# INFLUENCE OF IRREGULAR CONFIGURATIONS IN THE DAMAGE OF RC BUILDINGS IN 2010 CHILEAN (MAULE) EARTHQUAKE

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## *Abstract*

In the 27 February 2010, Chilean (Maule) Earthquake, there were many R.C. buildings severely damaged, specially in structural walls with irregularities. On the base of data of 83 high rise buildings inspected after the earthquake, geometrical and structured indexes were devised to show that (a) almost all the building that had severe damage had a set back in the first floor where they had a less robust wall system than the iconic Tajamar Building, however many building in similar situation behaved well; (b) the Chilean buildings have large reserve of shear capacity; and (c) irregularities in walls are not properly designed.

*Keywords: Structural walls, irregularities, strut and tie models, 2010 Maule earthquake, reinforcement detailing.*

## 1. Introduction

On 27 February 2010 at 03:34 hours occurred an earthquake of moment magnitude 8.8 that lasted 120 seconds; the epicenter was located approximately 105 kilometers north of Concepción, affecting the south-central Chile.

The characteristics of the earthquake is described elsewhere [1,2,3]. Nevertheless, the intensity can be described by a simple statistics of the Department of Architecture of the Ministry of Public Works, in charge of designing all governmental building along the country: there were 1202 governmental buildings damaged in the zone affected by the earthquake: 43% were in the Maule Region (Talca city), 30% in Santiago Region, 15% in Bio-Bio Region (Concepcion city), 10% in O'Higgins Region (Rancagua city) and 2% in Valparaiso Region[4] (Chile is geographically organized in 15 regions). The population of Santiago is 6.7 millions, O'Higgins 873,000, Maule 960,000, Bio-Bio 2 million, and Valparaiso 1.7 Millions. (The 2010 earthquake is now named Maule 2010 earthquake) This statistics indicates that Concepción was not the higher intensity zone, nevertheless many relatively new building had to be demolished there.

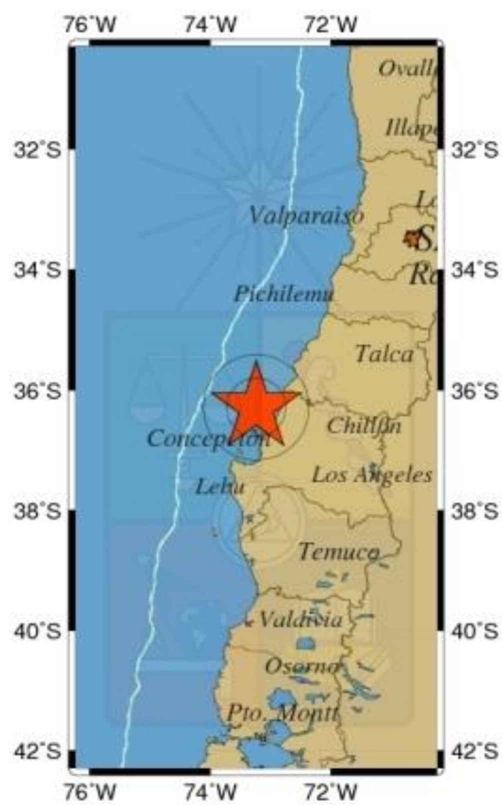



Fig. 1 – Maule 2010 earthquake epicenter[1].

We inspected more than 150 high rise building after the earthquake in Santiago and Concepción, and got information of 83 buildings. The height of the buildings was in the range of 10 – 28 stories, as shown in Fig. 2. In this figure the buildings of Santiago are numbered from 1 to 59, and of Concepción from 60 to 83. The periods of the buildings are shown in Fig. 3.

Santiago is in seismic zone 2, and Concepción in zone 3. The soil of Santiago is type II, and the soil of Concepción in some areas is type II, and type III in others, as indicated in Fig. 2. In Table 1 and Table 2 the seismic zones and soil types are shown according to the Chilean standard NCh 433[5].

