

BUILDINGS BEHAVIOUR TO SEVERE SURFACE EARTHQUAKES (CHILE 2010, NEW ZEALAND 2011, TAIWAN 2016)

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Abstract

The paper presents the analysis of collapse and damage state to some special buildings during recent severe surface earthquakes. The presented irregular reinforced concrete buildings (with irregular distributions of mass, stiffness, strength, geometrical configurations), especially the setback structures and tall structures, have exhibited a poor seismic performance. Through graphical representations of response spectra obtained from records, it is revealed that most of the damage occurred to buildings due to exceeding the code requirements, the level of accelerations and to the spectral composition, unusual to the surface earthquakes, which justify the shown interest. Comparative analysis of numerical data is shown in order to put into evidence the effects of surface earthquakes on buildings. It must be accepted that an important role in the overall behaviour of the presented buildings had the higher modes of vibration and shaking time, beyond the technical aspects. What is noticeable, also, are the long periods of motions, although the earthquakes are of surface. The more accelerograms recorded are, we find that the possible values may be higher than those from design codes and sometimes the codes provisions are taken from other design rules (with other local features).

Key words: seismic records, severe irregularities, long periods, damages.

INTRODUCTION

The analysis of collapse and damage to some special buildings is made for these severe surface earthquakes:

Chile earthquake of February 27, 2010: $M_w = 8.8$, $a_g = 0.65g$, $a_g = 0.14g - 0.47g$ in Santiago de Chile (with $a_g = 0.40g$ as the minimum current design code value), duration of shaking 3min (40-50s strong vibration), Pacific coastline, depth source 35 km, 320 km south-west of the capital Santiago de Chile and approx. 100 km north of Concepcion, the second city of the country.

Christchurch-New Zealand earthquake of February 22, 2011: $M_w = 6.3$, depth source 5km, $a_g = 1.68g$ ($a_g = 0.5g \dots 0.9g$ in many locations in the city), duration of shaking 10sec, on an alluvial plain near a volcanic chain.

Taiwan earthquake of February 8, 2016: $M_w = 6.4$, depth source 16.7 km, duration of shaking 20...44sec, $a_g = 0.3g$, the most damaged buildings scattered in the east area of Tainan.

MATERIALS AND METHODS

SEISMIC RECORDINGS AND OBSERVATIONS

Regarding Chile earthquake 2010 effects, collapse mechanisms, structural assessment.

a. Collapse of 15-storeys building Torre Rio Alto of Concepcion, $H = 42m$ (it was completed in 2009), Figure 1.

The structural system of the building consisted of an array of reinforced concrete structural walls and pilasters, designed taking into account the earthquake (thin structural walls, which support concrete slabs without

intermediary beams, with many openings and structural discontinuities; the walls had a thickness of 200mm, L and T shapes, many of them were retired from outside of the building at the first two levels).

The period of vibration for the first mode of vibration was approx. $T=1.09s$ (Kohrangi et al, 2010).

Its collapse was not the most destructive type because the structural walls have ensured a high rigidity and did not allow a "sandwich" overlapping of floors, considered most dangerous for residents.

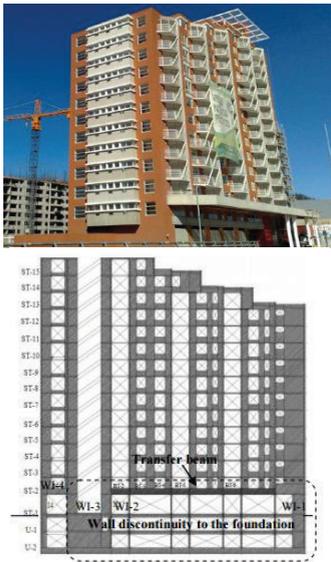


Figure 1. Some severe irregularities and discontinuity of walls at the base of the structural system (Kohrangi et al, 2012)

It has been found that, Figures 1...5:

- building was in an eccentric position respect to the whole basement;
- some severe irregularities and discontinuity of walls at the base of the structural system;
- the transverse reinforcement was not done properly with sizes of building and the wall thickness was not increased proportionately to reflect a greater height regime;
- the lack of bulbs at the ends of the structural walls;
- the relatively high levels of wall axial load (the wall axial tensile forces were probably underestimated) etc.

Also, the walls on the first floor and the basements (parking lot) are smaller than those on the second and above floors in both length and there is a difference in wall volume and layout among basement, first floor, and second floor, Figure 2, 3.

