Structural Joints in Large Building Structures

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Abstract

The present work intends to evaluate the need of structural joints in large dimension building structures. These structural joints are currently used to guarantee an acceptable serviceability behaviour. For building structures, actions such as concrete shrinkage or other imposed deformations may, in fact, contribute for an inconvenient concrete cracking or an unacceptable deformations on vertical structural elements which influence the non-structural elements behaviour. Solutions that include structural joints may, however, become themselves the origin of behaviour problems in buildings causing thermal bridges or even becoming water permeable. Thus, it is important to evaluate the real need of these joints and study the usage of partial joints which, by affecting only bottom storeys, may avoid problems such as water permeability in top floors.

The evaluation of structural joints necessity is done by evaluating the serviceability behaviour of the structures. To evaluate this kind of behaviour the sections of all structural elements, slabs, beams and columns, were analyzed in order to obtain crack width estimations for: constant flexure moments and variable axial force, in slabs and beams; constant axial force and variable flexure moments, in columns. Additionally, columns’ diferencial deformations were analyzed in order to estimate the effects on non-structural elements behaviour.

Cross-referencing the information obtained in this analysis and the information obtained from the models on SAP2000 for each structure, one concludes that the need for structural joints depends not only on the length but also on the type of structure. So, for the studied structures, it was possible to verify an acceptable serviceability behaviour for a 200 meter structure without the inclusion of any structural joins. On the other hand, for a 100 meter structure with important restriction to storey deformation the inclusion of one partial joint 3 storeys high was necessary.

1 Introduction

This work’s purpose is to evaluate the need of structural joints in large building structures of armed concrete. There are two main reasons for the use of this joints, the first one is to avoid great asymmetries or brusque transitions in the rigidity of the structures. This asymmetries may cause, if not avoided, a bad dynamic behavior of the structures which is particularly relevant in an earthquake situation. The second reason is to avoid serviceability behavior problems which are, in this type of structures, caused mainly by imposed deformations action, such as concrete shrinkage or thermal variations. This second reason is this study’s object.

Structural joints, however, may become themselves the origin of serviceability problems, as they can form thermal bridges and permeability problems, once the materials that fill the joint start to worn out. Thus, it is important to evaluate the real need of these joints and study the possibility of using partial joints which may avoid permeability problems in top storeys if they only affect bottom ones. Figure 1 shows the diference between the behaviour of a total joint and a partial joint affecting the first two bottom floors.
Figure 1: Undeformed structure (left) and deformed structure (right) with a structural joint: a) Total; b) Parcial.

2 Material behaviour

Since this study main focus is related with imposed deformations, one should acknowledge the difference between two types of actions. Direct loads, such as gravity loads, are forces applied to structures, thus, in order for the system to remain balanced, the structures must reply with reaction forces of its own which means there is the need of resistance. Indirect actions, such as imposed deformation, cause auto-equilibrate reactions in hyperstatic structures. This reactions, however, are proportional to the available resistance and may not cause a lack of it which means only ductility is required in order to maintain balance.

After understanding the different types of action one must acknowledge how material behaviour may influence the overall structural behaviour. This is specially important in these study since one of the main actions, concrete shrinkage, is part of the material behavior.

One of the best ways to characterize a material is through it’s tension-strain relation. On serviceability behavior it isn’t common to overcome 40% of concrete resistance which means it may be acceptable to characterize the tension-strain relation solo by the material’s modulus of elasticity. When it comes to long term actions, such as concrete shrinkage, however this modulus needs to be readjust to take in account concrete’s creeping and aging effects. Figure 2 shows how, for the studied concrete, the adjusted modulus of elasticity evolves with time.

Figure 2: Adjust modulus of elasticity time evolution.
Concrete shrinkage strain is itself a time dependent value which was estimated to evolve, in this case, as showed in Figure 3.

![Figure 3: Shrinkage strain time evolution (total, drying and autogenous).](image)

Since these two parameters are time dependent so must their combined effects in structures be, thus it is possible to relate them both in order to reveal when this effects will have more significance. The parameter $\zeta$, presented on the following equation does just that. The presented tensions ($\sigma$), however, should not be understood as the actual value that will be acting in the structures but as proportional to it for the $\zeta$ parameter does not intend to calculate forces or tensions but to estimate their greatness with time evolution.

$$\sigma_t \approx \sigma_0 \times \zeta(t) = E_{c,\text{ajust}}(t) \times \varepsilon_{c s}(t)$$ (1)

Where, $\sigma_0 = E_{c,28} \times \varepsilon_{c s}(t_\infty)$.

