JOINTLESS CONSTRUCTION – OPTIMIZATION OF ASEISMIC MULTI-STOREY BUILDINGS

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ABSTRACT

In current practice, large floor plans are treated with intermediate joints to create statically independent structures with layout dimensions lower than 35m. This scheme is used to minimize and “ignore” the effect of in-service restraints in building design. Moreover, in aseismic buildings the required joint widths are further increased due to seismic requirements which result in significant joint costs. Recently, research efforts on the development jointless high-rise structures have changed the perspective on the phenomena that induce restraints and the quantitative tools to calculate them significantly. It has been found that stiffness of vertical structural members plays a key role in in-service demand. In particular, although large stiffness of vertical structural members influences positively the seismic structural response, it increases the intensity of in-service loading. As a result, it is important to develop effective novel methods to control the stiffness of vertical members apart from changing cross-section dimensions or ground floor height. The aim is to compromise the opposite in-service and seismic requirements. In the present study, a proposal for “controlling” the stiffness of columns of multi-story structures is investigated analytically and evaluated quantitatively. Based on the inter-story drift effects on jointless high-rise structures, it is illustrated that even large floor plans can be treated with the optimal solution of eliminating joints. The analysis was performed on a 15-story aseismic building with 80mx20m layout dimensions and the proposed method was applied. The results indicate the improved structural safety, especially regarding seismic requirements, and the accommodation of in-service restraints.

Keywords: reinforced concrete buildings; jointless structure; seismic design; in-service loads; stiffness control;

1. INTRODUCTION

In building design practice and current Codes (CEN [Comité Européen de Normalisation] 2002), intermediate joints are included in structures of large floor plan, i.e. with dimensions larger than 35m, as means to avoid increased stresses due to imposed deformations in the serviceability limit state. The vulnerable members of such buildings are the columns of the ground floor, as the differential deformations between ground and first floor slab induce additional stresses on these columns. Moreover, in seismic areas, requirements are further increased due to seismic loading and the finally required joint widths are significantly large, even ten times larger than the serviceability demand (i.e. EC8 specifications). A reason for the wide use of joints has been the idea that is more convenient to let the structures expand rather than resist the increased demand produced by the imposed serviceability deformations (i.e. temperature, shrinkage) in integral structures. The wide use of joints has also been based partly on the difficulties of traditional design in analyzing large undetermined structures, as nonlinearities have to be considered. Furthermore, the presence of joints allows the use of various construction techniques, i.e. the efficient use of precast members. However, experience of construction practice has shown that joints in structures arise even more issues than those that they accommodate. The presence of joints undermines durability of structures with problems such as moisture and corrosion or additional deformations induced by faulty joints. They also

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arise aesthetic-design issues, since they need double supports at their ends. In addition, their cost (construction and maintenance) is not negligible to that total lifetime cost of structures (Wanninger, R.; Brinsa 2006). Therefore, joints can be characterized as uneconomical design solutions, especially, in seismic prone areas, where seismic demand increases significantly joint widths due to large seismic relative displacements.

These concerns have motivated a turn to the construction of jointless, also called integral, structures to accommodate the problems that arise from the use of joints. Integral structures are expected to have a long lifetime without additional maintenance costs. The elimination of joints in combination with efficient serviceability and seismic response is considered as an optimal solution. These structures provide advanced level of safety as undetermined structures and have improved seismic performance under strong excitations due to the reduced impact possibility between their members in contrast to structures with joints, i.e. simply supported bridges (Muthukumar and DesRoches 2006). Fully integral structures have been extensively in the last century. However, when complex structural issues arise, the use of joints is the common construction practice. From mid-20th century there have been some efforts to avoid joints but mainly in bridge design practice. In the last years, the advancements in technology and in numerical analysis for undetermined structures has led to successful construction of a large number of jointless structures, i.e. Kaltern swimming center in Germany (Fig. 1), that have satisfactory behavior against design requirements. There have been some publications also for jointless or with limited joints structures (Glimm, M.; Quast 2003; Rapolder 2004) but they are mainly empirical.