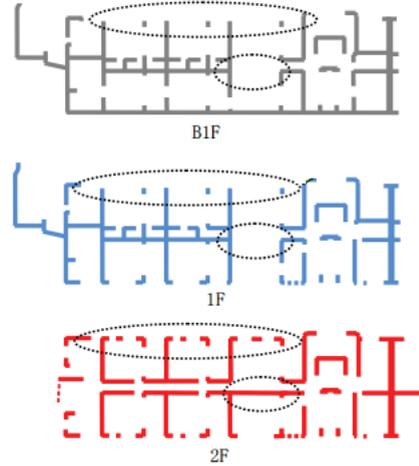


Figure 2. Difference in wall volume and layout among basement, first floor, and second floor (Saito et al, 2010)

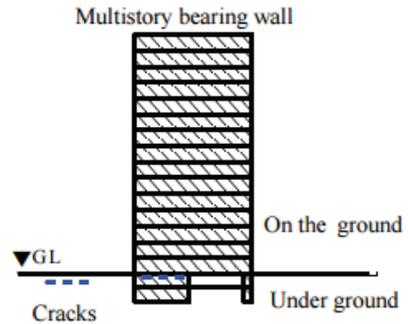


Figure 3. Schematic illustration of multi-story bearing wall with large opening at basement (<http://www.kenken.go.jp>)



Figure 4. Damage modelling and view of collapsed building (<http://www.nehrp.gov/pdf>)

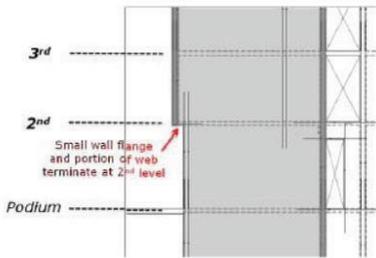


Figure 5. Details of vertical discontinuities of a shear wall (http://www.nehrp.gov/pdf/GCR%2014-917-25_RecommendationsforRCWallBuildings_std.pdf)

From this fact, the damages on the wall footing on the basements have resulted. The walls failed in the first story, causing the overturning of the entire building.

- b. Partial collapse of 23-storeys modern tall building Torre O'Higgins, $H=73m$ (it was completed in 2008 and demolished in 2012), Figures 6...9.

The reinforced concrete structural system of the building consists of a dual system of frames and shear walls (the shear walls positioned in the west side and on the other sides of the columns and beams), was designed to the latest Chilean seismic code and its failure was not anticipated, given the construction standards). The period of vibration for the first mode of vibration was approx. $T=0.9s$ (Silva et al, 2013).



Figure 6. Torre O'Higgins building before earthquake

The building experienced dramatic failures in several of its upper levels above mid-height (12, 16, 20), each coincident with a framing setback, that were likely a result of stiffness discontinuities and vertical irregularities in the structure.

The localized failures on the south and east faces were most likely due to the horizontal and vertical irregularities of the structure (irregular torsional stiffness, structural wrong torsion effects combined with structural defects). The perforated shear walls on the east face and south face showed damage to both wall piers and spandrels.

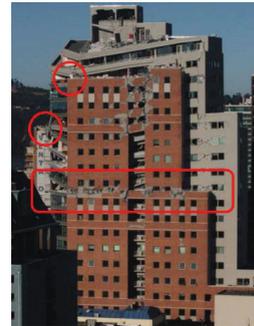


Figure 7. Torre O'Higgins building before earthquake and after, with dramatic failures in several of its upper levels (12, 16, 20) (Decanini, 2010)



Figure 8. Details of the main collapse at story 1 (Decanini, 2010)



Figure 9. Details of facade failure, high axial/shear and bending forces (Decanini, 2010)

The causes that lead to shear wall failures are:

- low percentage of longitudinal and transversal reinforcement;
- rigidity variations in the elevation of the structures (wall thickness variations in the height of the building);

- due to the higher modes of vibration and large deformations of bending, inelastic shear values can develop much higher values (than those prescribed by the Eurocodes).

Regarding New Zealand earthquake 2011 effects, collapse mechanisms, structural assessment.

a. Collapse of 5-storeys Pyne Gould Building Co., $H = 15\text{m}$ (it was built in 1964), Figure 10...16.

The reinforced concrete structural system of the building consists in a six-storey five-by-five bays frames (frame with a core wall offset towards the rear of the building).



