Tall buildings seismic behavior comparative study by increasing the concrete mechanical strength through non-linear static analysis and seismic performance

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Abstract. The use of high strength concrete in tall buildings implies less lateral displacement, however, an important question is: In what other measures the seismic behavior of a building improves with high strength concrete (550 kg/cm²). As a result, the material allows the construction of slender structural elements, however we won’t be able to know how much the ductility of the structure in front of a seismic event will improve. Therefore, the reason of this study is to analyze the dynamic response of a tall building with different sections and resistance to the compression of its structural elements. The aim of this study is to obtain the curve of capacity and the point of seismic performance of each structural model.

1. Introduction

During the last decade, earthquakes have caused economic loss as well as in terms of social costs. Since the year 2000, more than 1,000,000 people have died due to earthquakes, for which the associated cost was 445,700,000 million dollars [1]. Evidence of this was the Tangshan earthquake (China) that occurred in 1975 that killed 200,000 people. Also, the 1994 earthquake in Northridge (USA) left 75 dead and approximately 8,700 wounded. In turn, it had a loss of about 42 billion dollars. Likewise, the 1995 earthquake in Kobe (Japan) caused about 6,000 deaths and more than 120 billion in economic losses. On the other hand, in August 1996, the earthquake that occurred Izmet (Turkey) killed 20,000 people and caused economic losses of 12 billion. Finally, the Chi-chi (Taiwan) earthquake in 1999 caused an estimate of 8 billion loss, as did the Gujarat (India) earthquake in 2006 where there were around 18,000 dead and 330,000 demolished buildings [2].

High-strength concrete (HPC) has allowed the continuous development of high-rise buildings, with the purpose of using floor areas more efficiently and obtaining a better lateral response [3]. However, unknowns arise about the seismic behavior of the structure with the use of high strength concrete; particularly, in the plastic hinge mechanism, the structure ductility and its seismic performance [4]. It is also an advanced cement material with excellent durability, which offers a potential to become a practical solution to improve the sustainability of buildings and other infrastructure [5].

That’s why, it is evidenced that with this concrete there are advantages such as the inertia moment decreases in the structural elements and as it had been mentioned a better durability, since it increases the structure life period in comparison with the normal resistance concrete. However, for most countries, demonstration projects that used ultra-high strength concrete (UHPC) didn’t generate acceptance. This is due to the lack of design codes, limited knowledge of both material, production technology and high costs, limiting the implementation of this material [6].
To address the problem of the structure seismic behavior by increasing mechanical resistance in Latin America, a non-linear static Push-over analysis is proposed for five models over 18 levels, increasing the strength of concrete in structural elements (plates, columns and beams) in order to obtain the capacity curve and the seismic performance point of the structure. For this, Model A has a resistance of 210 kg/cm², Model B of 350 kg/cm², Model C of 550 kg/cm², Model D of 210-350 kg/cm² and finally Model E of 350-550 kg/cm².

In the buildings analyzed, the resulting structure implies that for a 60-meter building with compressive strengths of the E model (350-550 kg/cm²) was obtained a basal shear of 1349.80 tons, a maximum displacement of 48.75 cm and a maximum ductility of R = 6.

2. Method

This research evaluates the non-linear response of structural models taking into account the variation of the mechanical strength of concrete, with the aim of obtaining structures with greater resistance capacity, a reduction of the own weight and the decrease of its cost in the face of a lower volume of flexo-compression and cut, so it will have to be verified with a non-linear analysis and seismic performance.

The displacement that is generated in the upper level and the performance point of the building is done before a pattern of incremental lateral loads and constant gravitational loads; therefore, ductility, the mechanism of formation of plastic kneecaps and damage to different levels of seismic demand will be evaluated. The latter implies the progressive degradation of stiffness and the collapse of the structure.

The five models with different compressive strengths (210, 350, 550 kg/cm² and two combinations of 210-350 and 350-550 kg/cm²) are analyzed by computer. They are defined as structural elements (beams, columns and plates) for which is proposed sections of (30x60) cm in main beams, (30x40) cm in secondary beams, (70x70) cm in central columns, (40x100) cm in non-central columns and plates with 50cm of thickness, all with a high-strength concrete of 550 kg/cm².

On the other hand, the structures are considered to be embedded in the base, column nodes and walls. It should be noted that the slab was modeled as a “membrane” type element so that the loads are transmitted in their entirety and not lost due to the rigidity contribution.

