Numerical and experimental assessment of the soil-structure interaction effect of a building in the Brazilian central area

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ABSTRACT
The study of the soil-structure interaction (SSI) in buildings is commonly neglected in superstructure calculations, in other words, the supports, or the foundations, in such structural programs are considered fixed, leading to a rigidity equal to infinity. Therefore, the soil-structure interaction in building design has been the objective of a thesis from the University of Brasília, to be summarized in the present paper. The paper describes the results and problems during the experimental works and related numerical analyses of the structure. The gained experience and knowledge is of value to those interested in calculating the structural behaviour of buildings with the proper incorporation of soil-foundation interaction & geotechnical 3D foundation effects.

PRESENTACIONES TÉCNICAS
La investigación de la interacción suelo–estructura (SSI) en edificaciones es normalmente despreciada en cálculos de superestructura, en otras palabras, los apoyos, o las cimentaciones, en estos programas estructurales son considerados empotrados, causando una rigidez igual al infinito. Por lo tanto, el uso de la interacción suelo–estructura en el diseño de edificios fue el principal objetivo de una tesis de la Universidad de Brasília, a ser resumida en este artículo. La experiencia y conocimiento ganados aquí son de gran valía aquellos interesados en calcular el comportamiento estructural de edificios con la correcta incorporación de la interacción suelo-cimentación & el efecto geotécnico 3D de las fundaciones.

1 INTRODUCTION

The study of the soil-structure interaction (SSI) in buildings is commonly neglected in superstructure calculations, in other words, the supports, or the foundations, in such structural programs are considered fixed, leading to a rigidity equal to infinity. In general, the calculations of the structural internal stresses of parts of the building (beams and columns) are solely accomplished in this condition, which, as stated before, does not consider the existing deformability of the soil/foundation system.

Therefore, the soil-structure interaction in building design has been the objective of a defended (Soares 2004) and on-going (Cámbar 2011) D.Sc. thesis from the University of Brasília, to be summarized in the present paper. By and large, the original study was done in both numerical as well as experimental points of view, by combining results from column settlement and internal force measurements during the erection of a chosen building, with numerical uncoupled analyses. In the numerical point of view, several methods can be used to model the problem, and, among them, the most common software adopts the Finite Element Method (FEM) and the Boundary Element Method (BEM) to simulate the building behavior during its construction. In general, programs in the geotechnical area, as well as in the structural area, can be used with this purpose.

This work presents real data obtained in a pioneering manner for the region from an instrumented building from the Central Plateau of Brazil, more particularly located close to the Brazilian capital Brasília, in the heart of this nation. It shall be noticed that the pioneering aspect is related to the region of Brazil, and not to the whole country, as earlier work on this subject has already been successfully carried out by colleagues from other Brazilian institutions elsewhere (Gusmão 1994, Figueiredo & Lucena 2003, among others).

In the present case the experimental focus was directed towards the settlement and load measurements in columns of the structure and their measurement during its erection. Problems off course were encountered, and hindered somehow the interpretation of some of the experimental data. Field loading tests on piles of the same characteristics of those adopted in the building were carried out, in order to "back-analyze" initial geotechnical parameters for some of the numerical analyses. Actually, the soil-structure interaction was considered in an uncoupled manner, with a Brazilian structural program to simulate the erection of the structure (TQS) with and without flexibility of the foundation springs, and another well-known geotechnical software (PLAXIS-3D) was adopted to simulate the overall system also providing the flexibility factors to be used in the foundation springs of the structural, aforementioned, software.

Comparative results are presented in terms of vertical loads and bending moments in the columns elements, at distinct levels or floors of the studied edification, with and without soil-structure interaction (SSI) effects. That means, assuming normal structural design procedures (generally done in Brazil) and assuming a more realistically approach, which incorporates foundation spring flexibility or its displacement during the building construction. In the present study with this particular building it could be demonstrated that SSI effects can be of importance, especially at the lower pavement floors of the edification.