Figure 10. Pyne Gould Co. building in October 2010

The lateral load resisting system consists of 200 mm thick RC core walls with two 15 m long RC walls, acting in the North-South direction, and three shorter (two 5 m and one 2.6 m long) RC walls along, acting in the East-West direction).

The period of vibration for the first mode of vibration was approx. $T = 0.7\text{s}$ ($0.09 \cdot H^{0.75}$ for RC moment-resisting frame).

Vertical stiffness irregularity, observed in construction plans, explains the collapse of type “sandwich” at the first level, with the collapse of the floors along the east-west axis of the building.

The poor performance led to the collapse, Figures 10...16:

- because it was a building built before the introduction of modern seismic design codes (1963 and 1964)
- due to the building had several critical reinforcement deficiencies (lack of shear reinforcement of nodes, inadequate anchoring beam-column/wall;
- soffit of slab showing where slab reinforcement has been ripped from slab;

- typical perimeter beam/column joint, with lack of joint ties;
- fractured slab reinforcement at wall/roof slab interface on the east side of the shear core etc.)

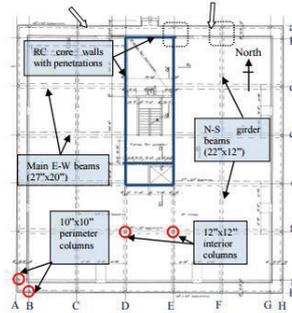


Figure 11. Plan view of the typical upper floors (2nd to 4th Floors).

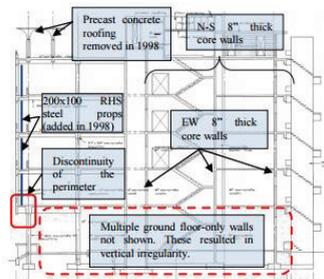


Figure 12. North-South elevation on Grid Line D.

Figure shows the East-West cross elevation view, which indicates some of the openings in the North-South 15 m long walls. The shorter RC walls have significant openings (two door openings of approximately 850 mm x 2.200 mm dimensions).

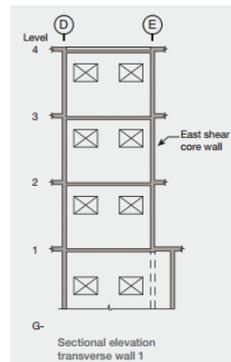


Figure 13. Sectional view
 (www.file:///C:/Documents%20and%20Settings/winxp/My%20Documents/Downloads/Final_Report_Volume_2_Web.pdf)



Figure 14. General Location of Main Structural Components after Collapse (<http://www.building.govt.nz>)

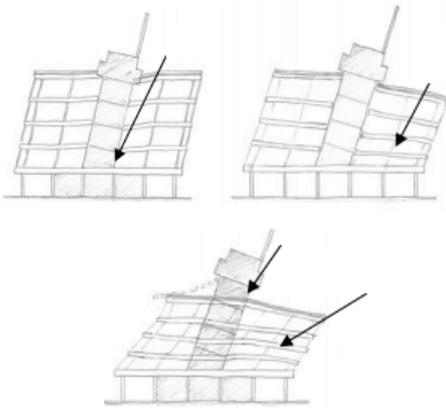


Figure 15. Time evolution of failure. Crushing of wall concrete, Failure of column/joints in compression, Failure of slab/wall connection, Failure of steel props (Beca Carter Hollings & Ferner Ltd (Beca) 26th September 2011)



Figure 16 a. Various failure mechanisms observed on the Northern elevation of the collapsed PGC building (<http://www.nzsee.org.nz>)

The building collapsed when the reinforced concrete walls of the core of the structure between level 1 and level 2 failed. Subsequently, the perimeter columns and/or joints between the columns and the beams and

the connections between the floor slabs and the shear-core failed, causing the floors to collapse.

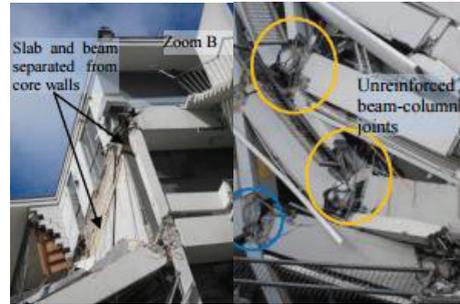


Figure 16 b. Various failure mechanisms observed on the Northern elevation of the collapsed PGC building (<http://www.nzsee.org.nz>)

b. Collapse of Grand Chancellor Hotel, $H=85m$ (it was built in 1986), Figures 17...25. Grand Chancellor Hotel was the city's tallest building from Christchurch, a reinforced concrete frame structure (no. levels=26), with some vertical and horizontal irregularities. The period of vibration for the first mode of vibration was approx. $T=2.5\text{ s}$ ($0.09 \cdot H^{0.75}$ for RC moment-resisting frame).



