Incremental analysis including shrinkage, creep and constructive effects on reinforced concrete transfer beam

Sara de Oliveira Marques Luna[1], Rodrigo Barros[2], José Neres da Silva Filho[3]

[1] marquessara95@hotmail.com
[2] barrosrn@ufrn.edu.br
[3] jneres@ect.ufrn.br / UFRN - Programa de Pós-Graduação em Engenharia Civil

Abstract

The numeric modeling and methods of measurement applied to reinforced concrete structures could differ from the buildings' real conditions. The staged constructive method effects, in which loads are positioned on the floors below, and time strains, such as creep and shrinkage when taken into account in conventional structural analysis, occur in a simplified manner. For hyperstatic structure under high loads cases - as is the case of transfer beams - are even more relevant the mentioned effects, which could be crucial for ensuring structural safety. Therefore, it is indispensable to apply the incremental analysis method and knowing the consequences of stress and strains caused by construction loads and time strains. This study aims to model a 10-storey standard building on SAP 2000 software containing a hyperstatic transfer beam on the ground floor. It will be considered shrinkage, creep and constructive effects, analyzing a 2D gantry, trying to identify how these deformations influence the values of maximum moments in transfer beam. The main results comprise a negative bending moment increase near the central columns when compared to the conventional analysis performed for transfer beams.

Keywords: Reinforced concrete buildings. Constructive effects. Staged construction. Creep and shrinkage.

1 Introduction

The building construction process can be understood as a group of actions executed in a predefined sequence in order to enable the rise of a construction (LEITE, 2015). However, the structural analysis commonly used in current projects considers that all loads are applied to the structure at once, disregarding the effects of constructive process, in which the load are applied in stages as the construction proceeds.

The building storeys support the structure, as they are being cast, through the shoring systems, enabling the casting of an upper storey. The Brazilian Standard Code ABNT NBR 15696:2009 (ABNT, 2009) defines the shoring system as temporary structures capable of resist and transmitting all of the loads during casting until the concrete has enough strength to support itself.

The staged construction process generates loads other than usage loads. These loads are imposed on the structure due to the weight of workers and equipment used in construction and, thus are called construction or assembly loads. The staged construction analysis takes into account all constructions phases, applying loads selectively in each part of the structure considering the time-dependent behaviour due to concrete ageing.
shrinkage and creep (SANTIAGO FILHO; PEREIRA; SILVA, 2014).

Project design experience shows that construction and time-dependent effects, as creep and shrinkage, are not normally accounted for, although several studies state the importance of these factors, especially in multiple storeys high buildings. According to Marques, Feitosa and Alves (2017), despite being well known the importance in previous researches, incremental analysis and staged construction are not commonly used in design practice. The finite element method facilitates this type of analysis allowing its wider utilization abording all kinds of construction.

It is known that the effects addressed in the present paper influences structure deformation, thereby, modifying internal stresses in hyperstatic structure, once they are widely dependent on support displacements. Additionally, when one needs a larger span in a storey and the support, a column, ends up being suppressed in the pre-design stage, leading to a transfer beam, it is expected larger displacements, with crucial impacts on the structure. Those displacements must be accounted for in a more reliable manner, which is possible using incremental analysis.

This element is usually quite robust; however, deformations due to time-related phenomena and construction effects could modify the element deformation pattern, substantially influencing the beam’s internal stresses. As it is an intensely loaded element, which supports directly a column, the rupture of a transfer beam could lead to a general or partial collapse of the structure, thus it is required a careful analysis ensuring a considerable safety level. On the other hand, an overly conservative design process would increase unnecessarily the cost, due to large dimensions and high reinforcement ratio. Keeping that in mind is imperative to study this subject, seeking to reach a cost-effective and safe analysis.