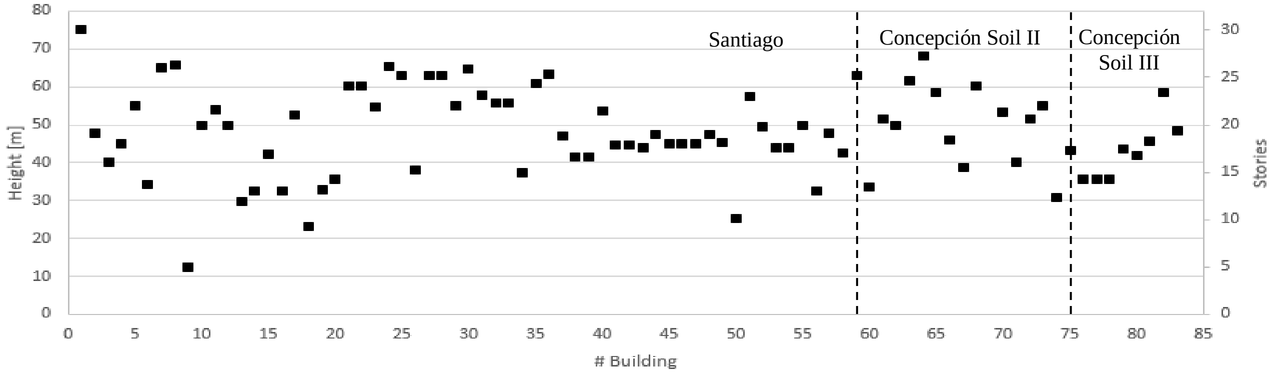
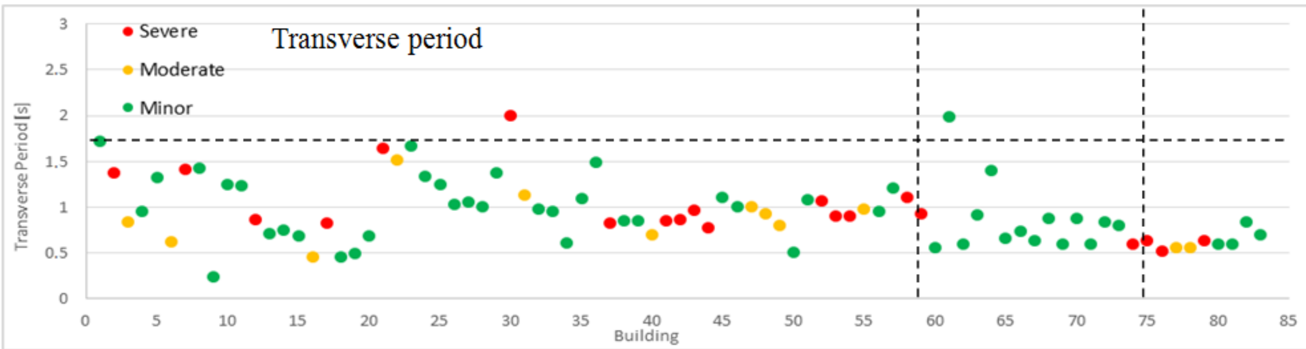
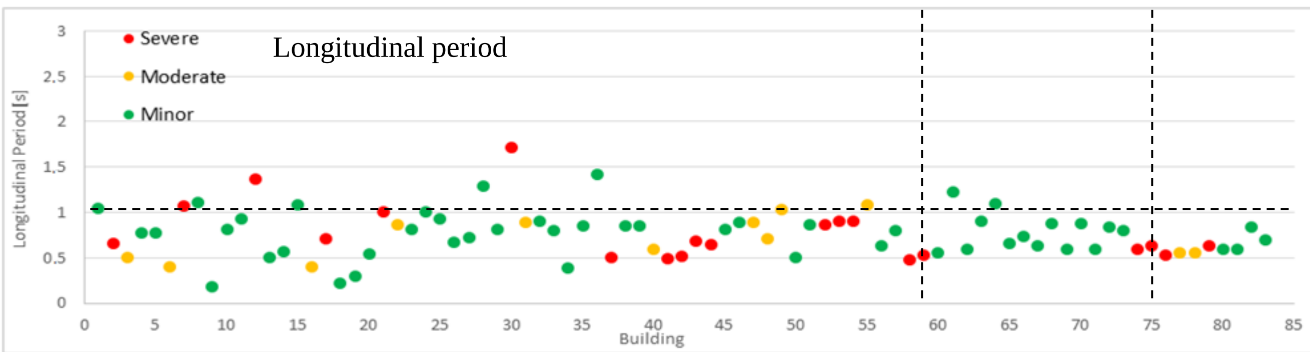



Fig. 2 – Height of buildings studied.




(a)




(b)

Fig. 3 – Transverse and longitudinal periods of buildings and grade of damage.

Table 1 Seismic zone definition, NCh433.

| Seismic Zone | Geographic Area | $A_0^{(+)}$ |
|---|---|---|
| Zone 1 | Andes zone | 0.20 g |
| Zone 2 | Central zone | 0.30 g |
| Zone 3 | Costal zone | 0.40 g |

$^{(+)}$ Maximum effective acceleration

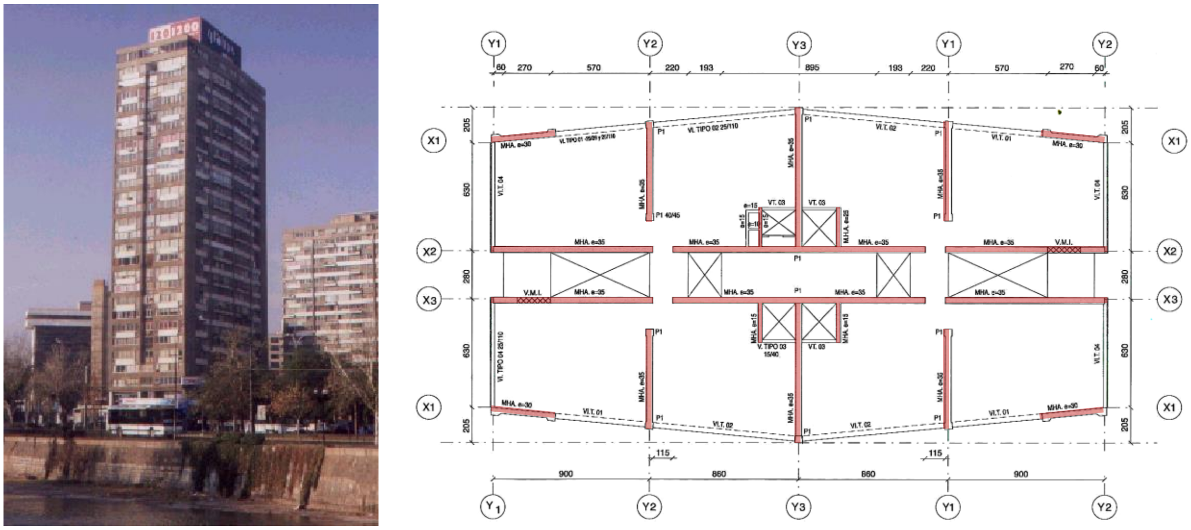
Table 2 Soil type descriptions, NCh433.