For loads of the entire structure, national standards will be considered in the models. A spectral modal analysis is performed to evaluate these models, granting 100% of the dead load plus a percentage of live load and live roof load, depending on the type of building and its comparison with the maximum allowable drift of 0.007 [7].

That is why the use of high strength concrete in flexural elements exhibits greater rotational ductility, improves the capacity of the column, as well as increasing the stiffness of the “beam-column” joints [8]. For this investigation we want to analyze the seismic behavior with respect to the increase in concrete resistance. For this, in each structural model the sections of the structural elements are reduced, since the f’c is increased in order to know the improvement of the non-linear response to the concrete variation.

Subsequently, the design [9] of each element for each model was carried out in order to obtain a resistant earthquake structure and to be able to insert the plastic labels manually and automatically to the face of each structural element, since if you do not have the design of the entire building could not know the mechanism of failure of the structure.

Therefore, once the bending and cutting labels have been assigned to each element, the moment of inertia of the shear and bending moment is reduced by factors of 0.4 - 0.3 respectively for beams, 0.4 - 0.7 in columns and factors of 0.4-0.5 in walls structural causing the structure to lose stiffness due to the cracking factor introduced.

With this procedure, the capacity curve is obtained and the performance point is generated by the capacity spectrum method [10] where the capacity curve is converted into capacity spectrum to obtain pseudo-accelerations (Sa) and pseudo-displacements (Sd) for each point of the shear and its displacement as shown in (1), (2), (3) and (4):
Where:

\( v \): shear force

\( w \): structure weight

\( \alpha_1 \): modal mass coefficient for mode 1

\( PF_1 \): seismic participation factor mode 1

\( w_i/g \): assigned mass of level \( i \)

\( \phi_{i,1} \): amplitude of mode 1 at level \( i \)

\( \Delta_{tope} \): roof shift

\( N \): N level, higher structure

Likewise, the seismic demand spectrum (\( S_a vs T \)) is converted to an ADRS format (\( S_a vs S_d \)), as shown in (5) and (6):

\[
S_d = \frac{1}{4\pi^2} S_a T^2
\]  

(5)

\[
T = 2\pi \sqrt{\frac{S_d}{S_a}}
\]  

(6)

Where:

\( S_d \): pseudo-displacement

\( T \): period

\( S_a \): pseudo-acceleration

Finally, after obtaining the performance point, the scale factor is iterated according to its return period. Therefore, for the present investigation a scale factor of 1.5 corresponding to a \( Tr = 1500 \) years is used in order to find the point of transfer. Thence, the maximum ductility of the structure resulting from the division between the last displacement and the point of yield is obtained.

3. Results

To confirm our method of analysis we chose a seismic and important capital of Latin America, we analyzed five models of a 20-story building with a rigid core. Also, this research was analyzed using
the ETABS 2016 version 16.2.0 software for the structural analysis and resistant earthquake of the five models.

3.1. Models’s Structural Weight

Next, the results will be presented in Table 1, the weights found in the five models analyzed.

### Table 1. Five model's weight

| Models | Total Weight | $f'_c$ | Us |
|--------|--------------|-------|-----|
| A      | 30707.63     | 210   | tonf |
| B      | 29290.22     | 350   | tonf |
| C      | 28727.99     | 550   | tonf |
| D      | 29271.56     | 210-350 | tonf |
| E      | 27458.88     | 350-550 | tonf |

Once the weights of the models have been found, a comparison and differences can be made to know when it varies in percentage, as presented in Table 2 below.

### Table 2. Weights comparison and percentage

| $f'_c$-F$'_c$ | Dif. | U   | %  |
|-------------|-------|-----|----|
| 210-350     | 1415.41 | tonf | 5  |
| 210-550     | 1979.64 | tonf | 6  |
| 350-550     | 564.23  | tonf | 2  |
| 210-210-350 | 1436.07 | tonf | 5  |
| 210-350-550 | 3248.74 | tonf | 11 |
| 350-210-350 | 20.67   | tonf | 0.07 |
| 350-350-550 | 1833.33 | tonf | 6  |
| 550-210-350 | 543.56  | tonf | 2  |
| 550-350-550 | 1269.1  | tonf | 4  |

3.2. Models Periods

The results of the periods of the five models are presented in Table 3.

