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GEOMETRICALLY NON-LINEAR ANALYSIS OF MULTI-STOREY BUILDINGS SUPPORTED ON THE DEFORMABLE MASS

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Abstract. These studies compare results of settlements and efforts of multi-storey buildings supported by flexible shallow foundations. The soil is simulated by the Mindlin´s continuous numerical model and the Winkler´s discrete model. The building is modeled by finite elements using a static nonlinear geometric analysis. The building is connected to shallow foundations, which is simulated by shell finite elements, where membrane effects and for plate bending are considered. The soil representation follows two models: the Mindlin´s continuous model via boundary element method (BEM), and the Winkler´s discrete model. The insertion of the geometrically nonlinear formulation considers moderate rotations. These are considered to be sufficient for the building analysis. The building is evaluated by comparing the geometric linearity and non-linearity for several support conditions and the differences between efforts that appear especially in the first floors.

1 INTRODUCTION

The mechanical performance of a construction is governed by the interaction of the superstructure, the substructure, and the soil mass within a mechanism denominated of soil-structure interaction (SSI).

In the engineering practice, the interaction mechanism is usually ignored. As a consequence, structure and foundation projects are still developed independently from each other.

In general, when dealing with the building structure, the engineer considers undeformable supports to determine the response of the building and of the foundation. This set of reactions is given to the engineer in charge of defining the foundation, calculating settlements, and comparing these settlements with admissible values. He also compares the acting stress with the bearing capacity of the soil according to existing code requirements.

However, the effort distribution in the superstructure is not usually re-evaluated. As a consequence of the soil deformation, the tension flow in the superstructure is different from that originally calculated when considering the hypothesis of undeformable soil. It is expected that the change of efforts in the structure be either absorbed by the safety coefficient or small enough to cause no significant disturbance in the effort distribution of the building-foundation-soil system. These possibilities not always occur and the consequences can be highly undesirable.

Several SSI studies have already enriched the specialized literature. In particular, Meyerhof (1953), Chamecki (1956, 1958, 1969), Lopes and Gusmão (1991), Moura (1995), Reis (2000), Romanel et al. (2000), Romanel and Kundu (1990), Gusmão (1990), Antunes and Iwamoto (2000), Holanda Jr. (1998), Almeida and Paiva (2004) are considered references in that literature.

In general, the studies mentioned and others present restrictions regarding their applicability. Some do not allow the inclusion of flexible foundation elements, others consider only vertical actions in the building, and another group considers SSI but it ignores the foundation influence.

An additional phenomenon to be considered in the analysis of the soil structure interaction is the second-order effect that occurs in the 3D frames. This must not be ignored in slender buildings. Because of the importance and consequences in the effort redistribution of these buildings, this subject has received special attention in the NBR 8800 (2003) from the Brazilian Association of Technical Norms (ABNT). However, SSI when considering the geometric non-linearity of the building is still a field little explored inside the current literature.

In this context, the present studies develop a numeric SSI formulation that simulates the 3D building and the flexible shallow foundation by applying finite element method (FEM). It also simulates the semi-continuum media via boundary element method (BEM) and the Winkler’s model. Finally, it is considered the equilibrium of the building in the deformed position. The model also considers the geometric non-linearity phenomenon and takes an incremental-iterative procedure for the analysis of the building-foundation-soil complex.

2 THE BOUNDARY ELEMENT METHOD APPLIED TO ELASTIC INFINITE STRATUM

In the absence of volume forces, the Navier-Cauchy equations are given by:

$$u_{i,jj}(s) + \frac{1}{1-2\cdot\upsilon}\cdot u_{j,ji}(s) = 0, \quad i, j = 1,2,3 \tag{1}$$

Where $u_i(s)$ is the displacement in the *i* orthogonal direction from *s*, a point inside the solid that satisfies certain boundary conditions and ν is the Poisson's ratio.