The paper therefore brings some experience in this technical field, for the Central Area of Brazil, in a typical tall and slender edification.

2 BUILDING & SITE CHARACTERISTICS

The Brazilian capital Brasília and its neighbouring areas (Federal District) are located in the Central Plateau of Brazil, as presented in Figure 1. This district has a total area of 5814 km^2 and is limited in the north by the 15°30' parallel and in the south by the 16°03' parallel. The city of Brasília is portrayed in this same figure by an “airplane” shape like form.

The investigated area is located near Brasília, in a district called “Taguatinga” (see Figure 1). In this district there are several areas under intensive expansion and urban occupation, attracting people and investments from all over the Federal District. New shopping malls, access highways and hotels are under construction as, for instance, the new apart-hotel building which analyses will be described herein.

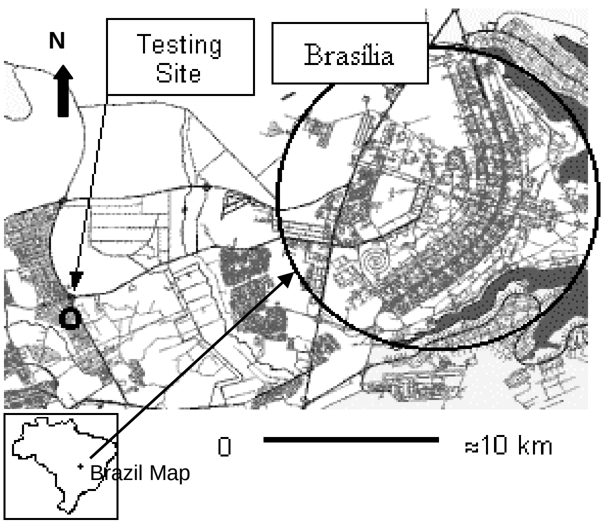

Figure 1. Building location close to Brasília City

Within the Federal District extensive areas (more than 80 % of the total) are covered by a weathered latosoil of the tertiary-quaternary age (Cunha et al. 1999). This latosoil has been extensively subjected to a laterization process and it presents a variable thickness throughout the District, varying from few centimetres to layers above 40m of thickness. There is a predominance of the clay mineral caulinite, oxides and hydroxides of iron and aluminum. In Taguatinga, the top latosoil overlays a saprolitic/residual soil, with a strong anisotropic mechanical behaviour and high standard penetration resistance (N_{SPT}), which originated from a weathered, foliate slate, the typical parent rock of this region.

Figure 2 presents a typical geotechnical profile of the investigated site. As noticed, it is composed by a superficial layer of silty clay (2 to 3m) followed by a medium compacted layer (2m) of clayey sand over a thick (10 to 12m) layer of compacted sandy silt. The last layer overlies the slate bedrock, where the foundation tips are founded.

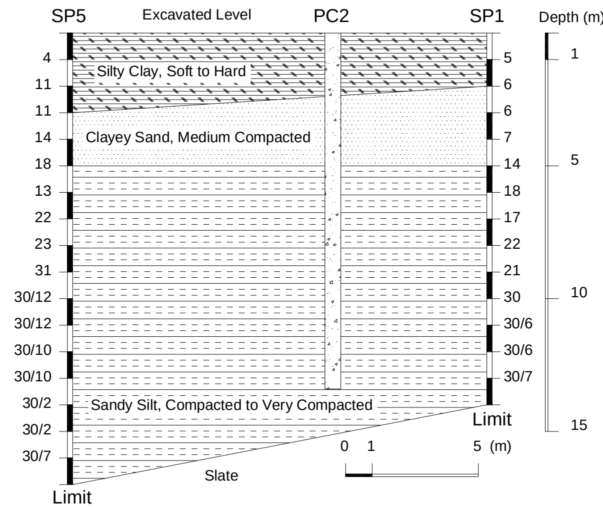

Figure 2. Typical geotechnical profile of the site

Bored with bentonite mud, uncased, molded in situ, large diameter (varying from 30 to 150 cm) foundations were specified for this site, with lengths that varied from 12 to 23 m, socketed in the bedrock. The foundations are isolated or packed in groups of 2 to 4 piles, which details are shown in Cunha et al. (2003).