![Kaltern swimming center, integral structure](image)

For optimal joint elimination solutions various phenomena have to be considered. Towards this direction, there have been some research efforts regarding the response of structures under induced restraints. Hock (1983) studied the origins and size of the imposed deformations on structures showing the importance of column capacity and deformability under large horizontal slab displacements, while Wohlfahrt et al (1986) tested columns under restrained deformations. Noakowski et al (2003) studied the relation of column stiffness and crack growth to shear forces developed in columns under imposed restraints. Knowledge on crack growth mechanism (Carpinteri 2012) and nonlinear response of reinforced concrete members (Chen 2007) are necessary to understand the nature of these phenomena. Other researchers, like Caldentey et al (Caldentey et al. 2012) and Groli et al (Groli et al. 2014), have focused on experimental and analytical investigations on the improvement of the design methods used for columns of long jointless structures, which have been identified as crucial structural members in terms of serviceability requirements, concluding that current Codes are quite conservative considering the limit lengths for jointless structures. However, these research studies lack any suggestions to improve the response of columns to address the increased demand of serviceability limitations and the need for intermediate joints.

On the light of the above, a method that contributes to the development of jointless structures, as an optimal construction solution, is proposed herein. The target is to control column stiffness and increase flexibility which is the dominant factor that influences the size of the imposed stresses and deformations on columns by the imposed in-service and seismic loading. Stiffness is controlled with a novel type of cross-section for the outer corner building columns which has the advantage of flexibility and consists of a bundle of columns, symmetrically arranged, statically independent, circular cross-sections in contact with each other. The cross-section has the characteristic of multiple smaller cross sections replacing a larger solid one. A similar
approach has been proposed by Tegos and Pilitsis (2014) for bridge piers to increase bridge flexibility without the use of any isolation devices. The proposed method is applied in a 15-story building with 80mx20m layout dimensions and seismic and in-service performance is investigated. The studied building, is a limited local joint case with non-integral connections between the outer columns and slabs at the ground floor level. Despite the fact that extensive expansion joints through the building layout are avoided, the optimal solution of a complete elimination of joints is sought herein. The analysis results indicate the improved structural safety with the accommodation of seismic demand and serviceability restraints.

2. NOVEL BUNDLE OF COLUMNS FOR STIFFNESS CONTROL

The proposed herein column cross-section solution originates from the need to develop jointless buildings as described in Introduction. The concept originates from a previous research work by Tegos and Pilitsis (2014) for the reduction of pier stiffness. Since column stiffness influences significantly the effect of the imposed deformations in long buildings, the present study proposes a way to control it and increase flexibility of columns. The proposed system aims to alter the flexural stiffness of columns, eliminate the need of joints and address in-service and seismic demand. In typical building structural systems, the ground floor beams are the most vulnerable structural components to serviceability deformations and, in particular, the demand on columns increases as their distance from the center increases as well. Therefore, the proposed column design approach discussed herein is considered to be applied at the ground floor level of long buildings. The proposed column approach includes the following design aspects: Smaller circular cross-sections, equal in diameter, replace the large solid cross section of the perimeter building columns. The schematic representation of the proposed configuration is illustrated in Fig. 2. In this way the massive solid columns are fragmentized and have increased flexibility to receive the serviceability and seismic demand. The smaller cross sections have lower stiffness that can accommodate the drift resulting from the expansion and compaction of the first floor slab due to serviceability demand. The columns are replaced only in the ground floor, while in the upper floors the initial column cross section is used. The continuity between the lower and upper floor columns is achieved through the height of the concrete slab between them. In such buildings, it is often to have the slabs with large height which provides adequate space for the appropriate anchorage of the longitudinal column reinforcement.

The proposed column type is easy to construct, as it is possible to use cardboard disposable formworks that are widely available and of low cost in current construction practice. Aesthetics are also taken into account, as the new column configuration has architectural characteristics that do not compromise aesthetics.

