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

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ABSTRACT: The numeric modelling 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 structures under high loads cases – as is the case of transfer beams – even more, relevant to the mentioned effects, which could be crucial for ensuring structural safety. Therefore, it is indispensable to apply the incremental analysis method and know 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, and trying to identify how these deformations influence the values of maximum moments in the 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: constructive effects; creep and shrinkage; reinforced concrete buildings; staged construction.

Análise incremental, incluindo retração, fluência e efeitos construtivos em vigas de transferência de concreto armado

RESUMO: *A modelagem numérica e os métodos de simulação aplicados a estruturas de concreto armado podem diferir das condições reais das edificações. Os efeitos do processo construtivo, no qual as cargas são posicionadas em cada pavimento, e as deformações ao longo do tempo, como fluência e retração, quando consideradas nas análises convencionais, ocorrem de forma simplificada. Nas estruturas hiperestáticas sob elevadas cargas, como é o caso das vigas de* *transição, são ainda mais relevantes os efeitos anteriormente mencionados, que podem comprometer a segurança estrutural. Portanto, é imprescindível aplicar o método de análise incremental e conhecer os efeitos para as tensões e deformações causadas pelas cargas construtivas, bem como pelas deformações ao longo do tempo. O objetivo deste trabalho é apresentar a modelagem de um edifício de 10 pavimentos no software SAP 2000, considerando a existência de uma viga de transição no pavimento térreo. São considerados os efeitos de retração, fluência e efeitos construtivos ao analisar um pórtico da estrutura, procurando identificar como essas deformações influenciam os valores de esforços internos na viga de transição. Os principais resultados indicam um aumento do momento fletor negativo próximo aos pilares centrais, quando comparados aos resultados da análise convencional.*

Palavras-chave: *: concreto armado; construção em estágios; efeitos construtivos; fluência e retração.*

1 Introdution

The building construction process can be understood as a group of actions executed in a predefined sequence to enable the rise of 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 (ABNT, 2009) defines the shoring system as temporary structures capable of resisting 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 considers all construction 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, such 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 importance in previous research, incremental analysis and staged construction are not commonly used in design practice. The finite element method facilitates this analysis, allowing its wider utilization to abording all kinds of construction.

It is known that the effects addressed in the present paper influence structure deformation, thereby, modifying internal stresses in hyperstatic structures, 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, are 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.

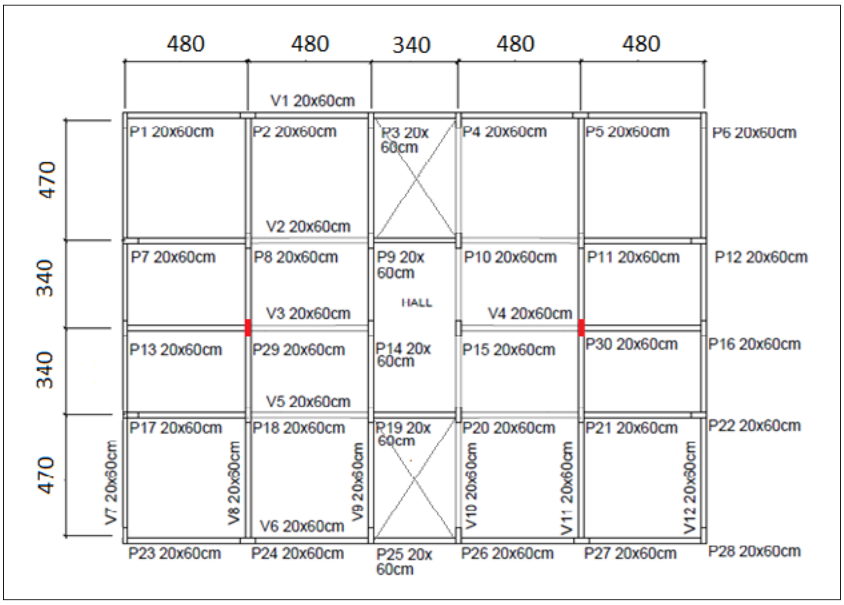
2 Materials and methods

As an 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 (CSI, 2014). To account for creep and shrinkage effects the Eurocode 2 (CEN 2004) code of practice, as well as the CEB-FIP (CEB, 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.

