Italy Tower Collapse in São José do Rio Preto–S. Paulo, Brazil

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Abstract: This paper discusses the collapse of Italy Tower, a 16-storey building, that occurred 10 years after construction, with 10 floors tumbling upside down into a side street. The designed structure and the structure as built were verified and a possible and probable cause was identified.

Key words: High-rise buildings, columns, transition, piles, collapse, brittle failure.

1. Introduction

The collapse occurred day 16th/Oct/97.

CREA-SP – SP Regional Engineering Council - in July/98, hired a technical report to undertake this collapse. The scope was the analysis of the structure as designed and as built based on data from CREA-SP Process and the IPT Technical Report – Technological Research Institute, concluding with an opinion on the probable causes of the collapse.

Some witnesses described the collapse as follows: the tower tilted and began slowly to settle, less in front façade than in back, up to an inclination of approximately 1:1. Suddenly the structure collapsed.

Photographs of the ruins show that the piles and the façade columns failed in tension. The upper floors tumbled and fell into side street upside down.

2. Italy Tower Description

Italy Tower had 16 floors and an attic built above a common floor (PUC) together with two other towers Portugal and Spain.

Below common floor are two parking floors and the ground floor for commercial use.

The typical floor-to-ceiling height is 3.15 m, growing in lower floors up to 5.2 m.

The structure of the typical floor features a skew angle of approximately 38° in relation to the side street Luiz de Camões. This rotation made the structure quite complex and Luiz de Camões façade zigzag (see Fig. 1 for shape of typical floor).

It also required a transition at the common floor, part parallel to Av. Bady Bassit and part parallel to side street Luiz de Camões (see Fig. 2 for the common floor layout).

It should be added that this is not a conventional structure due both to the transition and complexity of the typical floor. This complexity led to the creation of unfilled frames and to the increasing responsibility of the two elevator towers.

Below the ground floor are the foundations (no basements) built up of caps on Franki piles ø520 mm. These piles were around 10 m long. See Fig. 3 indicating the two elevator towers P62 and P70.

The subsoil profile consists of a thick layer of loose clayey sand, based on a compact sand layer on top of the sandstone. The piles were practically supported on sandstone, soon after crossing 1.5m of compact sand. The water level is 3 m deep.

3. Collapse Description

From the data gathered about the collapse, the following aspects are especially important:
Fig. 1  Typical floor (dimension in m).

Fig. 2  PUC floor.

Fig. 3  Foundation structure.
(a) Before the collapse, the building showed no signs of anomalies, such as cracks or differential settlements.
(b) The collapse began with a bang similar to a transformer overflow at 2:00 a.m.
(c) The glass and the façade frame broke and twisted due to significant differential settlements, for a few hours (2:00-6:00 a.m.).
(d) At around 5:30 a.m. Mr. Valter, the building superintendent, went upstairs just beside P62 to warn families who were still in the building. As he climbed he noticed that the plaster of walls and ceilings was crumbling.
(e) In Italy Tower’s collapse, it would have rotated slightly around its vertical axis, opening the connection with the Spain Tower at the back and soon caving in slowly as a whole much more to the back than the front. At a certain moment the settlement stopped but the building continued to rotate up to an inclination of approximately 1:1. Suddenly and quickly the structure collapsed. See Fig. 4.
(f) Observing the pictures of the debris three (3) further important aspects are noticeable:
   • The piles of the columns of the Av. Bady Bassitt façade show tensile failure of the rebars.
   • These same columns show tensile failure in splices at the PUC level. See Fig. 5.
   • The upper floors, when falling into the side street Luis de Camões, were placed in inverse order to the normal, i.e., the 9th floor is over the 10th floor and so on, all upside down. This fact is observed from the 6th floor to the roof. See Fig. 6.

4. Seeking Diagnosis

In these circumstances, eight months after the collapse, it is harder to determine its cause. Data collected by the IPT were of valuable help.

Structural defects, or even other, in a properly reinforced structure should generate a more ductile collapse, with more warning than that observed. As it was observed a brittle failure, it is more appropriate to seek the cause in very rigid but at the same time very brittle elements.

Looking for the truth we need to consider the largest number of collapse hypotheses.

The creation of these hypotheses, regardless of their being proven or not, must come from the analysis of the available data and results of a calculation model.

The available data, especially those gathered in the IPT report, pointed to considerable deviations of construction, emphasizing the following:
overlay thickness for slabs and walls far greater than those provided for in the design
- deviation of the columns in relation to the transition beam
- columns with reduced dimensions (P64 - 65 x 80 instead of 80 x 80 as designed)

These deviations were at least partially compensated by a concrete better than specified. Concrete $f_{ck}$ 25 MPa for the columns and 20 MPa for the beams, instead $f_{ck}$ 18 MPa.

5. Calculation Model

As can be seen from the calculations, the model used in the tower design includes, for wind study, a set of 2D frames chosen by the designer, fixed to the foundation.