2 Materials and methods

As object to the present paper, we have multiple storeys reinforced concrete buildings with a framed structure. The structural analysis was carried on with finite element software, SAP 2000, version 14. To account for creep and shrinkage effects, the EUROCODE 2 (2004) code of practice, as well as the CEB-FIP (1990) recommendations were used as SAP 2000 input parameters data. The construction effects were modelled through a methodology known as staged construction - an incremental numerical analysis method used to simulate a building’s construction stages.

The structure simulation was made in SAP 2000 using fixed supports to model the pile cap and deep foundation elements. The building’s floor typology (Figure 1a) was developed in order to resemble a typical medium standard building in the city of Natal-RN-Brazil, which have two planes of symmetry with a central hall for staircase and elevator shaft. A plain frame was modelled, in which the transfer beam is inserted, and two types of analyses were taken into account: conventional and incremental analysis.

The present building is residential and has type and ground floors, where columns P29 and P30 are withdrawn, leading to transfer beams shown in Figure 1b. The beams and columns concrete has $f_{ck} = 35$ MPa, elasticity modulus of $E = 28160$ MPa with CA-50 steel reinforcements. The storey span is 3 m, and concrete is considered to have a unit weight of 25 kN/m³, with a thermal dilation coefficient of 10-5 °C and Poisson coefficient of 0.20.
Figure 1: a) Formwork of type floors (cm); b) Formwork of ground floor (cm)

Source: research data
3.1. Loads

The considered loads can be classified as permanent loads; variable loads, which includes: structure assembly, wind loads and surcharge (or accidental); temperature, creep and shrinkage. The permanent loads are basically dead loads (from the structure itself or any other permanent non-structural material); this type of load makes the majority of all loads and it’s easily calculated with dimensions and unit weight.

The variable loads are due to elements as furniture and other actions inherent of human action commonly seen in a building. It is not as easily determinable as dead loads, a reason, why it is customary to use assumptions, presents in code of practice determined using statistical analysis.

Leite (2015) proposed that assembly loads are constructions actions that must be taken into account and they are due to the weight of materials used in construction, falsework and framework, worker and equipment weight and the load due to concrete casting. Regarding horizontal actions, we have loads from concrete casting and usage of construction equipment, which, according to ACI 347 (2014) – Guide to formwork, must be up to 2% of total vertical dead load or 1,5 kN/m for a framework support system.

Regarding the wind loads, a resultant value is applied on top of each storey, seeking to simplify the model, once the actual resultant load would include slightly above mid-level between first and second floors, considering the load’s variable distribution. The wind load is derived using guidelines presents in Brazilian standard code NBR 6123:1988, which has wind load formulations dependent on height above ground level. The basic wind speed in Natal is 30 m/s.

In the incremental load case, the wind is applied only on the last stage, as well as the surcharge, considering that the structure is already finished. As the wind is a variable load, it is unlikely that its action during construction would lead to relevant displacements, interfering in stage analysis. Therefore, it is consistent to insert those loads with all structures already constructed, as the same that would be done in conventional analysis.

Temperature loads will not be considered in the present paper, due to the low-temperature variation in Natal during all year. Creep and shrinkage are considered, and the methodology used will be further explained.

3.2. Model’s information

The foundation was considered as fixed support, which means that soil-structure interaction was not taken into account. The structure elements, namely beams and columns, were modelled using FRAME elements. The transfer beam was also modelled as a frame, once it can not be considered as a deep beam \( (l/h = 3.4/0.9 = 3.8 > 3.0) \).

In order to consider the construction schedule the load case used in the model was “Nonlinear staged construction” which uses an incremental type of analysis. To account for properties related to creep and shrinkage phenomena the option “Time-Dependent Material Properties” was used. A new storey was inserted every 14 days – that is a common construction cycle in Brazil – with exception of the last stage, where a surcharge is applied to the structure, characterizing the end of construction (totalizing 712 days, or 2 years).