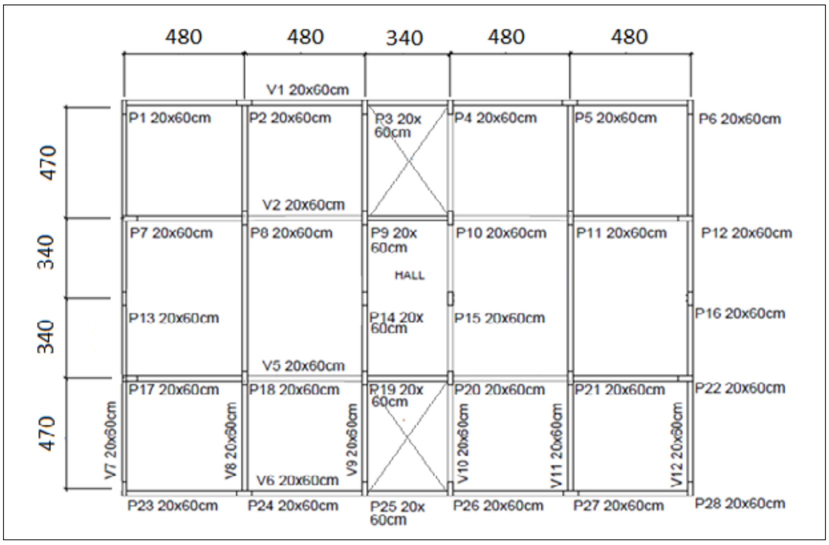
The structure simulation was made in SAP 2000 using fixed supports to model the pile cap and deep foundation elements. The building's floor typology (Figure 1a) was developed to resemble a typical medium standard building in the city of Natal-RN-Brazil, which has two planes of symmetry with a central hall for the 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 fck = 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 a Poisson coefficient of 0.20.

Figure 1 ▶
a) Formwork of type loors (cm); b) Formwork of ground floor (cm).
Source: research data



(a)



(b)

2.1 Loads

The considered loads can be classified as permanent loads; variable loads, which include: structure assembly, wind loads and surcharge (or accidental); temperature, creep and shrinkage. The permanent loads are 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 such as furniture and other actions inherent in 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, present in the code of practice determined using statistical analysis.

Leite (2015) proposed that assembly loads are construction 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 – Guide to formwork (ACI, 2014), 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 the first and second floors, considering the load's variable distribution. The wind load is derived using guidelines presented in Brazilian standard code NBR 6123 (ABNT, 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 actuation 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.

2.2 Model 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).

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 (ABNT, 2019). To calculate dead load from non-structural masonry, a 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, a 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 of 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. To do so, an influence area technique was employed.

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

Table 1 ▼
Loading schedule for a 5-storey building.
Source: research data

	Loading stages (load)					
Floor	**1st**	**2nd**	**3rd**	**4th**	**5th**	**6th**
1st	Dead load	Assembly load from upper storey	Dead load (non-structural)			Surcharge + wind load
2nd		Dead load	Assembly load from upper storey	Dead load (non-structural)		Surcharge + wind load
3rd			Dead load	Assembly load from upper storey	Dead load (non-structural)	Surcharge + wind load
4th				Dead load	Assembly load from upper storey	Surcharge + Wind load
5th					Dead load	Surcharge + Wind load
Stage duration: 14 days			Last stage start: 712 days			

It is possible to some 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.
Source: research data

Resultant load acting on beams from 2D frame (except dead load)		
Permanent	Permanent	29.0 kN/m
Surcharge	Variable	9.60 kN/m
Assembly	Variable	24.0 kN/m

The wind load was derived using guidelines from Brazilian standard code ABNT NBR 6123 (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 throughout 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 calculus.
Source: research data