| Soil Type | Description |
|---|---|
| I | Rock |
| II | Dense gravel, and soil with Vs $\geq$ 400 m/s in upper 10 m. |
| III | Unsaturated Gravel and low compaction sand |
| IV | Saturated cohesive soil with $q_u$ < 0.050 Mpa |

Four buildings are highlighted: Tajamar Building[6] in Santiago (#1) Fig. 4; O'Higgins Building (#59) Fig. 5; Alto Río Building (#74); and Centro Mayor Building (#75). Alto Río Building collapsed, O'Higgins Building was severe damaged, the upper part was demolished and the lower part retrofitted. Centro Mayor had severe damage and was demolished.

Tajamar Building in Santiago, built in 1964, 28 stories high, is shown in Fig. 4. This building is an icon; before it was built, the tallest buildings in Santiago were not more 15 stories high. Since then the Tajamar Building has endured many strong earthquakes without any damage. The structure is very regular with two longitudinal walls and parallel transverse wall, making T walls, as indicated in Fig. 4(b). The thickness of the walls in the first floor was 450 mm. It has just one basement floor, but it was not a parking floor.

For these reasons Tajamar Building has been selected in this study as a standard, a frame of reference, against which the geometrical and structural properties of the 83 building studied will be compared.



(a) (b)

Fig. 4 – Tajamar Building.

(a) After 2010 earthquake (b) Partial demolition in 2013 (c) Current retrofitted building in 2016

Fig. 5 – O'Higgins Building.

Notably, all building that were severely damaged (a) had the same structure configuration as Tajamar Building, but with many basement parking floors; (b) failed in flexural compression mode, as indicated in Fig. 6. Shear failures were not observed in the field inspections; (c) most walls had vertical irregularities; (d) the thickness of walls at the base was of the order of 250 mm.

On the base of geometrical and mechanical data of 83 buildings, it will be shown that: (a) almost all the buildings that were severely damaged had set backs in the first floor, which create "flag" walls. However, many other buildings in the same situation behaved quite well. This evidence indicates the fact that buildings that failed should have had some deficiencies in design and detailing of reinforcement, apart from the brittle performance shown by the "T" walls; (b) Chilean buildings have large reserve of shear capacity; (c) geometrical and mechanical irregularities should be carefully designed applying strut and tie models; and (d) structural software should be used very carefully in walls with irregularities.


Fig. 6 – Damage pattern observed in RC "T" walls.

## 2. Indexes

For the purpose of this study several geometrical and structural indexes will be used

- $d_x$: area of walls in the x direction divided by total floor area.
- $d_y$: area of walls in the y direction divided by total floor area.
- wall density = $d_i = d_{xi} + d_{yi}$ for floor i
- wall density ratio for the first floor = $d_1/d_2$
- wall density ratio for the first basement floor = $d_{-1}/d_1$
- base shear [$N/mm^2$] = the design base shear of the building, according to the Chilean standard NCh 433, divided by the area of the walls in the x or y direction.

The base shear was estimated as a reference according to the static approach of the Chilean standard NCh433, with the following equations:

$$Q_0 = C \cdot I \cdot P, \tag{1}$$

where $C$ is the seismic coefficient; $I$ is the coefficient associated to the importance of the building; and $P$ is the total weight of the building.

$$C = \frac{2{,}75 \cdot A_0}{g \cdot R} \cdot \left(\frac{T'}{T^*}\right)^n, \tag{2}$$

where $A_0$ is the maximum effective acceleration associated with the seismic zone; R is the response-modification factor; n and T' are parameters associated to the soil type; and T* is the period of the structure in the direction of analysis.

The seismic coefficient $C$ shall be between the following limits, according to the Chilean standard NCh433, where S is a parameter associated to the soil types, between 0.90 and 1.30:

$$C_{min} = \frac{A_o}{6 \cdot g}, \qquad C_{max} = \frac{0{,}35 \cdot S \cdot A_o}{g} \qquad (3),(4)$$

## 3. Wall density analysis

In Fig. 7 it is shown the [wall density/stories] index of the 83 buildings for the typical floor, first floor and the first basement floor. The figures show:

a) Almost all the buildings in Santiago with severe damage had in the first basement floor more wall density than Tajamar Building, but had less wall density in the first floor, where they typically failed, as shown in Fig. 7.

b) Many buildings inspected in Concepción behaved quite well, having more or less wall density than the standard Tajamar Building. The buildings that failed had large irregularities or design deficiencies.

c) O'Higgins Building (#59), Fig. 5, had less wall density than Tajamar Building in the first floor and the basement; however it failed in upper stories due to large irregularities.

d) Alto Rio Building (#74) had more wall density in the first floor and the first basement floor than the Tajamar Building, but collapsed.

e) Centro Mayor Building (#75) had less wall density in the first floor and first basement floor than the Tajamar Building, and mechanical irregularities (see Section 6). It was demolished.


Fig. 7 – Wall density / Stories index.

In Fig. 8 the wall density ratios in the lower stories are plotted against the index [wall density/stories]. It indicates that almost all the building of this sample that had severe damage had less wall density than the standard Tajamar Building, and had a set back in the first floor.