The load values were derived according to Brazilian standard code ABNT NBR 6120:2019. To calculate dead load from non-structural masonry, unit weight of 13 kN/m³ was used, considering 2.4 m height and 0.14 m thick walls. Regarding the dead load from internal and external wall mortar covering, as well as the laying mortar, unit weight of 21 kN/m³ was considered.

To derive values used as assembly loads the authors referred to Menon and Nogueira (2015). Variable loads from assembly and execution operations consist in upper storey unit weight, formwork and shoring; the mentioned reference uses a 4.79 kN/m² value, which was used in the present paper. As slabs were not modelled – once 2D frames were considered – it is necessary to calculate the resultant load acting on the beams from the modelled storey. In order to do so, an influence area technique was employed.

The loading schedule follows a standard logical sequence regardless building’s number of storey. Table 1 shows a loading schedule for a 5-storey building.
Table 1 – Loading schedule for a 5-storey building.

| Floor | 1ª | 2ª | 3ª | 4ª | 5ª | 6ª |
|-------|----|----|----|----|----|----|
| 1º    | Dead load | Assembly load from upper storey | Dead load (non structural) | | | Surcharge + wind load |
| 2º    | Dead load | Assembly load from upper storey | Dead load (non structural) | Assembly load from upper storey | | Surcharge + wind load |
| 3º    | Dead load | Assembly load from upper storey | Dead load (non structural) | Assembly load from upper storey | | Surcharge + Wind load |
| 4º    | Dead load | | Assembly load from upper storey | | | Surcharge + Wind load |
| 5º    | Dead load | | Assembly load from upper storey | Surcharge + Wind load | | |

Stage duration: 14 days
Last stage start: 712 days

Source: research data

It is possible to sum up all permanent loads acting on beams, as well as both assembly loads (from slabs and the ones directly on beams). Concerning the surcharge, it must be considered separately, once the load combination requires a partial factor to reduce the magnitude of one of them. Table 2 brings briefly all vertical loads from a 2D frame modelled in this study.

Table 2 – Summarization of all loads.

|       | Permanent | Variable |
|-------|-----------|----------|
| Load  | 29.0 kN/m | 9.60 kN/m |
| Source| research data |

The wind load was derived using guidelines from Brazilian standard code ABNT NBR 6123:1988 (ABNT, 1988), with a wind basic velocity of 30 m/s for the region of Natal. Therefore, horizontal loads due to wind action were calculated acting on each storey through the construction. The referred load will be put on the model acting on each joint from the 2D frame through the several storeys using values brought in Table 3.

Table 3 – Wind loads calculation

| Storey | Height (m) | S1.S3 | S2 | V_L (m/s) | q (kN/m²) | H/L_y | C_0y | A_y (m²) | F_y (kN) |
|--------|------------|-------|----|-----------|-----------|-------|------|----------|----------|
| 1      | 3          | 1     |    | 0.72      | 21.50     | 0.19  | 1.08 | 67.8     | 20.74    |
| 2      | 6          | 1     |    | 0.78      | 23.44     | 0.37  | 1.08 | 67.8     | 24.67    |
| 3      | 9          | 1     |    | 0.82      | 24.66     | 0.56  | 1.08 | 67.8     | 27.30    |
| 4      | 12         | 1     |    | 0.85      | 25.57     | 0.74  | 1.12 | 67.8     | 30.42    |
| 5      | 15         | 1     |    | 0.88      | 26.29     | 0.93  | 1.18 | 67.8     | 33.89    |
| 6      | 18         | 1     |    | 0.90      | 26.90     | 1.11  | 1.2  | 67.8     | 36.07    |
| 7      | 21         | 1     |    | 0.91      | 27.42     | 1.30  | 1.22 | 67.8     | 38.11    |
| 8      | 24         | 1     |    | 0.93      | 27.88     | 1.48  | 1.25 | 67.8     | 40.38    |
| 9      | 27         | 1     |    | 0.94      | 28.29     | 1.67  | 1.27 | 67.8     | 42.25    |
| 10     | 30         | 1     |    | 0.96      | 28.67     | 1.85  | 1.3  | 67.8     | 44.40    |