Figure 17. View of the Grand Chancellor before 2011 earthquake (<http://www.building.govt.nz>)

Vertical irregularity arises from the fact that the upper tower relies on frame action (moment-resisting reinforced concrete frames) for its seismic resistance while the lower tower relies on reinforced concrete shear walls. The two structural forms inherently have different stiffnesses and, if not linked, would respond differently to seismic shaking. The horizontal irregularity arises from the fact that the eastern bay of the building is cantilevered. Large cantilever transfer beams

extend out to the east at levels 12 to 14 above Tattersalls Lane to support the car park floors. The wall with a plate cantilever supported between 600 and 800 tons from weight higher levels. It is estimated that during the earthquake, this wall has undergone a request side 1800 to 2400tf. Two of these cantilevered transfer beams sit on top of the key supporting shear wall (wall D5-6), Figure 17. The damaged foundations lead to the visible leaning of the building to one side, Figure 18.

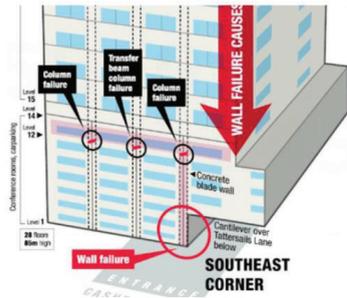


Figure 18. Failures of columns, transfer beam column, concrete wall (<http://sciblogs.co.nz>)

The wall failure and the south-east corner of the building dropped by approximately 800 mm and developed an accompanying horizontal lean of approximately 1300 mm at the top of the building, Figure 19.

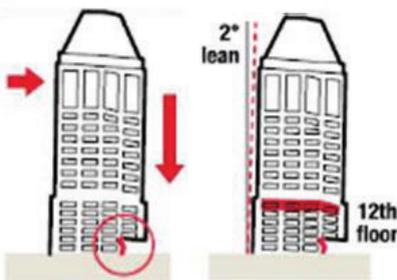


Figure 19. Wall failure and the south-east corner of the building (<http://sciblogs.co.nz>)

The building suffered structural damage, caused by the collapse of a key supporting shear wall "D5-6" located in the south-east corner of the building, Figures 20...23. The shear wall was responsible for roughly one-eighth of the Hotel Grand Chancellor's mass.

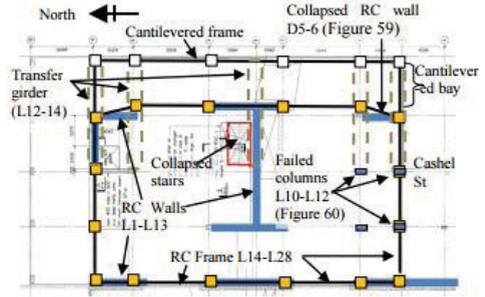


Figure 20. (<http://www.nzsee.org.nz>)

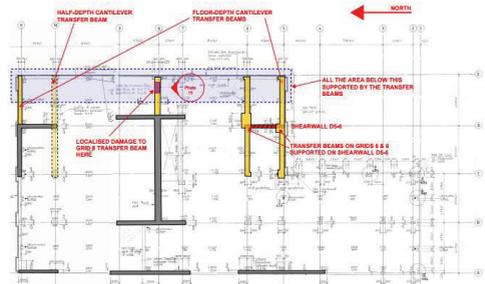


Figure 21. Level floor plan, with the shear wall D5-6 (<http://www.building.govt.nz>)



Figure 22. Shear wall D5-6 base failure. A lack of steel reinforcing in the concrete was the biggest factor in the shear wall's failure which failed in a brittle manner. This shear wall had supported vertically approximately one eighth of building's mass and was also expected to carry a portion of lateral earthquake loads

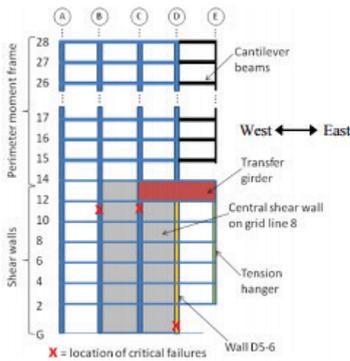


Figure 23. Schematic plan and elevation of the Grand Chancellor Hotel. The floor numbering is based on the original construction drawings – the lower 14 floors are half-height car park floors (<http://www.nzsee.org.nz>)

high-rise Tower was completed in 1994), Taiwan, Figures 26...29.