The commercial building analyzed herein is schematically presented in Figure 3. It contains 2 undergrounds, one ground level and mezzanine, nine “typical” floors (1st to 9th floors), one penthouse and one attic, totalizing 15 floors.

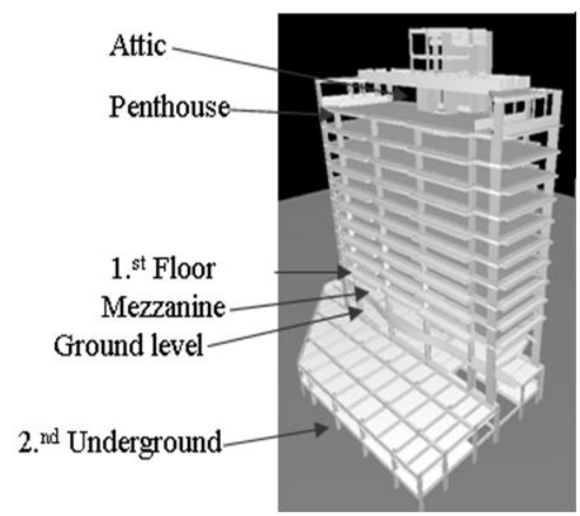

Figure 3. General view and aspects of the building

Construction initialized in 2001 and finished in 2003. In the end, the building was constructed with 88 columns, being 29 from the central projection (which extends up to the attic) and 59 from the underground garages. Vertical design loads (majored by 20%) varied from 420 to 13440 kN in the main columns. These loads were calculated with TQS "standard" (fixed springs) analyses, which were adopted in the common (commercial) building design.

The present series of analyses and comparisons did not consider the extra 20% overload, only the "correct" self-weight of the building. It also considered as 100% construction stage as the completion of all structural works, until the end of the attic – just before starting constructing walls and floors. The instrumentation in terms of load readings was carried out in columns of the 5th floor, just after its construction. Around this same time settlement readings started to take place on columns of the 2nd underground (details in Soares 2004).

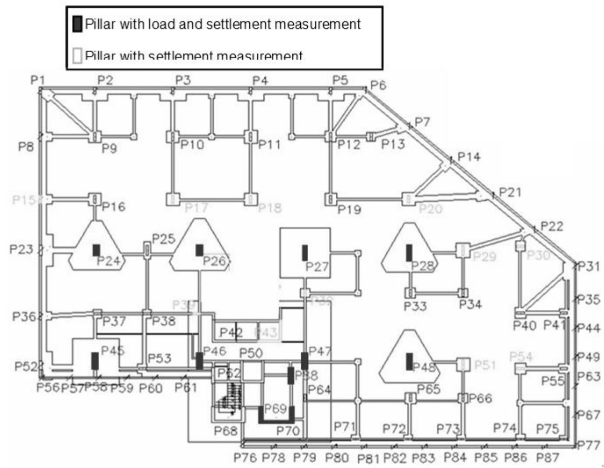

Figure 4. Instrumented columns of the building

3 BUILDING INSTRUMENTATION

The instrumentation of the columns of the 5th floor was carried out with ~1 m length instrumented bars wired with 4 strain gauges arranged in a complete Wheastone bridge. Ten columns of this floor were instrumented with one bar each, installed just before concreting.

Hence, in each instrumented column there were left four wires, protected with tapes, to be connected to a standard electronic reading device after the completion of each of the subsequent floors (6th to 15th). The instrumented bars were prepared by technicians of the University of Brasília. The same instrumented columns of the 5th floor (which are the main ones of this building) were chosen to be monitored for settlement at the 2nd underground level, with exception of columns 45 and 46 (given the fact that a wall had already been constructed around these columns). News columns from this lower level were also chosen to be monitored, leading to a total of 19 monitored points. In these points, a commercial standard metal pin was installed at ground level, just few decimetres above the foundation top cap.