Source: research data
The following load combinations were used: conventional with a surcharge as the main variable load; conventional with wind load as the main variable load; nonlinear staged construction with assembly load as the main variable load; nonlinear staged construction with a surcharge as the main variable load and nonlinear staged construction with the load as principal variable load. Table 4 shows the five combinations already mentioned.

| Case | Load combinations |
|------|--------------------|
| (A) Conventional with surcharge as principal | Fd = 1.4 * Permanent + 1.4 * Surcharge + 0.6 * Wind |
| (B) Conventional with wind as principal | Fd = 1.4 * Permanent + 1.4 * (0.5 * Surcharge + Wind) |
| (C) Incremental with assembly load as principal | Fd = 1.4 * Permanent + 1.4 * (Assembly + 0.5 * Surcharge + Wind) |
| (D) Incremental with surcharge as principal | Fd = 1.4 * Permanent + 1.4 * (0.5 * Assembly + Surcharge + 0.6 * Wind) |
| (E) Incremental with wind as principal | Fd = 1.4 * Permanent + 1.4 * (0.5 * Assembly + 0.5 * Surcharge + Wind) |

Table 4 – Load combinations on structure

In SAP 2000, creep and shrinkage can be accounted for in a nonlinear analysis as such as staged construction (“nonlinear staged construction”). In this type of analysis, it is possible to check for the option “time-dependent material properties” and, doing so, the program with seeking for all material properties assigned that are time-dependent. In material properties definition, one can address advanced properties to concrete and, in this case, it is possible to assign creep and shrinkage parameters from CEB-FIP (1990).

Relative air humidity was considered as 70%, that being an average value in the metropolitan region of Natal, where the present study takes place. The shrinkage was considered to occur seven days after concrete casting, once the curing procedure is finished. The shrinkage coefficient was taken as 5, based on code of practice guidelines. The fictional thickness was derived through Equation 1:

\[
h_0 = \frac{2 \times Ae}{u} = \frac{2 \times (0.2 \times 0.9)}{2 \times 0.2 + 0.9} = 0.16 \text{ m}
\]

where: \( Ae \) is cross-sectional area; \( u \) is part of the external perimeter in direct contact with air.

3 Results

Three different models were made varying transfer beam’s height: 90 cm, 120 cm and 150 cm. This was done in order to verify the influence of the transfer beam’s stiffness on overall structural behaviour. That influence was evaluated in terms of vertical displacements from the ground floor beam. Table 5 and Figure 3 shows a comparison between vertical displacements in the central node of the transfer beam in the 10-storey building model for all load combinations.

| Beam’s height (m) | (A) | (B) | (C) | (D) | (E) |
|------------------|-----|-----|-----|-----|-----|
| 0.90             | 0.007 | 0.007 | 0.032 | 0.026 | 0.024 |
| 1.20             | 0.005 | 0.005 | 0.023 | 0.018 | 0.017 |
| 1.50             | 0.003 | 0.003 | 0.017 | 0.013 | 0.012 |

Table 5 – Displacements of central transfer beam node for different beam’s height

Figure 3- Influence of transfer beam height in central node displacement
The results show that an increment on transfer beam’s height, regardless of the additional dead load, leads to smaller displacements for all analysed load combinations, independently of analysis type: conventional or nonlinear staged construction.