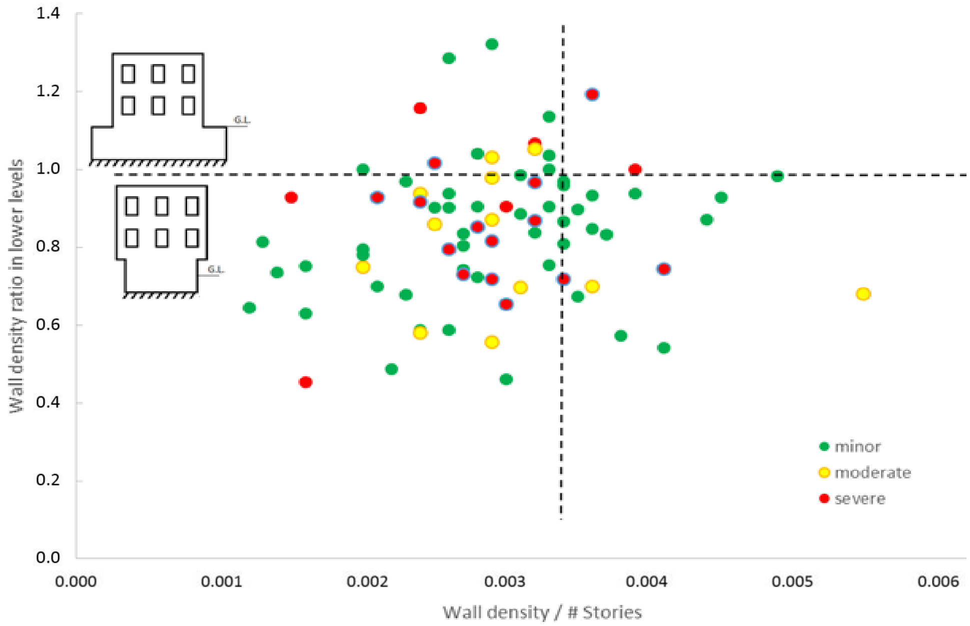



Fig. 8 – Effect of density of walls and wall density changes between lower floors in building behavior.

## 4. Base shear analysis

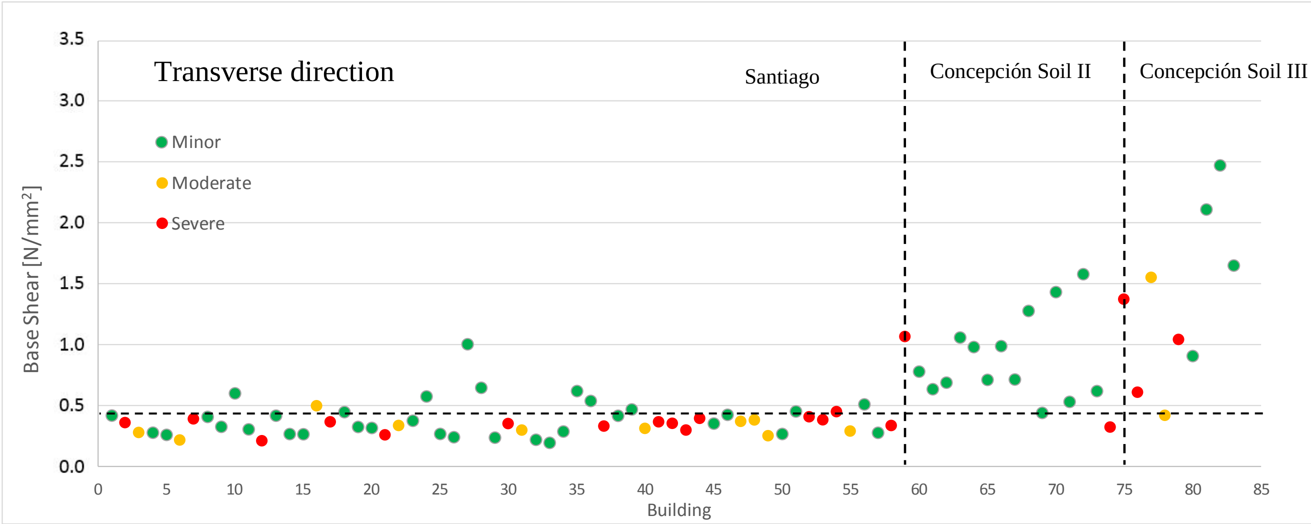
Fig. 9 shows that all building with severe damage in Santiago had a mean base shear (based on the design base shear of the Chilean standard NCh 433) less than 0,5 [N/mm$^2$]. As a reference, the concrete contribution to shear strength is