The configuration of the columns monitored for load and settlement (filled rectangles) and just for settlement (unfilled) is depicted in Figure 4.

Readings were taken by using a topography laser type total station, with accuracy of 1/100 mm. The readings were compared to a reference level outside the construction area, composed of a 30 cm diameter pile with 5 m in length. Inside this pile a 16 mm diameter steel bar, with spherical top, was inserted as benchmark. Four readings were taken till the completion of the structural work (100% construction stage). As stated before, readings started just after instrumentation of the 5th floor columns, which is equivalent to, around, 50% of the construction stage of the building.

4 NUMERICAL UNCOUPLED ANALYSES

As presented before, an uncoupled soil-structure interaction analysis was carried out, by using a structural software (TQS) to simulate the structure and a geotechnical one (Plaxis-3D) to represent the foundation/soil system. The overall view of the analyses is shown through the flowchart of Figure 5.

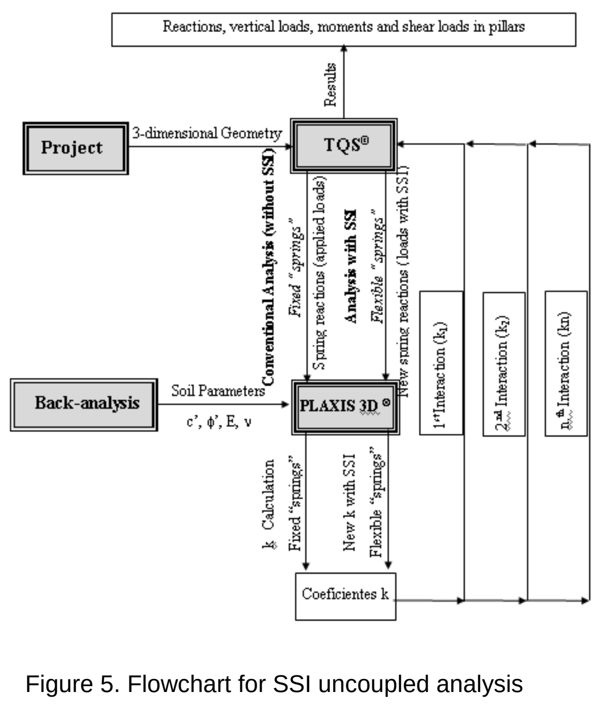

Figure 5. Flowchart for SSI uncoupled analysis

Using Figure 5 as reference, the analysis is initiated by gathering all the information of the project (3D structural arrangement, loads, connections, etc.) to be inputted into the TQS software. Initially this program is run by considering infinitely rigid foundation bases (each foundation under a column is simulated by a fixed spring). With this first analysis it is possible to obtain the structural reactions for all building elements, as column loads and moments. With the calculated loads of the underground columns, at the level of the springs, a second analysis is run – this time with Plaxis-3D software.

This latter program adopts the geometry and structural arrangement of the pile foundations, and uses geotechnical parameters from back-analyzed pile load tests in order to represent the soil behaviour. A simple elasto-plastic model with Mohr-Coulomb failure criteria was adopted for the soil, which was discretized with solid 3D elements of 15 points. For the whole foundation 11264 elements were used, as schematically detailed in Figure 6. The structural elements of the foundations were simply simulated as linear elastic. Given space constraints, details of this particular geotechnical analysis are not presented herein, but could be consulted in Soares (2004).