Not only vertical displacements are influenced by the beam’s stiffness, but the vertical load transmitted to the column resting over the transfer beam also varies. This happens due to the fact that a hyperstatic structure has the capacity to redistribute stresses according to stiffness distribution along with the structure. In this case, a stiffer transfer beam would lead to an increase of columns’ vertical load, which would, of course, increase normal stress in the element cross-section. Table 6 shows how normal force varies for different beam’s heights considering all load combinations for incremental and conventional analysis.

| Load combination | (A)  | (B)  | (C)  | (D)  | (E)  |
|------------------|------|------|------|------|------|
| Beam height (cm) |      |      |      |      |      |
| 90               | 1369 | 1219 | 1714 | 1678 | 1516 |
| 120              | 1726 | 1540 | 2249 | 2159 | 1959 |
| 150              | 1968 | 1759 | 2596 | 2466 | 2244 |

Load increasing in columns P29 and P30 is somewhat 15% to 30%. That behaviour implies a redistribution of stresses between other columns near the transfer beam, leading to a smaller normal force in these elements. Table 7 shows how normal force in columns P8 and P10 varies with the transfer beam’s height. The magnitude of normal force decreasing in those elements is between 10% to 20%.

| Load combination | (A)  | (B)  | (C)  | (D)  | (E)  |
|------------------|------|------|------|------|------|
| Beam height (cm) |      |      |      |      |      |
| 90               | 2255 | 2001 | 3966 | 3307 | 3048 |
| 120              | 1836 | 1643 | 3665 | 2940 | 2739 |
| 150              | 1397 | 1264 | 3326 | 2539 | 2395 |

3.1 Displacements for different load combinations

In order to compare different load combinations, only the 90 cm height transfer beam models were considered. The comparison is made for the middle...
node in the transfer beam (node 81 in the model) and

displacements are shown in Table 8. The vertical
displacement U3 (Z-axis direction), positive being
on a gravity direction. Regarding horizontal
displacement U1, positive values are in the same
direction as wind load application.

Table 8 – Horizontal and vertical displacements for all load combinations

| Load combination | (A)  | (B)  | (C)  | (D)  | (E)  |
|------------------|------|------|------|------|------|
| U1 – Horizontal displacement (m) | 0.002 | 0.004 | 0.005 | 0.005 | 0.008 |
| U3 – Vertical displacement (m)   | 0.008 | 0.007 | 0.032 | 0.026 | 0.246 |

Source: research data

With Figure 3 it is possible to see that
displacement's variation through construction stages
is linear, with very similar displacement increasing
in each stage, which occurs due to the similarity of
loads acting on each storey. At stage 13, although
there is no load increment, there is the passage of
time (544 days) and, due to creep and shrinkage, the
displacement increases. As all surcharge and wind
load are added at the same time in the last stage, it is
natural a more significant displacement increasing.

The largest displacements were found in load
combination case (C), with assembly load as the
main variable load. Table 9 and Figure 4 show the
displacement variation with incremental load
application in each nonlinear staged construction
analysis.

Table 9 – Vertical displacements in each loading stage for
nonlinear staged construction analysis

| Loading stage | U3 Displacement (m) |
|---------------|---------------------|
| 1             | 0.0001              |
| 2             | 0.0018              |
| 3             | 0.0024              |
| 4             | 0.0026              |
| 5             | 0.0029              |
| 6             | 0.0056              |
| 7             | 0.0084              |
| 8             | 0.0112              |
| 9             | 0.014               |
| 10            | 0.0167              |
| 11            | 0.0176              |
| 12            | 0.0185              |
| 13            | 0.0185              |
| 14            | 0.0202              |

Source: research data

Figure 4 - Vertical displacements' evolution (m) through each stage for nonlinear staged construction analysis (load combination C).

![DISPLACEMENT'S EVOLUTION THROUGH CONSTRUCTION STAGES](image)

Source: research data

It is important to notice that after the construction
of the 7th storey the displacement is already larger
than the ones obtained in conventional analysis.

For the conventional models, a case with a
surcharge as the main variable load led to larger
displacements, whilst in nonlinear staged
construction analysis, the assembly load as principal
variable load provided an extreme scenario, with larger displacements. Table 10 brings a comparison between those two load combinations.