These domain equations can be further expressed as surface equations, which are represented by the Somigliana Identity:

$$u_i(p) + \int_\Gamma p_{ij}^*(p,S)\cdot u_j(S)\partial\Gamma(S) = \int_\Gamma u_{ij}^*(p,S)\cdot p_j(S)\partial\Gamma(S) \tag{2}$$

Where *p* and *S* are the source point where the unit force is applied and the boundary point at the surface, respectively. Moreover, u_i and p_i are the real displacement field and the surface forces at the boundary point S in the i^{th} direction, respectively. u_{ij}^* and p_{ij}^* represent weighted field coefficients which indicate the response in the *j* direction at *S* to those forces applied in the *i* direction at the *p* point. This identity is based on Betti's reciprocal theorem. Fundamental solutions given by u_{ij}^* and p_{ij}^* represent particular solutions of the partial differential equations (1) for a given boundary condition.

The strategy to obtain boundary integral equations involves transforming *p*, which is inside the body, into *P* on the boundary. Thus equation (2) can be written as follows:

$$C_{ij}(P)\cdot u_j(P) + \int_\Gamma p_{ij}^*(P,S)\cdot u_j(S)\cdot\partial\Gamma(S) = \int_\Gamma u_{ij}^*(P,S)\cdot p_j(S)\cdot\partial\Gamma(S) \tag{3}$$

where the integral in equation (3) is defined in sense of Cauchy principal value (París and Cañas, 1997), and Cij are coefficients that depend on the problem geometry (Hartmann, 1980). The fundamental solutions used here are the known Mindlin's solutions presented by Mindlin (1936).

Since the analytical solutions of Expression (3) are not given in closed form, they have to be estimated numerically. Hence, BEM is based on the assemblage of a system of algebraic equations resulting from boundary integral equations, Equation (3), written in terms of nodal parameters that are approximated to boundary values by using shape functions. If the domain forces are not considered, the integral equations (3) are written as:

$$C_{ij}(P)\cdot u_j(P) + \sum_{k=1}^{NE}|J|\cdot\int_\Gamma p_{ij}^*(P,S)\cdot\Psi(S)\partial\xi(S)\cdot(U_i)^k = \sum_{k=1}^{NE}|J|\cdot\int_\Gamma u_{ij}^*(P,S)\cdot\Psi(S)\partial\xi(S)\cdot(P_i)^k \tag{4}$$

where NE, ψ, and J are the number of boundary elements, the shape function, and the Jacobian transformation, respectively. In this paper, linear shape functions of form $\Psi_i(\xi_1,\xi_2,\xi_3)=\xi_i$ are adopted. ξ_i are homogeneous coordinates defined for the flat triangular element, Brebbia and Dominguez (1989), in which the surface is discretized.

As the integrals of (4) cannot be solved analytically for any generic surface, they require the use of numerical techniques. The free surfaces are discretized in triangular boundary elements only in contact region with shallow foundations and linear shape functions are defined to represent displacements and traction fields associated with nodal points. Thus, it is possible to assemble shape matrices of the equation (4), which takes the following form:

$$[H]\cdot\{U\}=[G]\cdot\{P\} \tag{5}$$

Where the H and G matrices are defined as

$$H=C_{ki}(P)+\sum_{j=1}^{ne}|J|_j\cdot\int_{\Gamma}p^{*}_{ki}(P,S)\cdot\Psi(S)\;d\xi(S)\quad i,k=1,2,3 \tag{6}$$

$$G=\sum_{j=1}^{ne}|J|_j\cdot\int_{\Gamma}u^{*}_{ki}(P,S)\cdot\Psi(S)\;d\xi(S)\quad i,k=1,2,3 \tag{7}$$

The tractions indicated in equation (5) are prescribed or unknown values that can be associated with the foundation reactions which are coupled with the shallow foundation simulated by FEM. In this case, the expression (5) can be written as:

$$K_f\cdot U_f=P_f \tag{8}$$

The matrix K_f is the influence matrix indicates in eq. (6), P_f are the nodal foundation's reactions and U_f are the unknown nodal foundation's settlements.