Storey	**Height (m)**	**S1.S3**	**S2**	v_k **(m/s)**	q **(kN/m²)**	H/L_y	C_{ay}	A_y **(m²)**	F_{vy} **(kN)**
10	30	1	0.96	28.67	503.82	1.85	1.3	67.8	44.40
9	27	1	0.94	28.29	490.72	1.67	1.27	67.8	42.25
8	24	1	0.93	27.88	476.48	1.48	1.25	67.8	40.38
7	21	1	0.91	27.42	460.84	1.30	1.22	67.8	38.11
6	18	1	0.90	26.90	443.42	1.11	1.2	67.8	36.07
5	15	1	0.88	26.29	423.66	0.93	1.18	67.8	33.89
4	12	1	0.85	25.57	400.67	0.74	1.12	67.8	30.42
3	9	1	0.82	24.66	372.87	0.56	1.08	67.8	27.30
2	6	1	0.78	23.44	336.92	0.37	1.08	67.8	24.67
1	3	1	0.72	21.50	283.32	0.19	1.08	67.8	20.74

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.

Table 4 ▼
Load combinations on structure.
Source: research data

Case	**Load combinations**
(A) Conventional with a surcharge as principal	$Fd = 1.4 \cdot$ permanent + $1.4 \cdot$ (surcharge + $0.6 \cdot$ wind)
(B) Conventional with the wind as principal	$Fd = 1.4 \cdot$ permanent + $1.4 \cdot$ ($0.5 \cdot$ surcharge + wind)
(C) Incremental with assembly load as principal	$Fd = 1.4 \cdot$ permanent + $1.4 \cdot$ (assembly + $0.5 \cdot$ surcharge + $0.6 \cdot$ wind)
(D) Incremental with surcharge as principal	$Fd = 1.4 \cdot$ permanent + $1.4 \cdot$ ($0.5 \cdot$ assembly + surcharge + $0.6 \cdot$ wind)
(E) Incremental with the wind as principal	$Fd = 1.4 \cdot$ permanent + $1.4 \cdot$ ($0.5 \cdot$ assembly + $0.5 \cdot$ surcharge + wind)

In SAP 2000, creep and shrinkage can be accounted for in a nonlinear analysis 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, in doing so, the program with seeking all material properties assigned that are time-dependent. In material properties definition, one can address advanced properties of concrete and, in this case, it is possible to assign creep and shrinkage parameters from CEB-FIP (CEB, 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 the code of practice guidelines. The fictional thickness was derived through Equation 1:

$$h0 = \frac{2 \times A_c}{u} = \frac{2 \times (0.2 \times 0.9)}{(2 \times 0.2 + 0.9)} = 0.16\,m \tag{1}$$

where: *Ac* is the cross-sectional area; *u* is part of the external perimeter in direct contact with air.

3 Results

Three different models were made with varying transfer beam heights: 90 cm, 120 cm and 150 cm. This was done 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 2 show a comparison between vertical displacements in the central node of the transfer beam in the 10-storey building model for all load combinations.

Table 5 ▶
Displacements of central transfer beam node for different beam height.
Source: research data

Beam's height (m)	(A)	(B)	(C)	(D)	(E)
	Vertical displacement in the central node (m)				
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