### Table 3. Fundamental periods of the models analyzed in the xx direction

| Models | $f'_c$ | Period | U |
|--------|-------|--------|---|
| A      | 210   | 1.969  | s |
| B      | 350   | 1.903  | s |
| C      | 550   | 1.698  | s |
| D      | 210-350 | 1.856 | s |
| E      | 350-550 | 1.71  | s |

3.3. Models's Dynamic Shear

Regarding the dynamic shear, they were found for both the X and Y shears of the five models. Which are presented in Table 4.
Table 4. Five models's dynamic cutters in X and Y

| f'c  | V(x)     | V(y)     | U       |
|------|----------|----------|---------|
| 210  | 1525.8482| 1726.2288| tonf    |
| 350  | 1458.8161| 1650.8277| tonf    |
| 550  | 1384.9101| 1626.1457| tonf    |
| 210-350| 1642.9101| 1622.2411| tonf    |
| 350-550| 1349.9831| 1575.3812| tonf    |

3.4. Maximum model drifts

The maximum allowable drifts were obtained for each structural model with its respective f'c as shown in Figure 1.

Likewise, the displacement generated at the upper level and at the performance point of the building before a pattern of incremental lateral loads and constant gravitational loads; whence, ductility, the mechanism of plastics kneecaps formation and damage to different seismic's levels demand will be evaluated. The latter implies the stiffness's progressive degradation and the structure collapse as can be seen in the following Figure 2.

3.5. Models Capacity Curve

About the capacity curve, it was obtained in both X and Y, which can be seen in the following figures 3 and 4.
The results obtained of the weight structure, was very advantageous for model E, since it weighs 11% less compared to model A. This is because sections of less moment of inertia were used, due to the increase in the modulus of concrete elasticity.

Also, in table 5 it is possible to show the total weight, the concrete volume used and the amount of formwork required for each model. Whereby, model E contemplates a 14.3% savings in concrete and 6.3% savings in formwork with respect to model A. However, for the remaining models the savings in concrete and formwork are within a range 3.7% -10.5%; 2.0% -4.6% respectively.

Table 5. Concrete and formwork quantity table regarding to model A

| Models | Weight | Concrete | Formwork |
|--------|--------|----------|----------|
|        | Ton    | M3 Saving| M2 Saving|
| A      | 30707.63 | 0.0%     | 10088 | 0.0% | 51438.6 | 0.0% |
| B      | 29292.22 | -4.6%    | 9663  | -4.2% | 50305.8 | -2.2% |
| C      | 28727.99 | -6.4%    | 9029  | -10.5%| 49054.6 | -4.6% |
| D      | 29271.56 | -4.7%    | 9710  | -3.7% | 50397  | -2.0% |
| E      | 27458.56 | -10.6%   | 8641  | -14.3%| 48203.8 | -6.3% |

On the other hand, table 6 shows the total cost of concrete per m3 and the total formwork cost per m2, for each model with its respective f’c. Therefore, model E has an increase of 2.2% of the total cost with respect to model A, while for the other models the percentage of increase in total cost is within a range of 3.5% - 9.6% with respect to the first model.

Table 6. Concrete and formwork total cost table regarding to model A

| Models | f’c | Concrete | Formwork | Total  | Saving |
|--------|----|----------|----------|--------|--------|
|        | Kg/cm2 | M3 | Cost($) | M2 | Cost($) | Cost($) | %    |
| A      | 210      | 10088 | 63.33 | 51438.6 | 12.62 | 1287880.70 | 0.0% |
| B      | 350      | 9663  | 79.70 | 50305.76 | 12.62 | 1404856.06 | 9.1% |
| C      | 550      | 9029  | 83.94 | 49054.56 | 12.62 | 1376822.65 | 6.9% |
| D      | 210      | 4740  | 63.33 | 24946.48 | 12.62 | 1332158.84 | 3.4% |
| E      | 350      | 4970  | 79.70 | 25450.48 | 12.62 | 1316148.85 | 2.2% |
| E      | 550      | 4563  | 83.94 | 24330.48 | 12.62 | 1290130.48 | 1.8% |

The periods of each structure were decreasing as the strength of the concrete increased, obtaining a shorter period in models C and E, having high strength concrete. For them, a more rigid structure was obtained limiting it to excessive displacements.

The results of the capacity curve that each model yielded according to the procedure of ASCE41-13 and FEMA 440 standards were the following.

- Model A: Maximum shear of 4536.81 ton, maximum displacement of 57.73 cm.
- Model B: Maximum shear of 3907.05 ton, maximum displacement of 50.10 cm.
- Model C: Maximum shear of 3810.46 ton, maximum displacement of 58.24 cm.