3 GEOMETRICALLY NONLINEAR MODEL OF THE BUILDING

In the analysis of slender structures and in many applications, their components deform along the application of the actions in such a manner that the initial configuration changes and, as a consequence, influences and modifies significantly the final effects. Then, the static equilibrium must be considered in an incremental way. The problem is then led to a nonlinear force displacement relation. In other words, it does not follow the classic linear or in other way, the balance is evaluated in the deformed position. This kind of phenomenon is called geometrically nonlinear analysis (GNA).

In the GNA of tall buildings, a simplified GNA procedure can be used when displacement fields are not large and rotations are moderated. This is an approximated second-order procedure that is able to evaluate the coupling of axial and transversal displacements. Then, it is possible to use the Green-Lagrange deformation measures for the ε_x component of the axial deformation in the following way:

$$\varepsilon_x = \frac{\partial u}{\partial x} + \frac{1}{2}\left[\left(\frac{\partial v}{\partial x}\right)^2 + \left(\frac{\partial w}{\partial x}\right)^2\right] \tag{9}$$

Considering plane sections remain plane, the displacements in the 3-orthogonal directions are given by:

$$u = u_o(x) - yv' - zw'\,;\ v = v_o(x);\ w = w_o(x) \tag{10}$$

Where u, v **and** w are, respectively, axial and transverse displacements. Applying equation (10) to (9), the following is obtained:

$$\varepsilon_x = u' + \frac{1}{2}\left[(v')^2 + (w')^2\right] - yv'' - zw'' \text{ or } \varepsilon_x = \varepsilon_o - yk_z + zk_y \tag{11}$$

where

$$\varepsilon_o = u' + \frac{1}{2}\left[(v')^2 + (w')^2\right]\,;\ k_y = -w'' \text{ and } k_z = v'' \tag{12}$$

To establish the formulation of the equilibrium problem, principle of virtual work is applied using both the components of the Piola-Kirchhoff tension and the Green-Lagrange deformation. The integration of the undisturbed volume characterizes the total Lagrangian formulation. In this sense, the following expression represents the work of the internal forces:

$$\delta W_{\text{int}} = \sum_{m=1}^{ne}\left\{\int_{\ell_m}\left[N_x\left(\delta u' + v'\delta v' + w'\delta w'\right) - M_y\delta w'' + M_z\delta v''\right]dx\right\} \tag{13}$$

where *ne* is total number of flat elements. After considering elastic properties for all prismatic elements, equation (14) gives the following expressions for the normal effort and bending moments, respectively:

$$N_x = \iint_{A_m}\sigma_x dA = EA\varepsilon_0\,;\ M_y = \iint_{A_m}\sigma_x z dA = EI_y k_y\,;\ M_z = -\iint_{A_m}\sigma_x y dA = EI_z k_z \tag{14}$$

The work of the external forces is indicated by:

$$\delta W_{ext} = \delta\mathbf{q}^T\mathbf{r}_m \tag{15}$$

where $\mathbf{r_m}$ represents the external load and q the set of nodal displacements generalized for a given element.

Thus, considering the principle of virtual work with kinematic relations (12) and equations (13), (14), and (15), the following relation can be obtained:

$$\delta\mathbf{q}^T\left(\int_{\ell_m}\left[N_x\left(\frac{\partial u'}{\partial\mathbf{q}} + v'\frac{\partial v'}{\partial\mathbf{q}} + w'\frac{\partial w'}{\partial\mathbf{q}}\right) - M_y\frac{\partial w''}{\partial\mathbf{q}} + M_z\frac{\partial v''}{\partial\mathbf{q}}\right]dx - \mathbf{r}_m\right) = 0 \tag{16}$$