Figure 4 presents the graphical evolution of $\zeta$ for the present study.

![Figure 4: $\zeta$ parameter evolution in time.](image)
3 Structural behaviour

Once the understanding of material behaviour and actions type is accomplished there is the need to understand how acting in each structural element is reacting to those actions. On one hand we have slabs and beams which have gravity loads causing flexural moments and imposed deformations causing axial forces. These kind of behavior is demonstrated by Figure 5.

![Figure 5: Comparison between the behaviour of an isolated axial action and an axial action superposed with vertical loads [1].](image)

On the other hand, column elements have gravity loads causing axial forces and imposed deformations causing flexural moments. Their behaviour is similar to the one illustrated in Figure 6.

![Figure 6: Comparison between the behavior in simple flexure and in composed flexure with constant axial force [2].](image)

4 Analysis and conclusions

In accordance with what was previously mentioned, the evaluation of serviceability behaviour, in this study, is based on two parameters: crack width and deformations in non-structural elements. The first parameter estimation is based on Eurocode 2 §7.3 and the second parameter limitation is based in an
adjustment for vertical elements of §7.4 of the same document in the lacking of specific limits. This adjustment’s purpose is to adapt the limits of deformation in horizontal elements to vertical elements with the comparison of the different length between inflection points on both type of elements.

The structures in analysis are four storeys high, have 15 meter depth (7.5 meters between columns), 100, 150 and 200 meter length (7.2 meters between columns) and are composed only by slabs, beams and columns with one exception. This exception is the "100 meter structure with walls" which has four walls one in each corner. These walls have the building’s height and a $7.20 \times 0.40 \, m^2$ section, with the 7.2 meters parallel to the greatest length of the building in order to create a greater restriction to the storeys axial deformation. The slabs have a 0.20 $m$ thickness and the beams have a unique $0.70 \times 0.40 \, m^2$ section. The columns have two kinds of section, $0.60 \times 0.60 \, m^2$ in the interior and $0.60 \times 0.40 \, m^2$ in the contour. Figure 7 globally illustrates these structures.

The considered materials were C25/30 concrete and A500 rebars and the analysis was made, in accordance with $\zeta$ top values shown in Figure 4, considering shrinkage strain and adjusted modulus of elasticity at 450 days.

To obtain the forces applied in the elements and columns differential deformations for each structure, for the quasi-permanent combination of actions, the structures were modeled in SAP2000. The main conclusions were the following:

100 m Structure - without the inclusion of any structural joint the maximum crack width estimation was $w_k = 0.23\, mm$, $w_k = 0.24\, mm$ and $w_k = 0.37\, mm$ for columns, slabs and beams which means there are no problems forecasted at this level. Column deformation agrees with the maximum values stipulated and consequently the serviceability behavior is satisfactory for this structure without the inclusion of structural joints.

100 m Structure with Walls - for these structures, and due to the high level of restriction to the storey axial deformation, there is the need to include a partial joint affecting three storeys. In spite of that, the crack width is estimated to be high at $w_k = 0.65\, mm$ in the third storey contour beams. This problem, however, may be suppressed with a local reinforcement at these beams and, accordingly, the structure presents a satisfactory structural behavior with these kind of joint.

150 m Structure - in this case, and without any structural joint, the maximum crack width estimation is $w_k = 0.45\, mm$ on contour beams which may or may not be a problem depending on the aggressiveness of the environment. However, and as mentioned earlier, this crack width may be reduced with local reinforcement. Column deformation is slightly above the more exigent criteria but, due to the timing of non-structural elements construction, the deformation obtained in SAP2000 models may be
reduce down to 70% of its original value for the narrower criteria verification. Thus, one would predict an acceptable serviceability behavior for this structure without the inclusion of structural joints.

**200 m Structure** - in the line of what has happen with the other structures, the only possible problems with crack width were found in contour beam elements with \( w_k = 0.54 \text{mm} \) and, here too, a local reinforcement is needed. Without structural joints the column deformation level is high and can only meet the narrower criteria if the 70% reduction mentioned before is applied. These reduction, however, is valid and so there is no reason not to apply it. This means that this structure also presents a satisfactory serviceability behavior prediction with no structural joint.

For the analyzed structures, an acceptable serviceable behavior is expected for structures without walls up to 200 meter length without the inclusion of any joint and for structures with heavy restriction to storey axial deformation and up to 100 meter length with the inclusion of one partial joint affecting three storeys. None of the analyzed structures needed a full joint to behave properly in a service situation however this behavior depends not only on structural length but also on the structure type itself which means these conclusion should not be generalized.

**References**