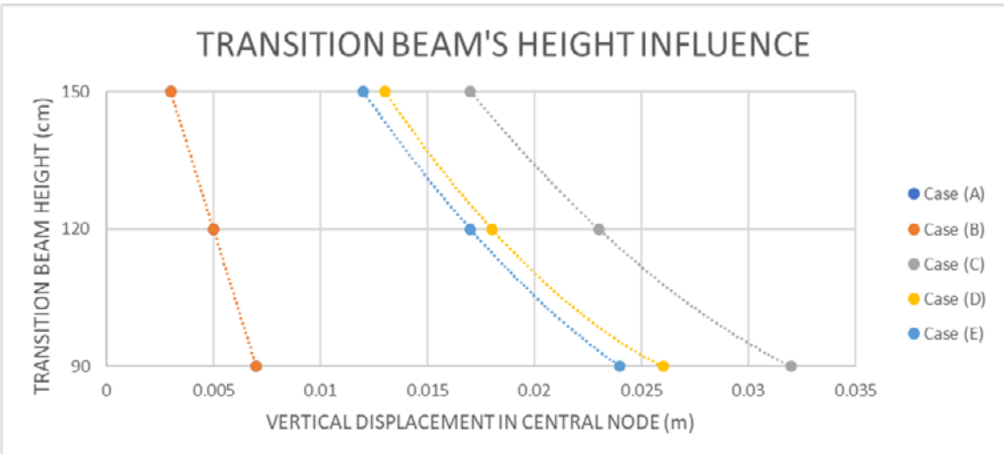


Figure 2 ▶
Influence of transfer beam height in central node displacement.
Source: research data

The results show that an increment in the 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 because a hyperstatic structure can redistribute stresses according to stiffness distribution along with the structure. In this case, a stiffer transfer beam would lead to an increase in the 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 heights considering all load combinations for incremental and conventional analysis.

Table 6 ▶
Normal force in column resting over the transfer beam for different beam's height.
Source: research data

Load combination	(A)	(B)	(C)	(D)	(E)
Beam height (cm)	**Normal force at P29 = P30 (kN)**				
90	1369	1219	1714	1678	1516
120	1726	1540	2249	2159	1959
150	1968	1759	2596	2466	2244

Load increases in columns P29 and P30 are 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%.

Table 7 ▶
Normal force at support columns (P8 and P10) of the transfer beams for different beams' height.
Source: research data

Load combination	(A)	(B)	(C)	(D)	(E)
Beam height (cm)	**Normal force at P8 = P10 (kN)**				
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

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.

Tabel 8 ▶
Horizontal and vertical displacements for all load combinations.
Source: research data

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

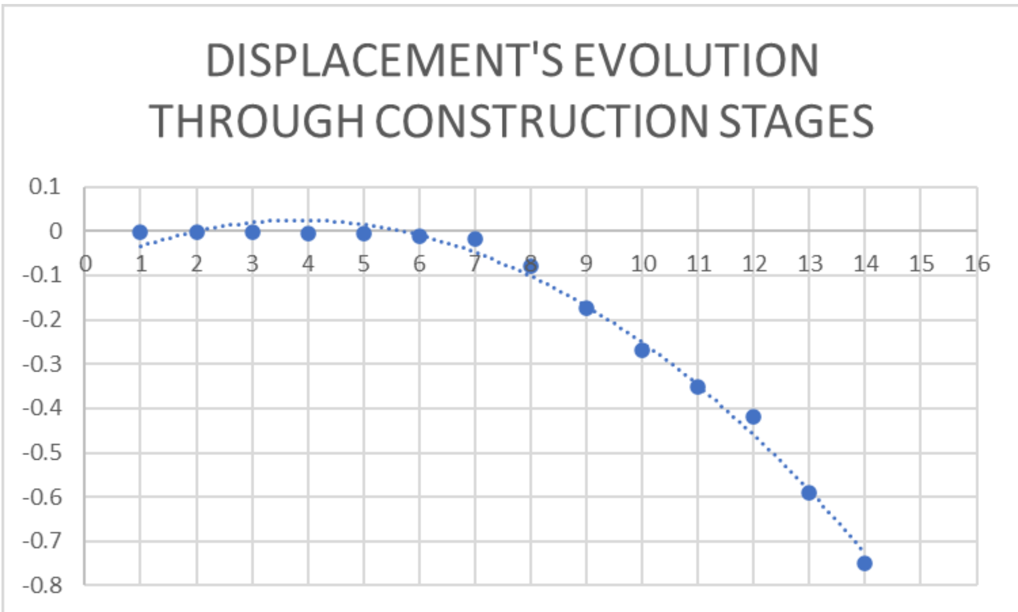
With Figure 2 it is possible to see that displacement's variation through construction stages is linear, with very similar displacement increasing in each stage, which occurs due to the similarity of loads acting on each storey. At stage 13, although there is no load increment, there is the passage of 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 naturally a more significant displacement increase.