The first part of the expression (16) represents the vector of the internal forces of an element. Equation (16) is valid for any $\delta\mathbf{q}^T$ variations. There is a set of *neq* nonlinear equations, where *neq* is the number of freedom degrees of the element. Thus equation (16) becomes:

$$\boldsymbol{\psi} = \int_{\ell_m} \left[N_x \left(\frac{\partial u'}{\partial \mathbf{q}} + v' \frac{\partial v'}{\partial \mathbf{q}} + w' \frac{\partial w'}{\partial \mathbf{q}} \right) - M_y \frac{\partial w''}{\partial \mathbf{q}} + M_z \frac{\partial v''}{\partial \mathbf{q}} \right] dx - \mathbf{r}_m = \mathbf{f}_m - \mathbf{r}_m = 0 \tag{17}$$

In this expression, $\mathbf{f_m}$ represents the vector of internal forces for one element. The vector of forces of the structure is assembled by the contribution of each finite element of conventional form.

The tangent stiffness matrix is assembled from the derivative of the $\mathbf{f_m}$ vector in relation to $\mathbf{q}$, the nodal displacements of the set. The following expression is valid for a finite element:

$$\mathbf{k}_T = \int_{\ell_m} \begin{bmatrix} \left(\frac{\partial u'}{\partial \mathbf{q}} + v' \frac{\partial v'}{\partial \mathbf{q}} + w' \frac{\partial w'}{\partial \mathbf{q}} \right) \left\{ \frac{\partial N_x}{\partial \mathbf{q}} \right\}^T + N_x \frac{\partial v'}{\partial \mathbf{q}} \left(\frac{\partial v'}{\partial \mathbf{q}} \right)^T + N_x \frac{\partial w'}{\partial \mathbf{q}} \left(\frac{\partial w'}{\partial \mathbf{q}} \right)^T \\ - \frac{\partial w''}{\partial \mathbf{q}} \left\{ \frac{\partial M_y}{\partial \mathbf{q}} \right\}^T + \frac{\partial v''}{\partial \mathbf{q}} \left\{ \frac{\partial M_z}{\partial \mathbf{q}} \right\}^T \end{bmatrix} dx \tag{18}$$

In this work, the following sets of nodal parameters were applied to the finite-element assemblage of the 3D frame:

$$\mathbf{q}_u^T = [u_1 \quad u_2]; \quad \mathbf{q}_v^T = \lfloor v_1 \quad \theta_{z1} \quad v_2 \quad \theta_{z2} \rfloor; \quad \mathbf{q}_w^T = \lfloor w_1 \quad \theta_{y1} \quad w_2 \quad \theta_{y2} \rfloor \tag{19}$$

The following form functions represent displacement fields:

$$\phi_u = \begin{bmatrix} \frac{1}{2}(1-\xi) \\ \frac{1}{2}(1+\xi) \end{bmatrix}; \quad \phi_V = \begin{bmatrix} \frac{1}{2} - \frac{3}{4}\xi + \frac{1}{4}\xi^3 \\ \frac{L}{2}\left(\frac{1}{4} - \frac{1}{4}\xi - \frac{1}{4}\xi^2 + \frac{1}{4}\xi^3 \right) \\ \frac{1}{2} + \frac{3}{4}\xi - \frac{1}{4}\xi^3 \\ \frac{L}{2}\left(-\frac{1}{4} - \frac{1}{4}\xi + \frac{1}{4}\xi^2 + \frac{1}{4}\xi^3 \right) \end{bmatrix}; \quad \phi_W = \begin{bmatrix} \frac{1}{2} - \frac{3}{4}\xi + \frac{1}{4}\xi^3 \\ -\frac{L}{2}\left(\frac{1}{4} - \frac{1}{4}\xi - \frac{1}{4}\xi^2 + \frac{1}{4}\xi^3 \right) \\ \frac{1}{2} + \frac{3}{4}\xi - \frac{1}{4}\xi^3 \\ -\frac{L}{2}\left(-\frac{1}{4} - \frac{1}{4}\xi + \frac{1}{4}\xi^2 + \frac{1}{4}\xi^3 \right) \end{bmatrix} \tag{20}$$