$$v_c = \frac{V_c}{b_w \cdot d} = \frac{\sqrt{f'c}}{6} \tag{5}$$

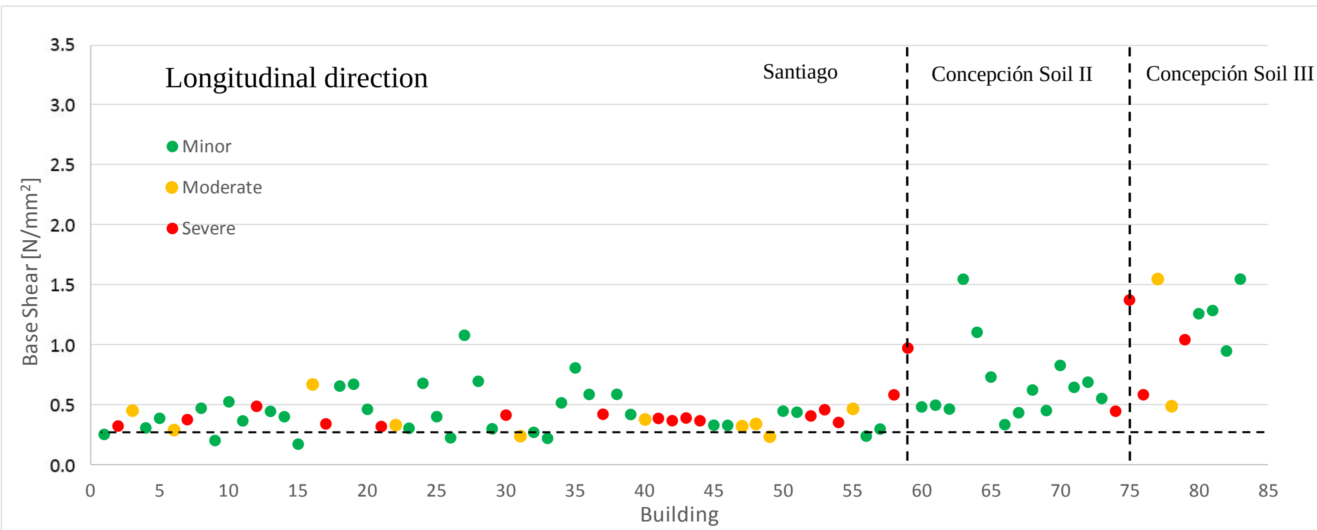
and if we assumed f'c = 25 MPa, the most common concrete grade used, then $v_c$=√(25)/6 =0.83 MPa.

Now, in Concepción, O'Higgins Building (#59) had the value of the index of the base shear in both directions of the order of 1.0 MPa. Alto Rio Building (#74) had 0.5 MPa in the longitudinal direction, and 0.35 in the transverse direction. Centro Mayor Building (#75) had the largest index, of the order of 1.4 MPa. In this building $v_s$ ($V_s/(b_w \cdot d)$) was estimated as 1.4 $v_c$.

These values indicate that the Chilean buildings have large reserve of shear capacity, and confirm what occurred in the field, where no shear failure was observed.




(a)




(b)

Fig. 9 – Base Shear index (The horizontal dotted line corresponds to the Tajamar Building (#1)).

## 4. Vertical irregularities

A collection of typical irregularities in walls with their damage due to 2010 Maule Earthquake is exhibited in Fig. 10. In each case the ground level (GL) is indicated. The reinforcement detailing of these walls was very poor. These figures show the inability of many Chilean architects to deal with parking basement floor. It also indicates the inability of many structural engineers to fix these deficiencies. It seems that they are not designing the structure properly but just determining the amount of reinforcement for the structure actually designed by the architect. If these irregularities in the walls are not solved during the architectural design process, it is difficult for the engineer to fix them afterwards.