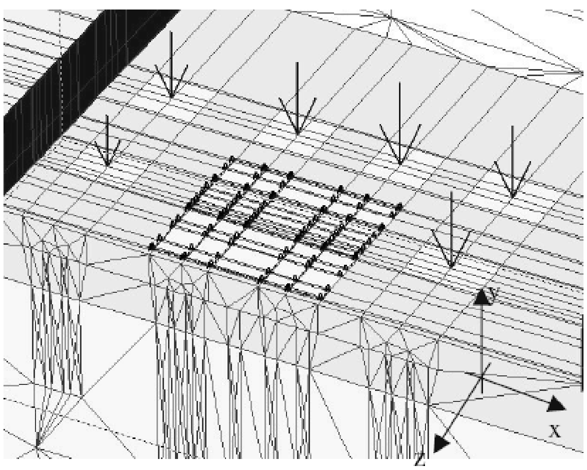

Figure 6. Details of the 3D adopted mesh

With the initial analysis from Plaxis-3D it was possible to obtain the first series of estimated settlements of the foundation groups and related columns. This outcome allowed the determination of the first series of spring stiffnesses (k_1 = load from fixed spring of TQS divided by column settlement of Plaxis-3D), related to the "1st interaction". These stiffnesses were reintroduced in TQS for a second, now denominated "with SSI", structural analysis. New (flexible) spring reactions were obtained and, again, Plaxis-3D was used to obtain settlements.

A second series of stiffnesses (k_2 = same aforementioned equation) was obtained and compared to k_1, in order to see the differences and to determine if a new round of analyses ("2nd interaction") would be necessary. Since the average difference was less than 1% ($100^*(k_2-k_1)/k_1$) for the main columns of Figure 4, the overall procedure was considered completed, and the results in terms of loads and moments for the main elements were compared in both conditions of without (initial TQS analysis) and with SSI effects.

Based on these series of analyses, results were obtained, compared and discussed, as will be shown next.

5 COMPARISON OF RESULTS

5.1 Comparison of measured and calculated loads

- The comparison of measured versus calculated values was done in terms of the vertical loads of the main columns of this structure, in the 5th floor, and in terms of underground column settlements. The calculated values adopted herein were those from the first run of TQS software, i.e., considering standard structural analysis with fixed spring support (without SSI). The results are valid for the loads measured at completion of each floor, after (and including) the 6th one – till the 15th floor. For the sake of space only the results up to the 12th floor are shown, given the fact that a similar trend was noticed beyond the 10th floor. Results from this comparison are shown in Figure 7.

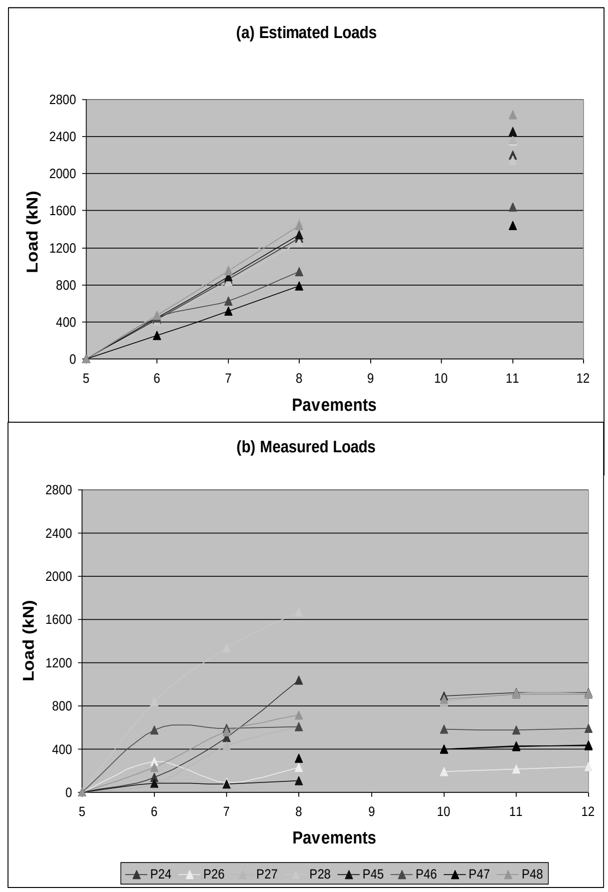

Figure 7. Comparison of column loads

From Figure 7 some observations can be drawn:

- There is a reasonable qualitative agreement between measured versus estimated loads, although, in quantitative terms the estimated values are a bit higher than the measured ones. Indeed, when comparing the estimated with SSI results (not shown), there is a tendency to decrease the differences. Nevertheless, even in this latter case the estimated values are higher than the measured ones;
- There is no agreement, whatsoever, for loads after construction of the 10th floor. After (and including) this case, the estimated loads are considerably higher than the measured ones, and the "linear" tendency of load increase, from the 5th floor on, is not observed anymore. Actually, the measured loads tend to become constant in a flat tendency after this stage.