Since the building has a complex structure, function of architecture, and an important transition at the PUC level, it is convenient to use a 3D frame, including the foundations in order to examine further the ground-structure interaction.

In order not to deviate too far from the usual practice we decided to adopt a linear behavior for the materials, that is, admitting Hooke's law and small displacements.

To evaluate second-order effects, we decided to use the coefficient $\gamma_z$, as proposed by NBR6118/2003 -Design Structural Concrete, not yet published in 1998.

So we opted for the TQS, Brazilian linear elastic software that automatically models, calculates, scales and details the structure.

If the physical nonlinearity of an element needs to be considered, it can be done approximately through equivalent secant stiffness.

Moreover, some other problems must be remembered and overcome:

The first of these concerns is the stiffness of the columns for normal forces. Since columns undergo different stresses, we need to correct their stiffness in order to avoid significant unreal differential settlements. In fact, during the construction process, floor by floor, these differences are neutralized significantly.

The second concern is about the beam-column connections. As the elevator columns has relatively thin walls, a rigid node connection was provided to avoid loss of stiffness.

6. Situations to Be Verified

The purpose of this item is to discuss in general terms the verifications to be done and the sequence in which they must be performed.

6.1 Designed Structure

Considering only the data contained in the design drawings for foundations, structures and architecture.

6.2 Structure as Built

Considering the design data and only the construction deviation explicitly identified in the rubble.

As aforementioned, considerable construction deviations have been observed specifically in the Italy Tower. See item 4.

6.3 Portrait of collapse

We tried to portray a collapse from plausible assumptions consistent with the underlying data and with previous verifications 6.1 and 6.2.

The designed structure (6.1) did not indicate potentially critical points.

Verifying the structure as built (6.2) showed that it was still acceptably safe, but there were at least two potentially critical points that could be the source of the collapse: the transition beam and the foundation.

When looking for brittle elements, we found the piles with less reinforcement than the minimum for columns, and they may present a structural brittle failure that occurs without warning, as in the collapse in question (ultimate structural resistance is far smaller than the geotechnical resistance).

Trying to identify cases that could create a brittle failure, four hypotheses of collapse were proposed:
- Eccentricities on startup of the columns from the transition beam at the PUC level.
- Defect in a pile.
- Geometric construction errors when beginning the driving operation.
- Piles with different stiffness in the same cap (load tests in short Franki piles on rock showed stiffnes changing from 1 to 3.5).

7. Safety Criteria

7.1 Foundation Safety

We checked the piles by using working loads, as usual.

So, for a design with only vertical loads, the load limit for the average in one cap was 1500kN per pile.

When the vertical loads are combined with the wind, according to the standard - NBR 6122/86- Foundation Design and Execution, this value was increased by 30%.

So, the maximum load per pile was \( 1500 \times 1.3 = 1950 \text{kN} \) when wind forces are considered.

The ultimate structural capacity varies from 2400kN for concrete of \( f_{ck} = 14 \text{ MPa} \) (minimum obtained from boring of specimens by IPT) to 2800kN for \( f_{ck} = 17 \text{ MPa} \).

7.2 Structural Safety

When checking the structure, the safety criteria are based on the Method of Limit States according to the standard - NBR 6118/78- Design and Execution of Reinforced Concrete Works - revision underway at the time (1998).

7.2.1 Designed Structure

The partial safety factors in Brazilian codes were at 1998 the following:

\[
S_d = \gamma_{fg1} S_{g1} + \gamma_{fg2} S_{g2} + \gamma_{fq}(S_q + 0, 8_0)
\]

with \( \gamma_{fg1} = \gamma_{fg2} = \gamma_{fq} = 1.4 \)

Resistances were: concrete \( f_{ck} = 18 \text{ MPa}, \gamma_c = 1.4 \) and steel \( f_y = 500 \text{MPa}, \gamma_s = 1.15 \).

7.2.2 Structure as Built

The structure as built was verified for the same combination of actions, by changing some values depending on the knowledge generated by IPT surveys. This additional knowledge allowed us to reduce the partial safety factors:

The new safety factors were:

\( \gamma_{fg1} = 1.3, \gamma_{fg2} = 1.25, \gamma_c = 1.26 \).

7.2.3 Collapse Hypotheses

When verifying each of the collapse hypotheses the following actions were considered:

(a) vertical loads for the structure as built, that is, according to the IPT surveys.

(b) wind loads, equal to 30% of standard loads, since on the day of the accident the wind was moderate.

(c) A collapse hypothesis

To check that a structural element had not contributed to the collapse or failure, it should show, for the same combination of actions previously set, plus one collapse hypothesis, the following safety factors:

\( \gamma_{fg1} = \gamma_{fg2} = \gamma_{fq} = 1.1; \gamma_c = 1.1 \) and \( \gamma_s = 1.0 \)

8. Checking Results

8.1 Checking foundations for 6.1 and 6.2

Table 1 shows the checking foundations results for the Structure as Designed (6.1) and Structure as Built (6.2).