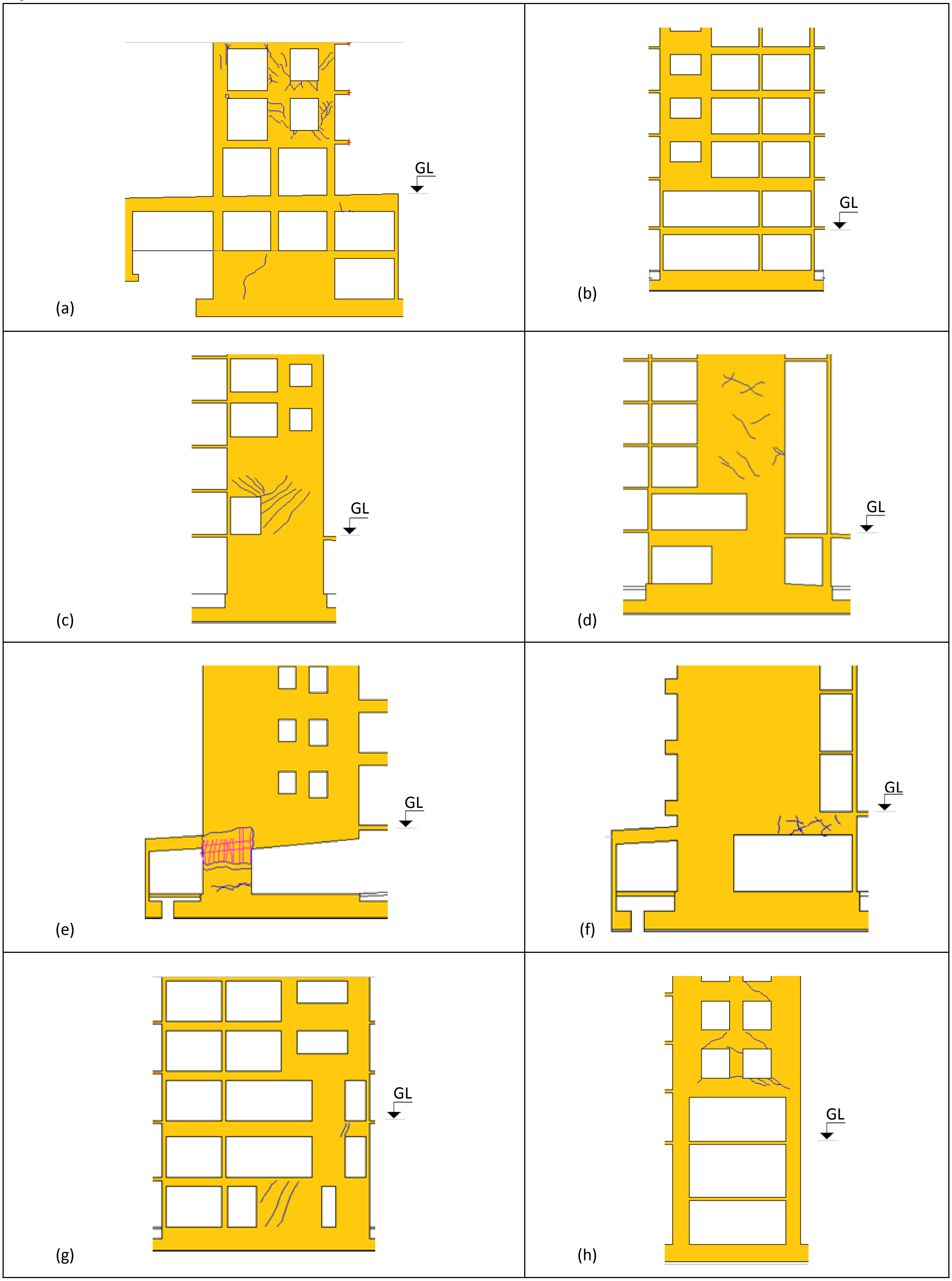


Fig. 10 – Selection of vertical irregularities in buildings that were damaged during the Maule 2010 earthquake.

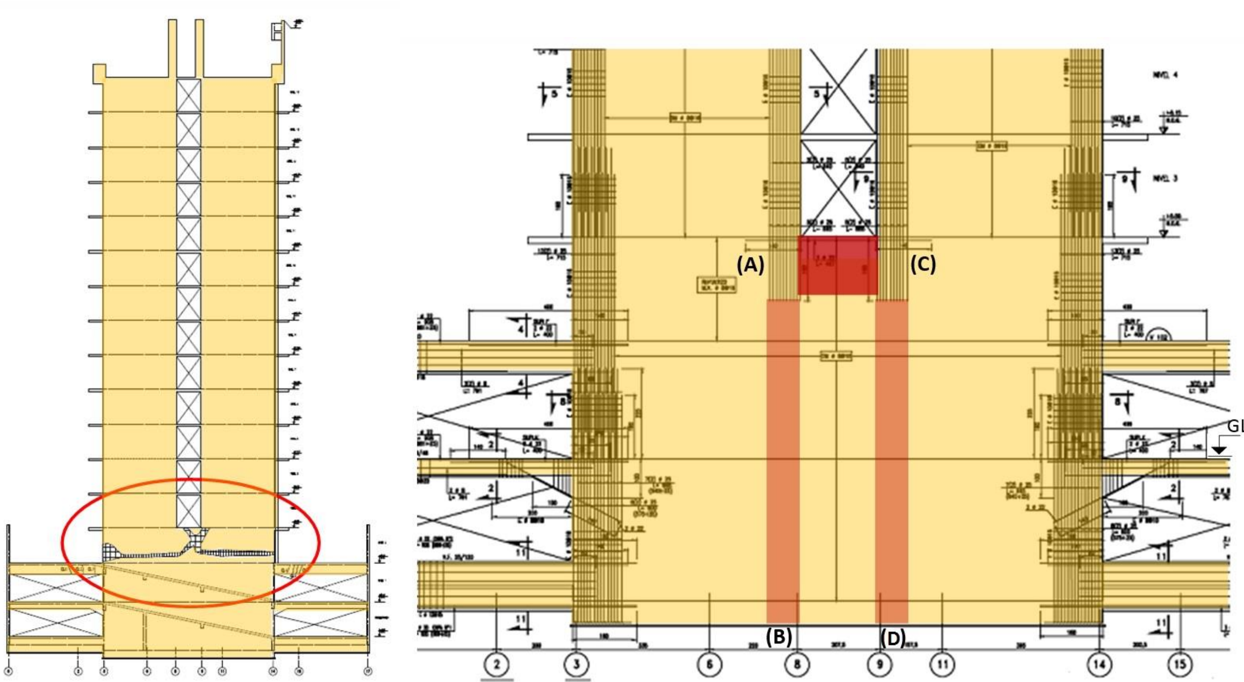
## 6. Detailing of reinforcement for irregularities

### 6.1 Final points of resistance

Most irregularities, geometrical or mechanical, are not designed properly. As an example, Fig. 11 shows a wall of Centro Mayor Building (#75) in Concepción. The larger lower wall failed at the end of the central vertical reinforcement points A and C. The failure zone is shown in Fig. 12. The central vertical main reinforcement had enough development length into the larger wall; however this design does not comply with the requirement 4.4.4 of ACI 318-14[7].

"The structural system shall be designed to resist the factored load combinations given in 4.3 without exceeding the appropriate member design strengths, considering one or more continuous load paths from the point of load application or origination to the **final point of resistance**."

The only "final points of resistance" for this central vertical reinforcement are points B and D in the foundation mat, Fig. 11.




(a) (b)

Fig. 11 – Failure of wall with mechanical irregularities.

(a) (b)

Fig. 12 – Pictures of the failure zone AC of the building of Fig. 11.

On the other hand, Fig. 12 shows that in the panel AC there is damage (disintegrated concrete) due to local shear force; however the horizontal reinforcement in the panel ($\phi$8 at 160 mm.) did not yield. This shear reinforcement was about 1.4 times $V_c$, the contribution of concrete to shear strength. This value indicates that the shear demand was not high; however the concrete in the panel disintegrated. To avoid disintegration, it is suggested to confine this panel with transverse ties at each intersection of the panel vertical and horizontal bars. The confinement zone should be of the same horizontal dimension of the opening and extend downwards 1.5 m but not larger than half the story high.