Aforementioned observations have, off course, an explanation, as will be delineated next. Initially it can be said that a good agreement of measured versus calculated loads, up to the 8^{th} floor, is a nice indication that the measuring system and site procedures have worked out. The almost "linear" load increase is expected, given the nature and constancy of loading at each load step (i.e., just placing similar amount of slabs, beams and columns at each "typical" floor). Indeed this aspect has been captured both in experimental and numerical terms. On the other hand, the difference of the results can be accounted to several factors, as the simplifications and (unknown) built in errors of the numerical analyses and all the experimental problems inherent to the *in situ* measurement of column loads, as concrete creep and temperature changes, change of "zero" reference values from one measurement to another (where the measuring unit has to be reconnected), unexpected geometric changes of the structure, etc. All these aspects have somehow been observed in another related university work (Almeida & Almeida 2000) of this same theme. The fact of having closer (measured x calculated) results with the numerical (with SSI) analysis also indicates that, indeed, soil-structure interaction has influenced to some extent the behaviour the structure.

The other aspect to be mentioned is the total lack of coherence, in experimental terms, after the construction of the 10^{th} floor. Note that, in numerical terms, load continues to "linearly" built up after the 8^{th} floor. This behaviour is not accomplished by the measurements and has been explained, by Soares (2004), to be primarily related to a malfunction of the electronic reading device rather than all other aforementioned experimental problems. It seems that it has accidentally "crashed" in one of the trips to the building site, but continued to work. This problem has unfortunately passed unnoticed by the fact that data reduction and interpretation only started after all the measurements were taken (after the 15^{th} floor) – as usually done by D.Sc. students...

5.2 Comparison of measured vs. calculated settlements

The comparison of measured versus estimated settlements is more complex, and will be briefly explained here, although details can be again found in Soares (2004).

The calculation of the settlement of the underground (main) columns of the structure, relative to the measured points shown in Figure 4, was carried out with Plaxis-3D software, adopting TQS loads which were obtained after the second series of analyses (with SSI). Plaxis-3D settlements, obtained in this manner, were subsequently used to define spring stiffnesses k_2, mentioned before.

Given the fact that settlement measurement started only after completion of the 5^{th} floor, around 50% of the construction stage of the building, only the difference of settlement values, from the last run (100%) to the first reading (50%), could be used. Therefore, Figure 8 presents the comparison of measured versus estimated values in which estimated Plaxis-3D results refer to the settlement difference (100-50%), with values respectively calculated in each of these stages.

The agreement of results is considered satisfactory, in engineering terms, and denotes that some differences can indeed be found given all the simplifications (numerical analyses) and local *on site* difficulties (experimental results) commonly found in this type of work. It is noticed that the displacement of the building, after 50% construction stage and up to 100% (hence, before laying down walls, floors and finishing), was in order of ≈2 mm. Considering the previous stages (0 to 50% construction stages, not shown), the average total settlement of the main columns of this building, at underground level and 100% construction stage, would go up to around 13 mm – acceptable value for this type of edification and foundation system.