Figure 26. Weiguan Jinlong residential building before earthquake

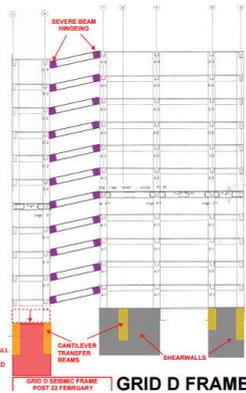


Figure 24. Details of cantilever transfer beams, with severe beam hinging (<http://www.building.govt.nz>)

This high-rise residential building is a mixed-use reinforced concrete structure, an U-shaped residential complex without walls on the ground floor.

The period of vibration for the first mode of vibration was approx. $T=1.5s$ ($0.07 \cdot H^{0.75}$ for RC moment-resisting frame)



Figure 27. Collapse scheme (H. Stone, 2016)



Figure 25. Grand Chancellor Hotel: The shear-axial failure of the RC columns below the transfer girders at Level 10 and 11.

There are some reasons for its collapse (H. Stone, 2016, nonsolidground.blogspot.com):

- the design of the construction did not match calculations made in its structural and steel blueprints;
- the difference in stiffness between the apartments above and the soft storey below causing failure at the junction between the two (at second floor level);
- at the bottom of the building, a possible soft storey which has caused the overturning of the entire structure;
- the possibility of structural problems related to poor-quality reinforced steel and cement;
- only half the designated amount of reinforced steel beams were used in the construction of the complex, which explains its weak structural integrity;

Regarding Taiwan earthquake 2016 effects, collapse mechanisms, structural assessment.

- a. Collapse of 17-storeys Weiguan Jinlong residential building in Yongkang District, $H=52m$, Tainan (Weiguan Golden Dragon

- there is a high likelihood that the basement acted as a soft storey and caused the building to collapse;
- some residents in the building broke and cleared many in fills and columns of a storey so that more rooms could be arranged for rent.

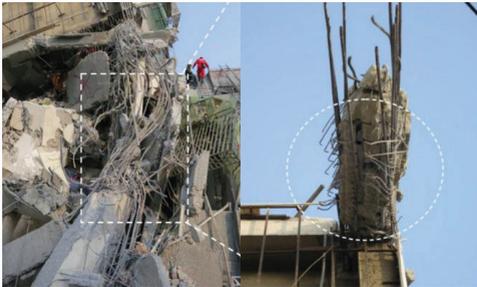


Figure 28. Some 90-degree bend hooks (<http://www.businessinsider.com>)



Figure 29. (<http://www.eqclearinghouse.org/2016-02-taiwan/files/2016/02/2WeiguanBuilding-3.pdf>)

COMPARATIVE PROCESSING DATA

- The recorded accelerograms for Chile *surface earthquake 2010*, at San Pedro Colegio Concepcion (CCSP), 2 horizontal components, and INCERC accelerogram, NS component, registered at Vrancea *intermediate earthquake 1977* (using the same scale) are shown in Fig. 28 [Dragomir et al, 2010].

The recording time of Vrancea seismic motion was approx. half from the recording time of Chile earthquake and the maximum amplitude of the acceleration recorded in Chile is about 3 times higher than the maximum of 1977 recording.

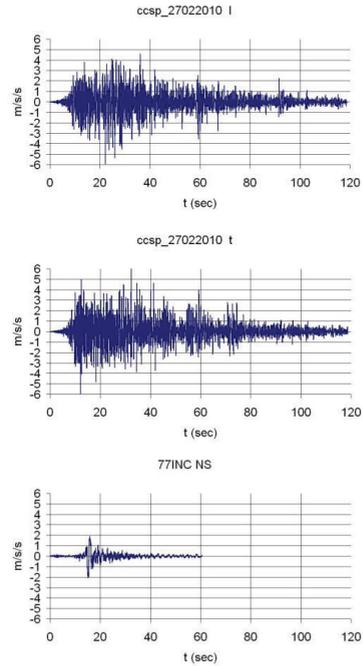


Figure 30. Recorded accelerograms for Chile earthquake 2010, at San Pedro Colegio Concepcion (CCSP)-2 horizontal components, and INCERC accelerogram- NS component, registered in Vrancea earthquake 1977 (Dragomir et al, 2010)