![Diagram of proposed column configurations](image)

**Figure 2.** Schematic plan of the proposed column for (a) rectangular solid cross section, (b) circular column cross section

Regarding the structural response of the smaller circular columns buckling issues shall be taken into account. Therefore, a criterion is determined regarding the minimum column dimensions. The criterion relates the
Euler buckling load to the upper bound of the normalized axial force due to ductility demand. In common building structures, the normalized axial load $v_d$ shall be smaller or equal in absolute values to 0.65 (CEN[Comité Européen de Normalisation] 2003). Eq. (1) and (2) show the derivation of the formula of the minimum critical diameter of each column cross section with respect to the upper bound of the axial force.

$$P_{cr} = \frac{\pi^2 E_c I_c}{k H^2}$$

$$v_d = 0.65 \geq \frac{P_{cr}}{A_c f_{ck}} \Rightarrow 0.65 \geq \frac{\pi^2 E_c I_c}{k H^2 (\pi d_{cr}^2 / 4) f_{ck}} \Rightarrow$$

$$\Rightarrow d_{cr} \geq \sqrt[22]{\frac{4\pi^2 E_c I_c}{0.65 k H^2 \pi f_{ck}}}$$

Where, $E_c$ is modulus of elasticity of column, $I_c$ column moment of inertia, $H$ is the column height and $k$ factor depending on fixity conditions, $A_c$ is the column cross section, $f_{ck}$ is the concrete strength, $d_{cr}$ is the column diameter.

Furthermore, P-Delta effects have to be considered in the selection of the appropriate column cross sections and column reinforcement is calculated from critical seismic and non-seismic load combinations.

3. CASE STUDY

The building used as the benchmark structure in the case study is a 15 – storey concrete building with one basement and a plan layout of 79m*19.5m. The building is a typical example of buildings with long dimensions. It consists of a basement, a ground floor that is 4.5m high and 14 typical floors that are 3.2m high. In Fig. 3 the structural layout of the typical floors is illustrated. The building is plan symmetric in the Y-direction, while the staircase and elevators break the symmetry in the X-direction.

According to common practice, expansion joints are required for lengths beyond 30-35m. In particular, in this building due to the presence of the concrete core in the middle a single joint is not feasible and two expansion joints are required. However, the building was studied with a limited local joint solution at the column-slab connections to avoid the need of large joint widths due to seismic demand and the need of double elements at joint locations.

There are circular columns with a diameter equal to 90cm around the perimeter, 4 shear walls at the perimeter and a central core of shear walls. A characteristic of the system due to operational needs is the absence of inner columns that result in span lengths equal to 18m and the absence of beams. The concrete slab (Fig. 3(b)) has a total height of 60cm, is sandwich-voided filled with polystyrene for lowering the self-weight of the slab and is prestressed due to the increased demand for controlling deflection, tension stresses and to avoid cracking. The tendons have parabolic geometry and consist of 5 strands. The foundation is a concrete slab of 1m height and prestressed as well. Further information on dimensions of the structural elements and their reinforcement detailing can be found in Fig. 3(b-d).

The concrete strength is $f_{ck}=35$MPa and steel strength $f_{yk}=500$MPa. The tendons are St1670/1860. The building is located in a moderate seismic hazard region, seismic zone II – 0.24g, and soil class is B (CEN[Comité Européen de Normalisation] 2003).

In the studied building for the accommodation of the increased serviceability demand two strategies were employed: a) the first strategy includes the division of the cross – section of the outer concrete shear walls at the ground floor level with polystyrene to 2 equal smaller cross sections, as shown in Fig. 4 (a). The initial width of the shear wall is 50cm and is divided to two 25cm wide cross sections. This separation results in lowering the shear wall stiffness in the critical (longitudinal) direction of the development serviceability deformations which lowers the resistance to deformations, as well, without reducing the stiffness in the transverse direction. b) the second strategy involved the introduction of joints, 2cm wide, at the connections of the 4 corner columns of the ground floor with the 1st floor slab and the extension of the column to slab support with a height of 40cm, as shown in Fig. 4(b). As the serviceability demand is lowered from the layout ends to the middle of the structure, the width of local joints is reduced as well.
Figure 3. Benchmark Structure (a) typical floor plan, (b) concrete slab detail, (c) typical column detail, (d) concrete core (upper part) detail, (e) foundation slab
These column to slab joints are located at all column positions that exceed 15m distance from the middle at both sides of the building. As an alternative, elastomeric bearings could be introduced at the support locations of the slab to the columns but such an investigation is out of the scope of the present study. The joints are introduced to avoid the development of cracking in the slabs due to serviceability deformations. Although this solution includes only limited joints and avoids large widths through the whole layout of the building, it still introduces a limited disconnection between columns and slabs, which does not address the issues that non-integral connections arise. Regarding the seismic response of the building in the benchmark (studied) solution, it should be taken into account that since the introduced joints for serviceability have small width they will close very quickly under seismic excitation which induces larger deformation than serviceability demand. Therefore, after the closure of the gap at the joints, the columns will be activated and will contribute to the seismic resistance of the building. Furthermore, the development of friction forces at the slab column support restrain the movement induced by lateral loads. However, as it will be shown in the analytical investigation, the participation of the columns to the total seismic resistance is very low since the concrete core and the shear walls are the dominant structural components in the seismic response.