Table 10 – Vertical and horizontal displacements for load combination A and C

| Load combination | (A) | (C) |
|------------------|-----|-----|
| U1 Displacement (m) | 0.0023 | 0.0045 |
| U3 Displacement (m) | 0.008 | 0.0324 |

Source: research data

In comparison between incremental and conventional analysis, vertical displacement showed three times increasing, whilst horizontal displacement increased two times.

An analysis considering all building frame structures can show vertical displacements in the middle joint for each storey – can be seen in Table 11. In addition, Figure 5 illustrates how vertical displacements increased between nonlinear staged construction and conventional analysis and through all storeys.

Table 11 – Vertical displacements for load combinations A and C for each storey

| Load combination | (A) | (C) |
|------------------|-----|-----|
| 1º storey | 0.008 | 0.0324 |
| 2º storey | 0.092 | 0.0371 |
| 3º storey | 0.0104 | 0.0401 |
| 4º storey | 0.0115 | 0.0480 |
| 5º storey | 0.0125 | 0.0491 |
| 6º storey | 0.0134 | 0.0493 |
| 7º storey | 0.0141 | 0.0485 |
| 8º storey | 0.0146 | 0.0467 |
| 9º storey | 0.0150 | 0.0439 |
| 10º storey | 0.0152 | 0.0400 |

Source: research data

It is possible to evaluate stress distribution in the transfer beam, comparing vertical displacements found in column-beam intersection nodes, as can be seen in Table 12.

It is possible to see a general displacement increasing due to additional assembly load as a variable load. However, in the column-transfer beam intersection node, displacement is three times the one shown in conventional analysis, whilst in other columns, those of which reach directly the foundation, have a larger displacement increment (about five times).

Table 12 – Vertical displacements in transfer beam-column intersection node for load combinations A and C

| Load combination | (A) | (C) |
|------------------|-----|-----|
| U3 Displacement (m) | | |
| 1º storey | 0.008 | 0.0324 |
| 2º storey | 0.092 | 0.0371 |
| 3º storey | 0.0104 | 0.0401 |
| 4º storey | 0.0115 | 0.0480 |
| 5º storey | 0.0125 | 0.0491 |
| 6º storey | 0.0134 | 0.0493 |
| 7º storey | 0.0141 | 0.0485 |
| 8º storey | 0.0146 | 0.0467 |
| 9º storey | 0.0150 | 0.0439 |
| 10º storey | 0.0152 | 0.0400 |

Source: research data
3.2 Internal stresses for conventional and nonlinear staged construction analysis load combination

In order to analyse the impacts on internal stresses, only the most unfavourable load combinations (load combination A and C) were accounted for. Results show a bending moment increasing of 40.74%, 39.72% and 34.88% considering, respectively middle of the first, second and third transfer beam’s span (Table 13).

Table 13 – Mid-span bending moments of transfer beam for all its spans

| Load combination               | (A)  | (C)  |
|-------------------------------|------|------|
| Middle section (FIRST SPAN)   | -378.84 | -533.18 |
| Middle section (SECOND SPAN)  | 811.56  | 1133.90  |
| Middle section (THIRD SPAN)   | -409.25 | -552.00  |

Source: research data

Table 14 brings shear force values through all transfer beam’s spans, especially in the supports and columns-transfer beam intersection node (one section on the right and the left of the load application point).

Table 14 – Shear force at column-beam nodes for load combinations A and C

| Load combinations          | (A)  | (C)  |
|----------------------------|------|------|
| Support 1                  | 588.83 | 528.50  |
| Support 2                  | -477.10 | -222.60 |
| Middle section to the left | -638.20 | -810.90 |
| Middle section to the right| 730.80  | 903.92  |
| Support 3                  | 589.50  | 335.40  |
| Support 4                  | -476.50 | -415.73 |

Source: research data

With those values, it is possible to see that the incremental analysis led to an increment of shear force near to where the main column unload on the transfer beam, while the shear forces on the other analysed sections (support sections) decreased. This behaviour highlights the stress redistribution mechanism that occurred in incremental analysis.