<table>
<thead>
<tr>
<th>Case</th>
<th>P61/P62</th>
<th>P70</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.1</td>
<td>Without wind mean value</td>
<td>1370 kN</td>
<td>1430 kN</td>
</tr>
<tr>
<td></td>
<td>With wind max value</td>
<td>1850 kN</td>
<td>1670 kN</td>
</tr>
<tr>
<td>6.2</td>
<td>Column 61/62</td>
<td>Column 70</td>
<td>Limits</td>
</tr>
<tr>
<td>6.1</td>
<td>Without wind mean value</td>
<td>1600 kN</td>
<td>1760 kN</td>
</tr>
<tr>
<td>6.2</td>
<td>With wind max value</td>
<td>2110 kN</td>
<td>2000 kN</td>
</tr>
</tbody>
</table>

No problem was identified for the Structure as Designed (6.1) but for the Structure as Built (6.2), the piles had not the required resistance and need strengthening.
8.2 Checking Structures for 6.1 and 6.2

\[ \gamma_z = \frac{1}{1 - \frac{\Delta M_{\text{tot},d}}{M_{1,\text{tot},d}}} \]

\( \Delta M_{\text{tot},d} \)

\( M_{1,\text{tot},d} \)

is the second order effect

is the first order moment

Checking the structure as designed (6.1) the stability parameter \( \gamma_z \) reaches a maximum of 1.13, which indicates the need to consider the overall second-order effect, which has been done. \( \gamma_z \) is defined by the equation below (NBR6118/2003):

The reinforcement of columns and beams determined from all calculated forces resulted in a deficiency in some columns and beams with wind. For vertical actions only, no deficiency has been found.

In order to improve the significance of the results from the point of view of non-linearities, the elastic stiffness of some elements was replaced by its secant value.

These corrections changed the distribution of forces, increased the displacements and the stability parameter \( \gamma_z \) reaches an acceptable 1.24.

The deficiencies were not major and since the sum of resistances in each level was greater than the sum of forces we decided that plastically the designed structure was OK.

Checking the structure as built (6.2) as defined previously, despite of measured load increase, no problem was found in the structure itself, even for column P64 with reduced dimensions of 65 x 80, instead 80 x 80 cm as designed. This result is due to the measured increase of concrete resistance from specimens taken by IPT - \( f_{ck} \) 25 MPa for columns and \( f_{ck} \) 20 MPa for beams in place of the design value (18 MPa).

8.2 Collapse Hypotheses

The collapse hypotheses studied were defined in 6.3.

8.2.1 Eccentricity of columns relatively to the transition beam, pile defect, pile positioning errors

These hypotheses do not identify a brittle failure able to explain the collapse.

Piles always present good redistribution capacity and the transition beam, despite of loss of safety were far from collapse and were also ductile.

8.2.2 Piles with different stiffness supporting the same cap

As aforementioned, load tests of \( \Phi 520 \) short Franki piles (6m-10m long) with their tip in rock, show that their stiffness could change considerably, between 1 and 3.5.

Within these hypotheses and variability, at least six cases were verified, all related to the cap P61/62, in an attempt to find a possible failure sequence that could have started the collapse. See Fig. 3.

The results show in firstly a great sensitivity to the variation of stiffness.

Secondly, they show that it is possible to obtain pile loads overpassing the limit established between 2400 and 2800kN.

The maximum pile load found was 3570kN, applied to the pile E108.

These results also show that the failure of one of these very stiff piles could trigger the collapse, with a pile failure sequence.

It is important to add that, in each of these cases the columns were verified just above the cap and had enough strength to impose these additional loads to the piles.

9. Conclusions

The findings lead to the conclusion that:

The design of both structure and foundation is primarily suitable for the time it was developed. The differences detected are acceptable and certainly have nothing to do with the collapse of the building.
The execution of the work introduced, according to the IPT survey, additional loads and geometric imperfections that should not be implemented without the designer’s verification.

These increased loads on piles reduce the safety required and foundation strengthening would be necessary.

The piles should be loaded at least till 2400 kN to justify the collapse, which was only reached through the hypothesis that some piles were stiffer than the nearby supporting exceptional loads (up to 3570kN in the cases studied) and displaying brittle fracture. This was the only feasible and quite possible hypothesis found.

To make this hypothesis possible, the decisive factor was the application of the conventional criterion of uniformly stiff piles to a case of short piles with their tip in rock where the geotechnical load capacity is much higher than the structural strength.

We suggested the technical community open a revision process of our foundations standard, discussing this problem and proposing specific criteria for such cases, what was accepted.

References
[1] CREA-SP Process of Italy Tower - SF/97 40255 from 12/30/97
[3] NBR 6122/86 - Foundation Design and Execution
[4] NBR 6118/78 - Design and Execution of Reinforced Concrete Works