The application of the proposed column solution is an attempt to optimize the construction solution of such buildings with the elimination all types (local or extended) of joints. The proposed solution aims to provide fully integral connections between columns - slabs where the local joints are located; at the ground floor columns that have a more than 15m distance from the middle of the layout, Fig. 5, to address the increased serviceability demand of the ground floor.

![Figure 4](image_url)

Figure 4. Vertical concrete members in benchmark structure (a) Ground floor corner concrete shear walls (b) Column-Slab joint detail

4. **ANALYTICAL 3D MODELS**

A 3-D non-linear model was created in SAP2000 (Computers and Structures Inc 1998) for the benchmark building and is shown in Fig. 6(a). The concrete system behavior is affected by other structural components, such as the infills, but the effects of such components on the structural response are not studied in this paper. Herein the main objective is the proposition of an innovative concrete column configuration. The columns were modeled as frame elements with concentrated plasticity. The moment-curvature curves were derived...
from cross-section analysis in the free software AnySection (Papanikolaou 2015) based on the dimensions and reinforcement of the circular solid columns. The concrete shear walls and cores were modeled with frame elements. Rigid elements are introduced at the top and bottom of the shear walls at each floor. The floor slab was modeled with equivalent I-shaped beams, $b_{eff}=1.2m$, $h=0.6m$, and shell elements in the perimetric full-concrete zones. The tendons were modeled with tendon elements with the respective dimensions, geometry and values for losses according to Eurocode 2 (CEN [Comité Européen de Normalisation] 2002). The foundation slab was modeled accordingly, with equivalent I-shaped beams, $b_{eff}=6m$, and shell elements in the perimetric full-concrete zones and the basement concrete shear walls. The soil was modeled with equivalent linear springs with $k_s=20000kN/m$. P-Delta effects were taken into account in the analysis.

![Local Joints around ground floor columns](image1)

![Local Joints around ground floor columns](image2)

Figure 5. Location of local joints

![Benchmark building 3d model](image3)

![Benchmark building 3d model](image4)

![Benchmark building 3d model](image5)

Figure 6. Benchmark building 3d model (a) finite element model, (b) frame elements, (c) shell elements

The 3-d model of the proposed solution has the same properties with the finite element of the benchmark model with different column cross sections, as shown in Fig.7. After a crack development analysis according to (I. Tegos, Giannakas, and Chrysanidis 2011) and from Eq. 2, the critical (minimum) column diameter is derived, $d_r=0.30m$. It is found that such a diameter can accommodate up to 5cm of column displacement. Longitudinal reinforcement ratio for the small 30cm diameter columns is assumed the minimum, $\rho_l=1\%$, which can be checked for all critical load combinations according to Eurocode 8 (CEN [Comité Européen de...
Normalisation) 2005). In Fig. 7, the column characteristics and the respective modeling are presented. Mass is assigned to the building 3d-models for seismic analysis. The mass of each floor is calculated based on the values of the permanent and part of the live loads. The total values for the characteristic floors are presented in Table 1. The input ground motions used for seismic analysis were seven ground acceleration records matching EC8 spectrum derived from Rexel (Iervolino, Galasso, and Cosenza 2009), Fig. 8.

![Figure 7. Modified building 3D Model](image)

Table 1. Mass distribution.

<table>
<thead>
<tr>
<th>Floor</th>
<th>G+0.3Q (kN)</th>
<th>m(tn)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>22683.93</td>
<td>2312.33</td>
</tr>
<tr>
<td>Typical Floor</td>
<td>22331.45</td>
<td>2276.40</td>
</tr>
<tr>
<td>Ground Floor</td>
<td>23111.45</td>
<td>2355.91</td>
</tr>
</tbody>
</table>

![Figure 8. Accelerograms matching code spectrum for seismic zone II](image)

5. ANALYSIS AND RESULTS DISCUSSION

The analytical investigation included analyses of the two example structures, the benchmark and the modified with the column bundle for the two critical conditions serviceability and seismic demand.
Serviceability is crucial for verifying the efficiency of the proposed system in controlling serviceability deformations with the absence of joints and seismic analyses is performed to verify that the changes introduced in the building structural system are not influencing negatively the seismic response of the structure.