It can be observed larger normal compression stresses in columns with incremental analysis (Table 15). This behaviour is due to a higher load magnitude caused by assembly load addition. However, this stress increasing is milder for transfer beam’s columns (P29 and P30), being something about 25%, while the other columns reached up to 40% increase.

This subtle increment occurs due to a larger displacement in the transfer beam with incremental analysis, which is similar to the displacement for a lower stiffness transfer beam evaluated previously with beam’s height modification in item 5.1. Therefore, the staged construction and time effects lead to a displacement increase and stress redistribution, which causes less stress increment for the central column.

Table 15 – Normal force at column-transfer beam intersection node for load combination A and C

| Load combination          | (A)  | (C)  |
|----------------------------|------|------|
| P2=P4                     | 1658.5  | 2349.9  |
| P8=P10                    | 2832.3  | 3881.9  |
| P29=P30                   | 1349.0  | 1714.9  |
| P18=P21                   | 3025.0  | 4084.8  |
| P24=P27                   | 2059.8  | 2747.3  |

Source: research data

The same analysis can be made regarding ground floor columns (Table 16) and it shows that the ones closer to the transfer beam loading point have a larger stress increment. All of them also have a stress increase due to assembly load.

Table 16 – Normal force at ground floor columns for load combination A and C

| Load combination          | (A)  | (C)  |
|----------------------------|------|------|
| P2=P4                     | 1102.50 | 1860.50 |
| P8=P10                    | 2255.11 | 3881.90 |
| P29=P30                   | 1349.0  | 1714.9  |
| P18=P21                   | 3025.0  | 4084.8  |
| P24=P27                   | 2059.8  | 2747.3  |

Source: research data
The transfer beam’s stress redistribution affects directly forces at top of ground floor columns, once it is a continuous beam. Table 17 shows changes in ground floor columns. As the columns get closer to the central span (the one which has the column-transfer beam intersection node) their bending moments suffer a higher increment, even leading to a more than double of bending moment in the case of P8 and P10. However, it is important to point out that the far end columns have their bending moment reduced, which occurs due to the already mentioned stress redistribution.

Table 17 – Bending moments at the top of ground floor columns for load combinations A and C

| Load combination | (A) | (C) |
|------------------|-----|-----|
| P2=P4            | 213.6 | 205.0 |
| P8=P10           | 40.5  | 86.5 |
| P18=P21          | 291.3 | 304.7 |
| P24=P27          | 52.5  | 42.0 |

Source: research data

Evaluating also shear forces at the ground floor column’s top (Table 18) it is possible to conclude that there are not relevant stress changes due to incremental analysis. This behaviour occurs because shear forces (and stresses) at columns are influenced mainly by wind load, and other horizontal loads which, in incremental analysis, are applied in the structure all at once only in the last construction stages, a similar scenario to what would happen in conventional analysis.

Table 18 – Shear force at the top of ground floor columns for load combinations A and C

| Load combination | (A) | (C) |
|------------------|-----|-----|
| P2=P4            | 128.4 | 146.0 |
| P8=P10           | 11.80 | 11.20 |
| P18=P21          | 167.20 | 167.30 |
| P24=P27          | 0.05  | 18.10 |

Source: research data

3.3 Building’s height influence

A point that cares to be analysed is how displacement varies according to the building’s height when compared to conventional and nonlinear construction staged analysis. The numerical model used to evaluate this is the one with a 90 cm height transfer beam. At Table 19 it is possible to see a significant displacement increasing when comparing a 4-storey to a 6-storey building and, as we continue to raise the building’s height to 8 and 10-storeys, the displacement surely get larger. Once the applied load is higher, but, the increasing proportion it lower, whatever is the load combination.