In this expression, non-dimensional coordinates correlate with the *x* coordinate by $\xi = \frac{2}{\ell}x - 1$. The derivatives of the displacement fields are represented by:

$$\frac{\partial u'}{\partial \mathbf{q}} = \begin{Bmatrix} \phi'_u \\ \mathbf{0}_v \\ \mathbf{0}_w \end{Bmatrix}, \quad \frac{\partial v'}{\partial \mathbf{q}} = \begin{Bmatrix} \mathbf{0}_u \\ \phi'_v \\ \mathbf{0}_w \end{Bmatrix}, \quad \frac{\partial w'}{\partial \mathbf{q}} = \begin{Bmatrix} \mathbf{0}_u \\ \mathbf{0}_v \\ \phi'_w \end{Bmatrix}, \quad \frac{\partial v''}{\partial \mathbf{q}} = \begin{Bmatrix} \mathbf{0}_u \\ \phi''_v \\ \mathbf{0}_w \end{Bmatrix}, \quad \frac{\partial w''}{\partial \mathbf{q}} = \begin{Bmatrix} \mathbf{0}_u \\ \mathbf{0}_v \\ \phi''_w \end{Bmatrix} \tag{21}$$

Considering the hypothesis of moderate rotations, their expressions are represented by $\theta_Z \cong v'$ and $\theta_Y \cong -w'$. Derivations of **u**, **v,** and **w** in relation to **(q)**, once and twice, give the expressions that are applied to equations (21), the vector of internal forces, and the matrix of tangent rigidity. The vector of internal forces, and the tangent stiffness matrix are given by the following expressions, respectively:

$$\mathbf{f}_m = \int_{\ell_m} \begin{bmatrix} N_x \phi'_u \\ N_x v' \phi'_v + M_z \phi''_v \\ N_x w' \phi'_w - M_y \phi''_w \end{bmatrix} dx \tag{22}$$

$$\mathbf{k}_T = \int_{\ell_m} \begin{bmatrix} \phi'_u \left\{ \frac{\partial N_x}{\partial \mathbf{q}} \right\}^T \\ \phi'_v \left(v' \left\{ \frac{\partial N_x}{\partial \mathbf{q}} \right\}^T + N_x \lfloor \mathbf{0}_u \quad \phi'_v \quad \mathbf{0}_w \rfloor \right) + \phi''_v \left\{ \frac{\partial M_z}{\partial \mathbf{q}} \right\}^T \\ \phi'_w \left(w' \left\{ \frac{\partial N_x}{\partial \mathbf{q}} \right\}^T + N_x \lfloor \mathbf{0}_u \quad \mathbf{0}_v \quad \phi'_w \rfloor \right) - \phi''_w \left\{ \frac{\partial M_y}{\partial \mathbf{q}} \right\}^T \end{bmatrix} dx \tag{23}$$

Equation (23) contains the derivatives of N_X, M_Y and M_Z in relation to $\mathbf{q}$, the set of generalized displacements. These derivatives are given by:

$$\frac{\partial N_x}{\partial \mathbf{q}} = \begin{Bmatrix} \phi'_u EA \\ v' \phi'_v EA \\ w' \phi'_w EA \end{Bmatrix} ; \frac{\partial M_y}{\partial \mathbf{q}} = \begin{Bmatrix} 0 \\ 0 \\ -\phi''_w \ I_y E \end{Bmatrix} ; \frac{\partial M_z}{\partial \mathbf{q}} = \begin{Bmatrix} 0 \\ \phi''_v \ I_z E \\ 0 \end{Bmatrix} \tag{24}$$