### 6.2 "Flag" walls

Another typical geometrical irregularity is shown in Fig. 13, corresponding to the design of a building before the peer review. Apart from the vertical main reinforcements, there is a light steel mesh in both faces. But there is not horizontal reinforcement in AB, CD, EF and GH, as a strut and tie model (Fig. 15) would suggest. If the strut are designed at 45º, then an horizontal reinforcement should be designed with the same amount of the vertical main reinforcement. An example of node design for node F in Fig. 13 is shown in Fig. 14 with bars with hair-pin shape for the vertical and horizontal $\phi$12 mm diameter bars. Finally the vertical main bar at E (Fig. 13) should be extended up to point I to develop 45º struts GI.

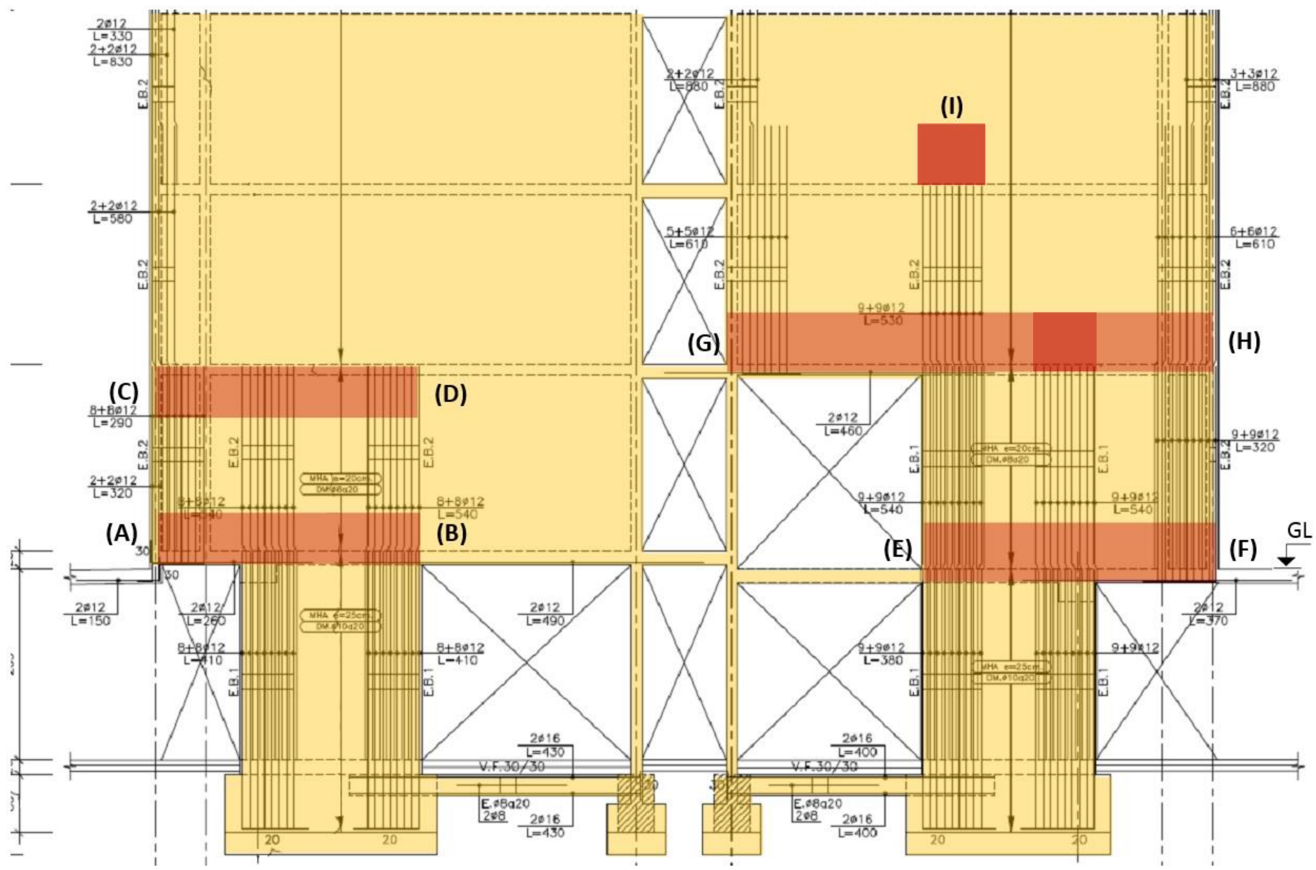



Fig. 13 – Geometrical irregularities in a wall with a set back in the first floor.