Table 19 - Vertical displacements of the central node on transfer beam for load combinations A, B, C, D and E

| Load combination model / Building’s height | (A) | (B) | (C) | (D) | (E) |
|-------------------------------------------|-----|-----|-----|-----|-----|
| MOD_90_4STR                               | 0.03 | 0.04 | 0.72 | 0.39 | 0.41 |
| MOD_90_6STR                               | 0.31 | 0.29 | 1.61 | 1.17 | 1.13 |
| MOD_90_8STR                               | 0.56 | 0.51 | 2.44 | 1.9  | 1.80 |
| MOD_90_10STR                              | 0.79 | 0.73 | 3.24 | 2.6  | 2.46 |

Source: research data

Displacements for a 6-storey building model are up to 10 times larger than the 4-storey building model, whilst comparing 10-storey to 8-storey models, displacement increment was only about 40%. These results show an interesting behaviour regarding displacement increment with the building’s height: for taller buildings, the displacement increasing tends to be more subtle. Taking load combination A, for example, the proportional increase is 10 times; 1.8 times and 1.4 times between models 4 to 6-storeys; 6 to 8-storeys and 8 to 10-storeys, respectively. While for load combination C, with incremental analysis, those increasements turned into 2.2, 1.51 and 1.33.

When one compares the difference between displacements found in conventional and incremental analysis models it has that for lower buildings lower increments are observed. Taking into account load combinations (A) and (C) it has displacement increment proportion of 24, 5.2, 4.4 and 4.1 to models with 4, 6, and 10-storeys, respectively.

These results show the importance of nonlinear staged construction analysis for lower height
buildings with a transfer beam. However, for conventional buildings (that mean, without a transfer beam) the effects of incremental construction analysis are more sensitive with increasing storeys number.

4 Conclusions

Buildings with transfer beams require more careful analysis, once we are dealing with a risk element (a transfer beam failure is prone to cause a progressive collapse), not mentioning the cost-effective factor (an overly conservative analysis will lead to a reinforcement ratio higher than it is necessary). Modifying the transfer beam’s stiffness made it possible to evaluate its effects on structural behaviour – for stiffer beams, the column supported by it carries a higher compressive load and, due to stress redistribution, a lower normal force is transmitted to the other columns.

The present paper brings an analysis of how displacements interfere with stress redistribution showing the importance of nonlinear staged construction analysis with the consideration of time-dependent effects (as creep and shrinkage), once this type of approach leads to larger displacements and deformations, which one can consider closer to reality. Regarding a comparison between conventional and incremental models: the surcharge as the main variable load was the critical load combination in conventional analysis, whilst, for staged construction, the assembly load as the main variable load led to the largest deformations, showing the importance of considering this load, which is, most of the times, disregarded.

The consequences of not taking into account the construction effect, as well as creep and shrinkage, can be seen when one looks at the most unfavourable results of the two types of models (conventional and incremental). It is important to highlight a 3 times displacement increasing in the column-transfer beam intersection node and a 35% larger bending moment resultant of bending moments at the top of ground floor columns.

The influence of a building’s height has been showing that lower height structures deserve more careful analysis, once its deformation increase was proportionally more significant. As an example, one can be referred to a comparison between 4 to 6-storey buildings and 8 to 10-storey buildings. The first one led to a much higher increasing proportion. Besides that, the incremental analysis effects are also more considerable.

References
SANTIAGO FILHO, H. A.; PEREIRA, R. G. S.; SILVA, F. A. N. Comportamento estrutural de edifícios de concreto armado devido aos efeitos construtivos. In: CONGRESSO BRASILEIRO DO CONCRETO, 56., Natal. Anais .... Natal: IBRACON, 2014. In Portuguese.

SAP 2000 — Integrated Solution for Structural Analysis and Design. CSI Analysis reference Manual. Advanced version 14.0.0, Computers and Structures, Inc.