Geometric and mechanical relations for all sections of the structure have already been taken into account. Each section is considered to be homogeneous and have a symmetry axis. These conditions are represented by the following expressions:

$$\iint_{A_m} E_T dA = EA \, ; \iint_{A_m} E_T y^2 dA = I_z E \, ; \iint_{A_m} E_T z^2 dA = I_y E \, ; \iint_{A_m} E_T zy dA = 0 ; \iint_{A_m} E_T y dA = 0 ;$$

$$\iint_{A_m} E_T z dA = 0 \tag{25}$$

E is the Young's modulus, A the transversal area, I_Z and I_Y correspond to the inertia moments of each section. The torsion effect is assumed to follow its classic linear relation.

Assuming geometric non-linearity, the tangent stiffness matrix (equation 23) and the vector of internal forces (equation 22) are assembled for the simulation of the finite element of the 3D frames. The incremental-iterative procedure and the Newton-Raphson's technique are applied after considering the soil and foundation influences as described in the following section.

4 THE BUILDING-FOUNDATION-SOIL SYSTEM

Both the superstructure and the infrastructure were modeled by FEM using flat triangular and bar elements. The flat elements simulate laminar structures employing the membrane formulation developed by Bergan and Felippa (1985) and the plate bending formulation presented by Batoz and Dhatt (1979). The building is simulated by 3D frame elements and the influence of floor slabs is not considered.

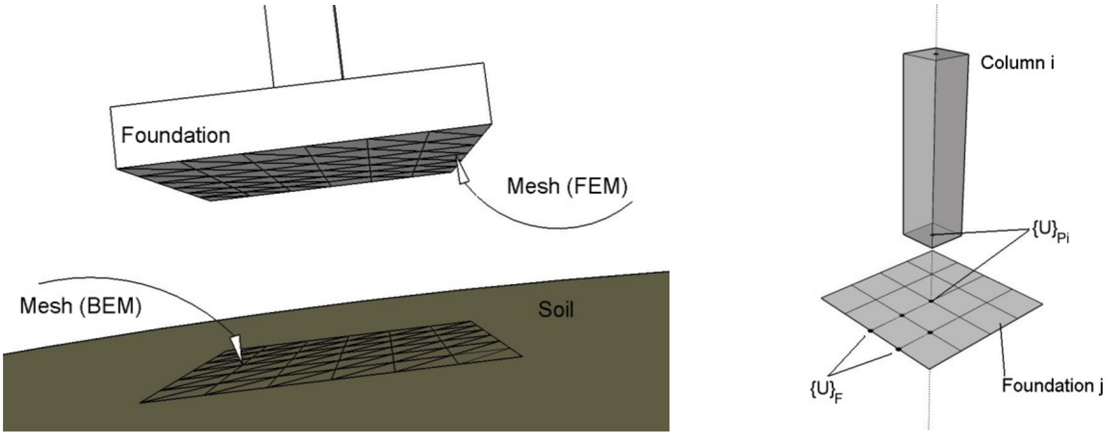

Figure 1: a) Mesh for the soil-foundation contact; b) Definition of the column and foundation nodes

The nonlinear problem is solved by the incremental iterative procedure which consists of the linearization of the equilibrium equation using Newton-Raphson's general procedure as described by Crisfield (1991).

The soil-foundation-building interaction can be achieved by considering the existing equilibrium conditions among nodal points that are common to the two methods. This generates a final algebraic system shown in Almeida and Paiva (2004) and represented in Figure 1a. However, this procedure is not used in this work. Considering that the building undergoes a nonlinear analysis, the computational time for the resolution of the final linear system of the BEM/FEM coupling increases drastically. This is a consequence of the BEM characteristics. The linear-system matrix to be handled for each iteration becomes larger, denser, and non-symmetrical.

To avoid this problem, contact nodes between foundation and soil, represented by equation (8), were condensed in order to produce building nodes only. Considering that both soil and foundation are idealized in the linear theory, their influences are inserted in the nodes at the base of the columns. In addition, they do not change during the incremental iterative procedure.