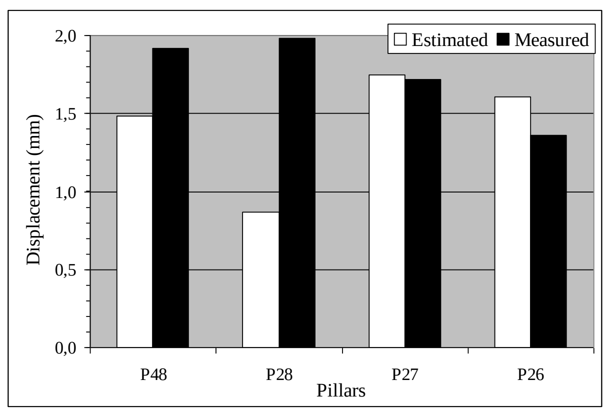

Figure 8. Comparison of underground settlements

5.3 Comparison between calculated results with and without soil-structure interaction

The comparison of measured TQS calculated values, with and without SSI, is initially shown in terms of the vertical loads of the (main) underground columns at 100% construction stage. These columns are depicted in Figure 4 and the results are shown in Figure 9.

As it is noticed in Figure 9, the loads with and without SSI are quite similar, indicating that the soil-structure effect was not high enough, in this case, to yield appreciably different values.

This result is undoubtedly linked to the fact that the level of displacements of this building was reasonably low (within normal design values for the region), thus indicating that, under such circumstances, SSI effects in terms of vertical column load variations are not of much importance. Besides, one also notices that SSI effects can increase or decrease the loads in comparison to the restrained (without SSI) case. This is invariably related to column position, and the post SSI configuration of displacements.

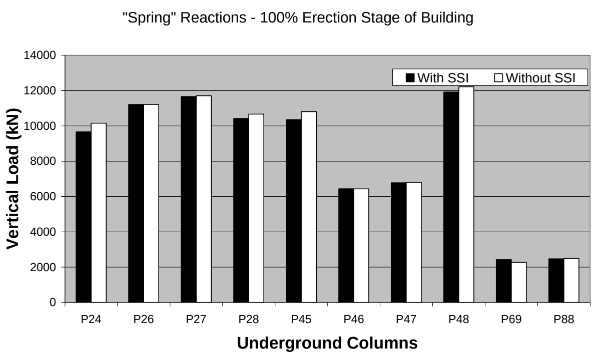

Figure 9. Vertical column loads with and without SSI

For instance, for column 26, reasonably well located in the central area of the projection (see Figure 4), there was almost no variation in vertical load due to SSI effects at all stages of construction, when compared to respective values of the restrained case. This result is clearly seen in Figure 10 with the distinct percentages of construction stage.

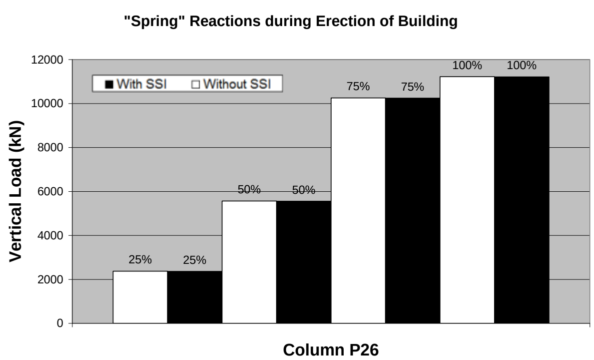

Figure 10. Vertical load of column 26 at distinct stages

Nevertheless, when comparing in terms of the column moments that were mobilized at distinct pavement levels (floors) at the end of the construction, SSI effects can be of noticeable influence. Figures 11 and 12 show the moments that were mobilized at each level of column 24 when considering, or not, the SSI effect. They respectively relate to moments in the y and x directions of the building.

It is noticed that the influence of the SSI effect, i.e., difference in values when considering or not unrestrained conditions for the springs at the base of the edification, also depend on pavement level and moment direction. These two variables are intrinsically related to the rigidity or spatial inertia of the structure, and, therefore, can vary from one type of edification to another depending on its geometrical conditions.