Response spectra in absolute acceleration (5% fraction of critical damping) for the horizontal components of recording from Colegio San Pedro Concepcion and NS component recording from INCERC Bucharest, with the spectrum of Chilean Code NCh433/96 and spectrum from Code P100-1/2006 (corresponding to an earthquake with a mean recurrence interval reference TNCR = 100 years) and for TNCR = 475 years are shown in Figure 29.

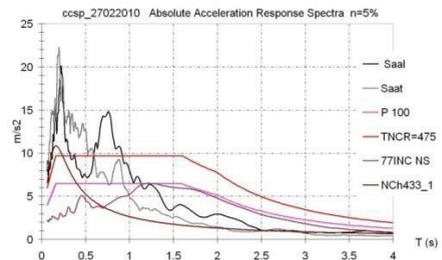


Figure 31. Comparative analysis of response spectra in absolute accelerations (Dragomir et al, 2010)

➤ The recorded accelerograms for Taiwan 2016

The response spectra have the followings characteristics:

- maximum acceleration 0.22 g EW
- spectrum E-W was approached to the spectrum code with return period of 475 yrs. and it was shifted to the period of 1 s
- the spectrum of the N-S remained in the spectrum code area for return period of 475 years but showed some major peaks between 0.6 s and 0.9 s

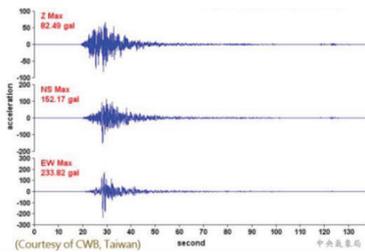


Figure 32. Recorded maximum accelerograms in Tainan City, $a_Z=0.82\text{m/s}^2$; $a_{NS}=1.52\text{m/s}^2$; $a_{EW}=2.34\text{m/s}^2$

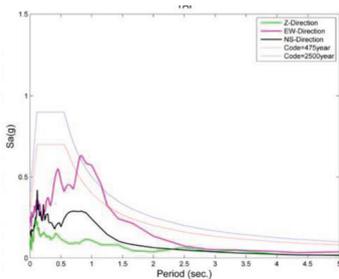


Figure 33. Response Spectra Tainan 2016 earthquake (Stone H, 2016, nonsolidground.blogspot.com, accessed February/March 2016)

RESULTS AND DISCUSSIONS

The comparative analysis of presented numerical data, for Chile 2010 earthquake and Vrancea 1977, leads us to the following conclusions (Dragomir et al, 2010):

- in respect of the peaks of the accelerograms, the records from Chile show higher values than those obtained on 4 March 1977 Vrancea in INCERC;
- in respect of the control period (corner) of the response spectrum, T_c , for the recordings in Chile, it does not exceed 1s, indicating that the entire spectral

amplification occurs to the left of this value (therefore, all records in Chile betrays a seismic motion of short period);

- the maximum value in the range of 0.2 s of Chile Code was exceeded by about 100%, while in the period $T = 0.7\text{s}$ the increase was approximately 250%; in this way, one can explain how some tall buildings of 20 floors, such as the building Torre O'Higgins in Concepcion, were badly damaged.

In terms of peak accelerations recorded in Taiwan, resilience depends greatly on the unique characteristics of the shaking at the particular site.

These characteristics include: duration and number of cycles of strong shaking; predominant frequencies of ground shaking; any directionality in shaking etc.

About the level of accelerations in Tainan city, they are only with 15% higher than the INCERC 1977 record, but they destroyed buildings with quantitatively consistent reinforcement, but with deficient compliance and detailing.

CONCLUSIONS

What is noticeable, are the long periods of motions, although the earthquakes are of surface.

The irregular reinforced concrete buildings, especially the setback and tall structures, exhibit generally a poor seismic performance. It must be accepted that an important role in the overall behaviour of the presented buildings had the higher modes of vibration and shaking time, beyond the technical aspects.

Also, the more accelerograms recorded are, we find that the possible values may be higher than those from design codes and sometimes the codes provisions are taken from other design rules (with other local features).

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