell'evento 5 sec. Valori simili sono stati ottenuti impiegando alcuni accelerogrammi reali, previa la normalizzazione delle loro ampiezze, registrati durante gli eventi di Kobe, 17 /1/1995, e L'Aquila, 6 /4/2009. Al fine di convalidare le loro osservazioni sul campo in merito al moto del terreno, specialmente secondo al direzione verticale, gli autori hanno tentato di ritrovare affinità tra le risposte ('congelate') di alcune strutture di Haiti con quelle riscontrare su strutture simili, colpite da terremoti ma strumentate, situate in altre parti del mondo da loro visitate di recente. I siti considerati sono, come esempio, Kobe (1995), L'Aquila (2009), Darfield (2010) e Christchurch (2011). Come risultato dei confronti da loro condotti, gli autori hanno concluso che l'accelerazione verticale ha raggiunto, ad Haiti, il valore di picco di 1g. Va posto in rilievo, in proposito, che la risposte delle strutture alla componente verticale con questi valori di picco risulta piuttosto indipendente da diversi parametri strutturali, dalle caratteristiche del terreno e da quelle sismotettoniche, a differenza di quanto avviene per la risposta all'eccitazione orizzontale con paragonabili valori di picco.

Grazie alla colonizzazione spagnola e la successiva francese di Haiti, molti dei suoi monumenti hanno caratteristiche comuni a quelli dei paesi dell'Europa occidentale. Per le stesse ragioni tale architettura europea ha esercitato una significativa influenza in molte altri paesi del mondo. Perciò sussistono logiche ragioni circa il fatto che i risultati delle indagini condotte in merito alla risposta sismica delle strutture tradizionale e storiche di Haiti possano valere anche per sistemi simili di zone epicentrali del mondo in occasione di terremoti superficiali. Questa circostanza potrebbe esser utile al fine di intraprendere adeguate misure preventive per la sicurezza, in quanto tali eventi possono manifestarsi in ovunque, specie in presenza di faglie non mappate. La grande maggioranza di tali edifici monumentali ha subito gravi danni, crolli parziali o crollo totale. Nella maggioranza dei casi il collasso si è manifestato dentro il perimetro dell'edificio, mentre le arcate perimetrali sono risultate prevalentemente quasi intatte. Il caso della Cattedrale di Notre-Dame, a Port-au-Prince, ha attirato l'attenzione degli autori, specie grazie alla sua affinità con la Cattedrale di Lisbona, i due monumenti risalgono al XVII ed al XVIII secolo. Basandosi sulle immagini relative al grande terremoto di Lisbona del 1755 gli autori giungono alla conclusione che le due simili strutture sono state soggetto a danni simili (crollo interno). Lo stesso quadro di danno è stato osservato in occasione dei terremoti di Kalamata (13 Settembre 1986, M = 6,0 R) e dell'Aquila, (6 Aprile 2009, M = 6,3 R), nei quali la componente verticale dell'eccitazione è stata assai importante. Inoltre gli autori sono pervenuti alla conclusione che il già citato terremoto di Lisbona fosse molto probabilmente costituito da due eventi in serie: il primo, piuttosto superficiale, di magnitudo moderata, ed il secondo di profondità elevata con epicentro nell'oceano; a quest'ultimo si riferiscono generalmente molti ricercatori.

Ad Haiti c'é un'altra categoria di strutture costituite da grandi agglomerati di abitazioni, consistenti prevalentemente di capanne precarie, autocostruite con materiali di risulta. In tali abitazioni vivono le famiglie più povere, la loro superficie è di circa 5-7 mq. In generale queste abitazioni non hanno subito gravi danni durante il terremoto.

Un'altra numerosa categoria di abitazioni contemporanee è costituita da edifici con pareti in muratura e solai in c.a. La muratura è realizzata con blocchi forati in cls. In molti casi questi edifici presentano agli angoli e lungo le stesse murature (a distanza di circa 3m) colonne in 'c.a.' La virgolettatura è necessaria in quanto le armature di tali colonne è in generale assai scarsa ed il conglomerato è di bassissima qualità; le sezioni di tali colonne sono in genere all'interno dei 20 cm. di spessore delle pareti. Tali edifici sono di 1-2 piani ma in periferia si possono trovare edifici fino a 4 piani. Un gran numero di queste costruzioni ha subito gravi danni o il crollo totale.

Gli edifici multipiano adibiti ad uso alberghiero, ad uffici o di tipo residenziale sono costruiti con sistema resistente costituito da telai i. c.a. Data l'assenza, nell'area, di norme tecniche ed in particolare di norme sismiche era lecito attendersi un pessimo comportamento di tali edifici, con danni estesi e crolli. Questo è quanto si è in effetti verificato: la modalità di crollo più frequentemente osservata è costituita dall'affastellamento dei solai uno sull'altro, con i solai dei piani superiori adagiati su quelli dei piani inferiori.

Il comportamento delle strutture in acciaio presenti nella regione è da considerare soddisfacente. Alcune di tali strutture, situate nella zona del porto, si sono inclinate per effetto di fenomeni di liquefazione. Tali fenomeni hanno comportato molti danni in corrispondenza delle zone paludose della città e si sono manifestati prima della fase principale del terremoto per poi estendersi dopo che questa si è esaurita. Il sopralluogo al porto ha riscontrato numerosi episodi di liquefazione e di diffusione laterale che hanno condotto al collasso del molo Nord ed allo sprofondamento di molte gru.

Una conclusione del nostro studio è che il disastro verificatosi è prevalentemente dovuto all'azione dell'uomo. Questo a causa delle infelici condizioni sociali, politiche ed economiche presenti ben prima del sisma, che hanno comportato l'assenza di ogni normativa tecnica, di procedure di controllo nell'uso del territorio e dall'assenza di servizi pubblici durante e dopo l'emergenza.

Considerate queste inefficienze, gli autori propongono infine alcune misure che si dovrebbero attuare prima della ricostruzione in modo che da questo disastro possano crearsi i presupposti per una vita migliore e più sicura. Per esempio: bisognerebbe predisporre mappe di microzonizzazione per un migliore uso del territorio; andrebbero predisposte normative tecniche e sismiche per le costruzioni, considerando gli sviluppi più recenti delle discipline coinvolte; andrebbe pianificata la costruzione di infrastrutture (smaltimento delle acque, fognature, acquedotti e linee elettriche). Nessuna attività di ricostruzione dovrebbe aver luogo prima della realizzazione pratica di quanto sopra.