For the case studied herein, which resembles a normal tall commercial edification of the region with a pronounced slenderness (and concentration of load in its central projection core), SSI effects in terms of moments are apparently more pronounced at lower floor levels. It seems that, for moments in the x direction, SSI effects can be of importance up to the 4th. floor, whereas for the y direction this level changes to the 6th. floor. In both conditions, when considering the SSI effect there was a tendency of increasing the derived moments, in some cases to values as high as 150% of the restrained (without SSI) calculation case – see for instance M_y results at the 2nd. pavement floor for both conditions.

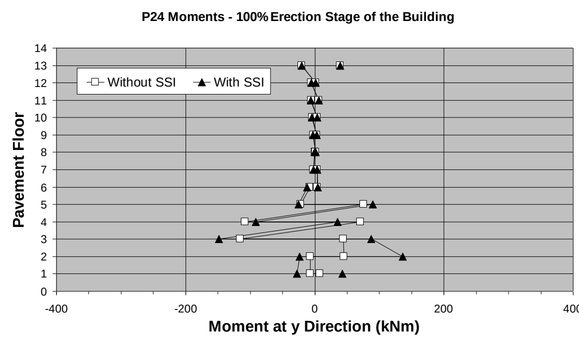

Figure 11. M_y values of column 24 at distinct levels

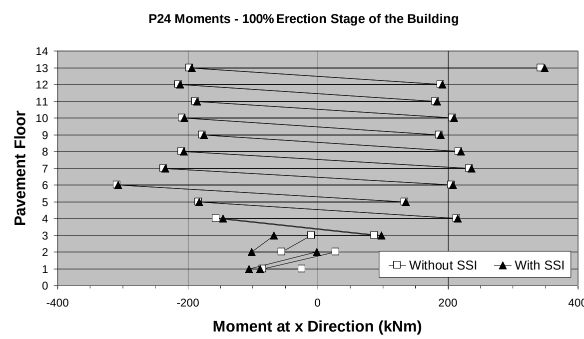

Figure 12. M_x values of column 24 at distinct levels

As a final remark one should understand that numerical uncoupled analyses do have deficiencies related by distinct operative (numerical) tools, and models, which are integrated in a cyclic manner to simulate the overall (real) behaviour. Nevertheless, given the simple and limited objectives of the study, which solely focused on direct comparisons of results with and without SSI, it can be affirmed that the main goal of the exercise was reasonably well addressed.

6 CONCLUSIONS

The present case study has demonstrated that SSI effects can be potentially important for the structural analysis of normal structures designed in the Brazilian Central Area and somewhere else. By considering an uncoupled numerical method of analysis, interaction and convergence, which adopted two well recognized programs respectively from the structural and geotechnical areas, it was possible to notice that:

- Soil-Structure Interaction effects can be of minor relevance in terms of the vertical (maximum) column loads calculated at the underground (base) level of the structure. Nevertheless, these loads can increase or decrease, in respect to the restrained (without SSI) case, depending on both column position and level of displacements of the building after uncoupled interaction;
- Soil-Structure Interaction effects can be of major relevance in terms of the moment loads mobilized in the columns of the edification, at each of the principal (x and y) design wind directions. These loads will increase negatively or positively, for each pavement floor, when calculating with SSI effect considerations. This is valid for both directions, although with distinct modules and percentages of variation;
- In the latter case, related to moment values at each direction, it was clearly noticed that the SSI effect was limited to a certain pavement floor of the edification, which, on the other hand, is inherently linked to the respective spatial inertia of the related direction (x or y). For instance, for the y direction, the SSI effects were limited to the 4^{th}. floor, whereas for the x direction this changed to the 6^{th}. floor. This is undoubtedly dependent on the geometric arrangement of the structure, the position of the structural element in regard to the whole arrangement, and the building's (flexible) behaviour in terms of the adopted foundation/soil system – among other factors (some unknown yet);
- Given all aforementioned items, it can be concluded that SSI effects should change somehow from one structure to another. Nevertheless, whenever possible it must be considered in the calculation of high-rises and sensitive or highly loaded buildings, if one wants to obtain design values that are closer to those that indeed take place on the structure during its working life.

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