The largest displacements were found in load combination case (C), with assembly load as the main variable load. Table 9 and Figure 3 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.
Source: research data

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

Figure 3 ▶
Vertical displacements' evolution (m) through each stage for nonlinear staged construction analysis (load combination C).
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 the 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 combinations A and C.
Source: research data

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

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

An analysis considering all building frame structures can show vertical displacements in the middle joint for each storey – which can be seen in Table 11. In addition, Figure 4 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.
Source: research data

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

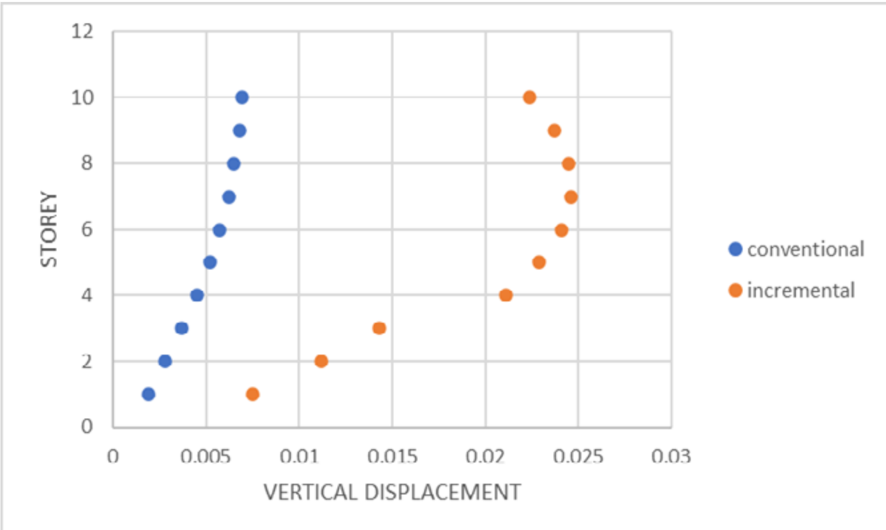


Figure 4 ▶
Vertical displacements (m) for each storey considering load combinations A and C.
Source: research data

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 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.
Source: research data

Load combination	(A)	(C)
P2 = P4	0.001	0.0052
P8 = P10	0.002	0.0098
P29 = P30	0.008	0.0324
P18 = P21	0.0021	0.0101
P24 = P27	0.0014	0.0061

3.2 Internal stresses for conventional and nonlinear staged construction analysis load combination

To analyse the impacts on internal stresses, only the most unfavourable load combinations (load combinations A and C) were accounted for. Results show a bending moment increase 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.
Source: research data

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

Table 14 brings shear force values through all transfer beams' 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 columns-beam nodes for load combinations A and C.
Source: research data

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

With those values, it is possible to see that the incremental analysis led to an increment of shear force near where the main column unloads 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 increase is milder for the 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 the beam's height modification. 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.
Source: research data

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

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 combinations A and C.
Source: research data

Load combination	(A)	(C)
P2 = P4	1102.50	1860.50
P8 = P10	2255.11	3965.90
P18 = P21	2428.50	4149.10
P24 = P27	1616.12	2370.60

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 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.
Source: research data

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

Evaluating also shear forces at the ground floor column's top (Table 18) it is possible to conclude that there are no 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.
Source: research data

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

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. In Table 19 it is possible to see a significant displacement increase 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 gets 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 the transfer beam for load combinations A, B, C, D and E.
Source: research data

Load combination model	(A)	(B)	(C)	(D)	(E)
/ Building's height	Vertical displacements at central node (cm)				
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

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 increase 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 proportions 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 means, 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 to mention 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 (such 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 increase 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.

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