The FEM algebraic system is used to separate the nodes of the building and columns from the nodes of the foundation as shown in Figure 1.b. This system can be written as:

$$\begin{bmatrix} K_{C1C1} & K_{C1C2} & \cdots & K_{C1Cn} & K_{P1F1} & \cdots & K_{C1Fn} \\ K_{C2C1} & K_{C2C2} & \cdots & K_{C2Cn} & K_{C2F1} & \cdots & K_{C2Fn} \\ \vdots & & & & & & \\ K_{CnC1} & K_{CnC2} & \cdots & K_{CnCn} & K_{CnFn} & \cdots & K_{CnFn} \\ K_{F1C1} & K_{F1C2} & \cdots & K_{F1Cn} & K_{F1F1} & \cdots & K_{F1Fn} \\ \vdots & & & & & & \\ K_{FnC1} & K_{FnC2} & \cdots & K_{FnCn} & K_{FnF1} & \cdots & K_{FnFn} \end{bmatrix} \begin{Bmatrix} U_{C1} \\ U_{C2} \\ \vdots \\ U_{Cn} \\ U_{F1} \\ \vdots \\ U_{Fn} \end{Bmatrix} = \begin{Bmatrix} F_{C1} \\ F_{C2} \\ \vdots \\ F_{Cn} \\ F_{F1} \\ \vdots \\ F_{Fn} \end{Bmatrix} \tag{26}$$

This can be organized in four sub-blocks: column-column (CC); column-foundation (CF); foundation-column (FC), and foundation-foundation (FF) as shown below:

$$\begin{bmatrix} K_{CC} & K_{CF} \\ K_{FC} & K_{FF} \end{bmatrix} \begin{Bmatrix} U_C \\ U_F \end{Bmatrix} = \begin{Bmatrix} F_C \\ F_F \end{Bmatrix} \tag{27)/(28}$$

Where

$$\{U_C\}^T = \{U_{C1} \quad U_{C2} \quad \cdots \quad U_{Cn}\}^T \text{ and } \{U_F\}^T = \{U_{F1} \quad U_{F2} \quad \cdots \quad U_{Fn}\}^T \tag{29}$$

In order to isolate $\{U_F\}$, the algebraic equation (28) is rewritten and becomes:

$$\{U_F\} = [K_{FF}]^{-1}\{F_F - K_{FC} \cdot U_C\} \tag{30}$$

After using (30), the (28) expression becomes:

$$[K]_C \cdot \{U_C\} = \{\overline{F_C}\} \tag{31}$$

With

$$[K]_C = [K_{CC}] - [K_{CF}] \cdot [K_{FF}]^{-1} \cdot [K_{FC}] \tag{32}$$

$$\{\overline{F_C}\} = \{F_C\} - [K_{CF}] \cdot [K_{FF}]^{-1}\{F_F\} \tag{33}$$

Thus, the foundation-soil influence is applied to the nodes at the base of each column. This can interpreted as if the building lay on an elastic base and considering the soil-foundation influences stored in spring constants as represented in Figure 2. After the incremental-iterative analysis, the structure converges to its balance position. Then, equation (30) is used to calculate settlements and foundation and soil efforts.

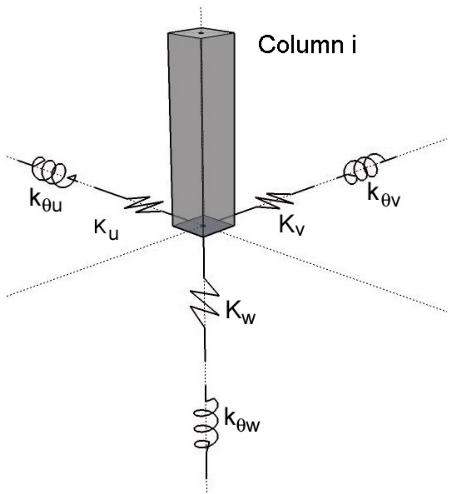

Figure 2: Representation of foundation and soil influences as springs

5 NUMERICAL EXAMPLES

5.1 2D Frame – Nonlinear Analysis

This example is a comparison to 2D Frame subjected to vertical and horizontal loads proposed by Elias (1986) and analyzed for Paula (2001).