On the other hand, for models D and E, shear of 3399.46 and 3623.36 tons and maximum displacements of 46.40 cm and 48.75 cm respectively were obtained. However, the real ductility that
was obtained in each model resulting from the division of the ultimate displacement and yield, as well as the seismic reduction force coefficient were the following:

- Model A: \( \mu = 4.14 \rightarrow R = 5 \)
- Model B: \( \mu = 2.54 \rightarrow R = 3 \)
- Model C: \( \mu = 4.27 \rightarrow R = 5 \)
- Model D: \( \mu = 3.91 \rightarrow R = 5 \)
- Model E: \( \mu = 5.15 \rightarrow R = 6 \)

That is why, with these results it is shown that the E model is so far one of the most optimal models because it has greater ductility compared to the other models. Regarding the load patterns for each model, it was observed that for the E model, lower lateral force coefficients are obtained in both directions due to the combination of the modulus of elasticity that the structure possesses.

Finally, regarding the seismic performance points that were found based on the capacity curve and different demand spectra, they were the following: With period and effective damping of Teff and Beff.

- Model A: \( \text{Teff} = 2.506, \text{Beff} = 0.112 \) and a performance of \( D = 26.486 \text{ cm} \) was obtained.
- Model B: \( \text{Teff} = 3.034, \text{Beff} = 0.1844 \) and a performance of \( D = 44.71 \text{ cm} \) was obtained.
- Model C: \( \text{Teff} = 2.161, \text{Beff} = 0.1221 \) and \( D = 18.87 \text{ cm} \).
- Model D: \( \text{Teff} = 2.396, \text{Beff} = 0.1296 \) and \( D = 21.01 \text{ cm} \).
- Model E: \( \text{Teff} = 1.672, \text{Beff} = 0.1338, D = 14.46 \text{ cm} \).

4. Conclusions

As a final conclusion based on the results provided in the previous paragraphs, it was shown that model E, has a better seismic behavior with respect to the other models, because the structure is more rigid, it presents a reduction of its total weight in 11% with regard to the model A which has a normal resistance concrete, also, its ductility is 5.15 higher than all models and its performance point is within the ranges of life safety (LS), therefore, the structure is oversized, since the standards recommend that for this type of building (offices) the structure be within the immediate occupation (IO) range. On the other hand, in terms of the costs generated by the construction of a building with high-strength concrete, the E model represents a higher cost of 2.2% compared to the model with lower compression resistance, which is very satisfactory since a high strength concrete has a higher cost compared to a normal strength. However, it should be noted that the structure is in the life safety range, which indicates that the cross sections of the elements can be reduced by a greater percentage, leading to less use of concrete and formwork, thus reducing the structure total cost.

Finally, it is demonstrated that the use of high-strength concrete together with other less resistant concrete not only provides good seismic behavior and a much lower cost, but also, the reduction of construction materials provides several environmental benefits, which, makes it an extremely profitable project in the short and long term and especially that is in balance with the ecosystem.

References

[1] G. Risks “Earthquakes and their socio-economic consequences”, (https://link.springer.com/referenceworkentry/10.1007/978-3-642-35344-4_259), 2013.
[2] A. Nolan, “Handbook of Earthquake Engineering”, 2015.
[3] A. Mwafy, “Assessment of Material Strength Implications on Seismic Design of Tall Buildings through Collapse Analysis”, 2015.
[4] American Society of Civil Engineers, ASCE 41-13, “Seismic evaluatioHihn and retrofit of existing buildings”, 2014.
[5] M. Schmidt, “High Performance Concrete”: Research Developmente and Appocation in Europe. The 7th International Symposium on the utilization of High Strength?High Performance Concrete, ACI, Washington DC, USA, 2005: 51-78.
[6] YL. Voo, “Ultra High Performance Concrete”– Technology for Present and Future. ACI Singapore, Building Construction Authority Joint Seminar on Concrete for Sustainability, Productivity and The Future, 2017.
[7] Reglamento Nacional de Edificaciones, Norma Técnica Peruana de Edificación E.030 “Diseño Sismo Resistente”, 2018.
[8] R. Pendyala, “Full Range Behavior of High-Strength Concrete Flexural Members, Part 1”, ACI Structural Journal, V93, N1, 1996, --30-35.. 
[9] Reglamento Nacional de Edificaciones Norma Técnica Peruana de Edificación E.060 “Concreto Armado”, 2009.
[10] FEMA 440, Departmentt of Homeland Security Federal Emergency Management Agency: “Improvement of nonlinear static seismic analysis procedures”, 2005.