The first analysis group was conducted for serviceability requirements. Regarding serviceability demand equivalent temperature loads of -50°C were applied to account for thermal contraction, creep and shrinkage which are the most critical in-service deformations. In the current regulations -25°C is suggested as the appropriate temperature for contraction (CEN [Comité Européen de Normalisation] 2003) while an additional 25°C temperature (Rhodes and Carreira 1997) can be used as a simplification to account for creep and shrinkage. As shown in Fig. 9 the largest deformation is observed at the outer columns of the longitudinal direction of the ground floor which is in accordance with the selection of the application of the proposed system at these locations. More specifically, in Fig. 9b, it is obvious that the increased demand is developed only in the ground floor columns while the upper floors deform in the same way and the resulting drift is remarkably small.

![Figure 9. Deformed Shape benchmark structure a) plan view, b) elevation](image)

Serviceability induced restraints were also investigated in the modified structural system with the smaller column cross sections. In Fig. 10, the comparative drift results for both structures (benchmark and modified)
are illustrated. The maximum displacement value resulting from serviceability demand for the benchmark is smaller than the width of the joints at the column to slab connections (1.7 cm). In the modified structural system, the serviceability induced cracking due to the integral (monolithic) column to slab connection is calculated based on the diagrams provided in Tegos, Giannakas, and Chrysanidis (2011). Crack widths are found lower than 0.2 mm which is an accepted value according to the allowable cracking level provided by EC2 (CEN [Comité Européen de Normalisation] 2002).

The next analysis group includes the investigation of the structural building response under seismic analysis. It is noted that the aim of the analysis under lateral seismic loads is the verification that the proposed method has efficient seismic response. The investigation is driven by the concept that serviceability and seismicity have opposite requirements and often interventions that are beneficial for the one have negative effects on the other.

In Fig. 11, the seismic drift for each floor for the benchmark and the modified building are illustrated and compared to drift limits provided by Eurocode 8 - Part 1 (CEN [Comité Européen de Normalisation] 2003). Although there are slightly larger drifts in the modified building, the resulting values are below the maximum value limit.

In Fig. 12 the ratio of the contribution of the shear walls and the columns in the seismic base shear is presented. It is observed that the differences between the two systems are very low. The modified system affects slightly the response in terms of forces and deformations, as well. This behavior is expected since the core and the shear walls contribute the most to the building seismic resistance. The small difference in the participation of the columns is also attributed to the fact that even in the case of the benchmark bridge with the limited joints; the columns are activated under seismic excitation as described in the previous paragraphs.
6. CONCLUSIONS

In the present study a novel column configuration is investigated for buildings with long layouts to avoid joints required in these structures. The description of the proposed solution and the comparative evaluation of both serviceability and seismic analyses can be summarized in the following conclusions:

- Smaller circular cross-sections, equal in diameter, replace the large solid cross section of the perimeter building columns. The proposed column bundles have smaller stiffness than the initial solid circular cross sections resulting in higher flexibility.
- Constructability of the suggested columns is a manageable issue since disposable cardboards can be applied. On the other hand, aesthetics may require further discussion. However, in the authors’ opinion, aesthetics is not harmed and, the column configuration can be used as an architectural feature.
- The proposed solution has been proven to efficiently accommodate both serviceability and seismic demand. The columns are replaced only in the ground floor, while in the upper floors the initial column cross section is used. Resulting top ground floor displacements from serviceability requirements are lower than the limit values for cracking development and seismic drift values are below EC8 limits. As expected, the modified system doesn’t compromise the structural seismic response in terms of forces and deformations, as well, since the concrete shear walls have the largest contribution to the seismic resistance.
- The proposed solution achieves absolute monolithic connections and avoids the introduction of joints, even the limited joints provided in the studied building which are required due to the large length of the longitudinal x direction. This elimination of joints addresses several issues that their presence arises, such as high construction and maintenance costs.

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8. REFERENCES