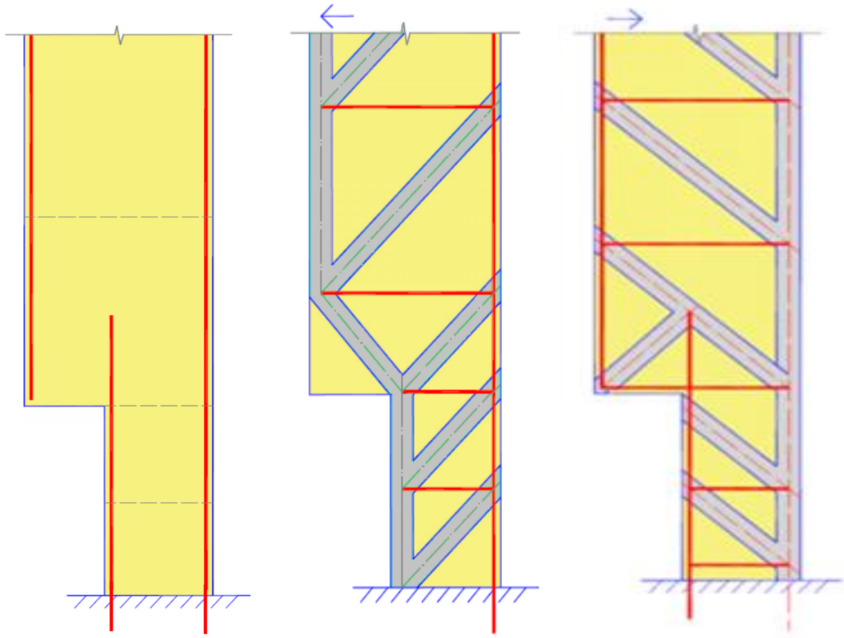


Fig. 15 – Strut and tie models.

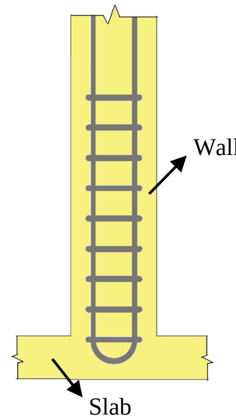
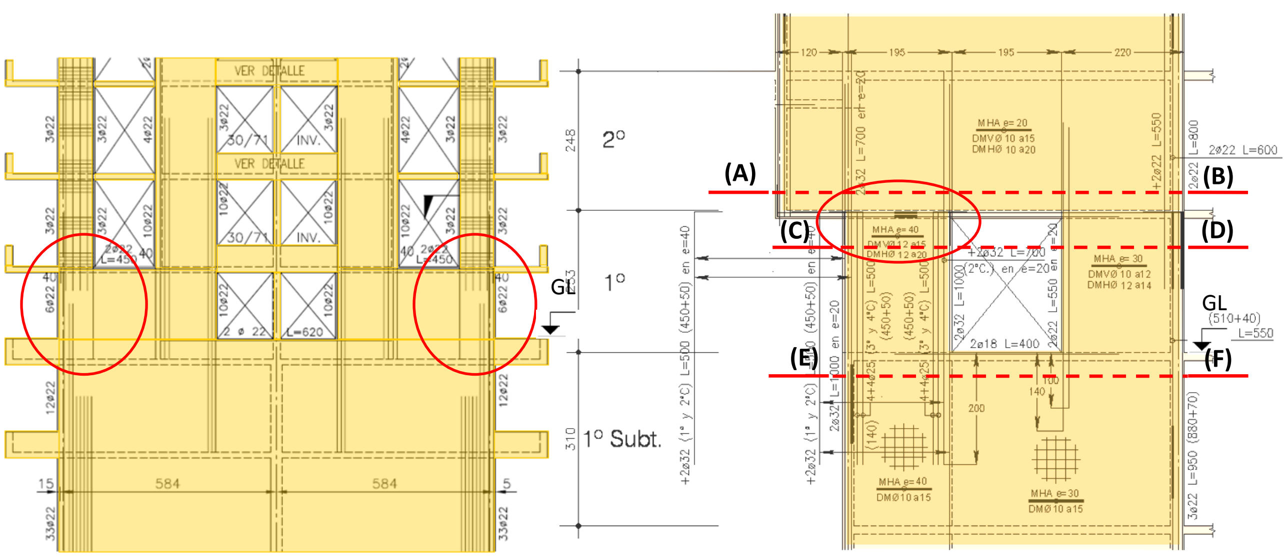



Fig. 14 - Example of node design for point F of Fig. 13.

## 7. Structural software

The design shown in Fig. 16, corresponding to drawings before the revision by the peer reviewer, demonstrates how some structural engineers use structural software that does not properly deal with irregularities. Fig. 16(a) shows discontinuities of the vertical main reinforcement. This is not understandable unless it is the outcome of some sort of automatic design software. This design obviously does not comply with ACI 318-14, 4.4.4 requirement.

Another typical example is shown in Fig. 16(b). In this case some vertical main bars at level CD are not properly developed upward. At level EF 4+4 $\phi$25 mm bars (i.e. 8 bars) in each side of the left wall do not exhibit a clear "final point of resistance". Finally at both sides of levels CD and EF there is not the symmetry of reinforcement one would expect.




(a) (b)

Fig. 16 – Example of curious detailing.

## 8. Conclusions

(1). Almost all the buildings in Santiago with severe damage had in the first basement floor more wall density than Tajamar Building, but had less wall density in the first floor, where they typically failed in flexural compression mode.

(2). Many buildings in Concepción behaved quite well, having more or less wall density than the standard Tajamar Building. The buildings that failed had large irregularities or design deficiencies. Concepción was not the higher intensity zone.

(3). Almost all the building inspected that had severe damage had less wall density than the standard Tajamar Building, and had a set back in the first floor.

(4). The Chilean buildings have large reserve of shear capacity; in the field no shear failure was observed.

(5). Many Chilean architect revealed inability to design parking basement floor without irregularities.

(6). Most irregularities, geometrical or mechanical, are not designed properly, in particular many reinforcements does not comply with the requirement 4.4.4 of ACI 318-14. Strut and tie models should be used to design irregularities where current structural software do not properly deal with them.

## 8. Acknowledgements

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## 9. References

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