The horizontal load is kept constant, the vertical load is divided in 10 steps and the tolerances in displacements and forces had been fixed in 10^{-6}. All elements have the same elastic and geometric properties shown in Figure 3.

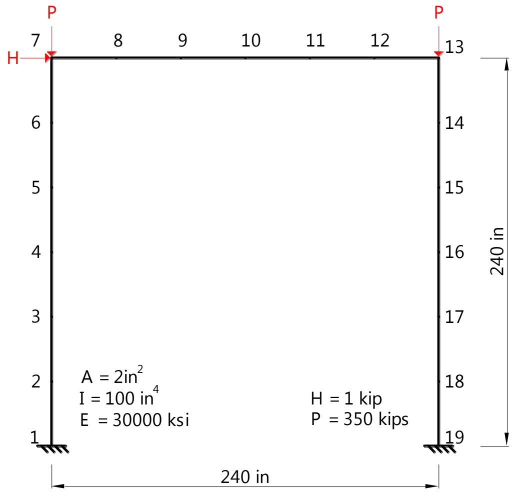

Figure 3: 2D Frame

In Figure 4 are presented the curves load-displacement gotten by Paula (2001) using the total Lagrangian lormulation and Update Lagrangian Formulation - with two developed tangent rigidity matrix, complete and approached, as well as the results presented by Elias (1986). The formulation used in the present work gets in results close to the founds for Paula (2001) and Elias (1986), proving the validity of the present work.

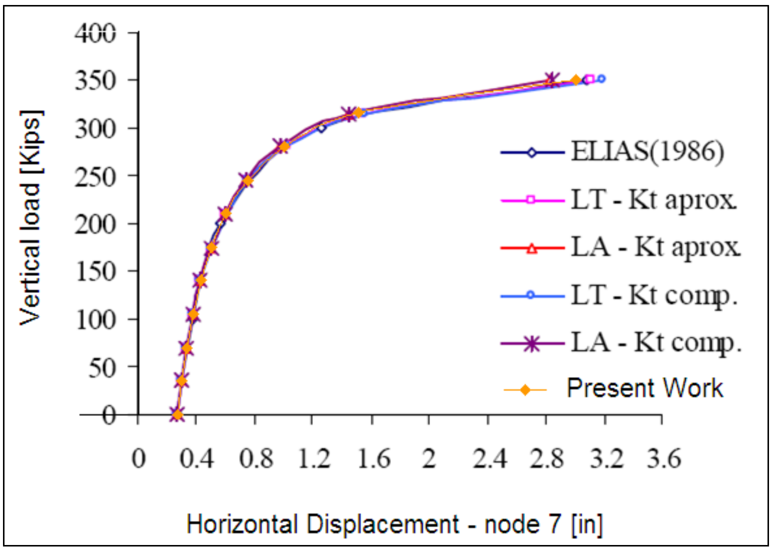

Figure 4: Curves load-displacement

5.2 3D Building – Soil-Structure Interaction – Linear / GNA

The following example represents a fictitious building elaborated to validate of the present work through software SAP 2000, the building is composed of 9 columns, 20 floors and 3m from floor to floor. In the building all the elements, beams and columns, have the same physical and geometric properties, gravitational loads uniformly distributed in the beams (10 kN/m) and horizontal loads concentrated in the top of each pillar in X direction (0.25 kN).

The building is simulated over several connecting conditions: clamped and interaction soil-structure through discrete (Winkler) and continues (Mindlin) models, and over linear and nonlinear analysis.

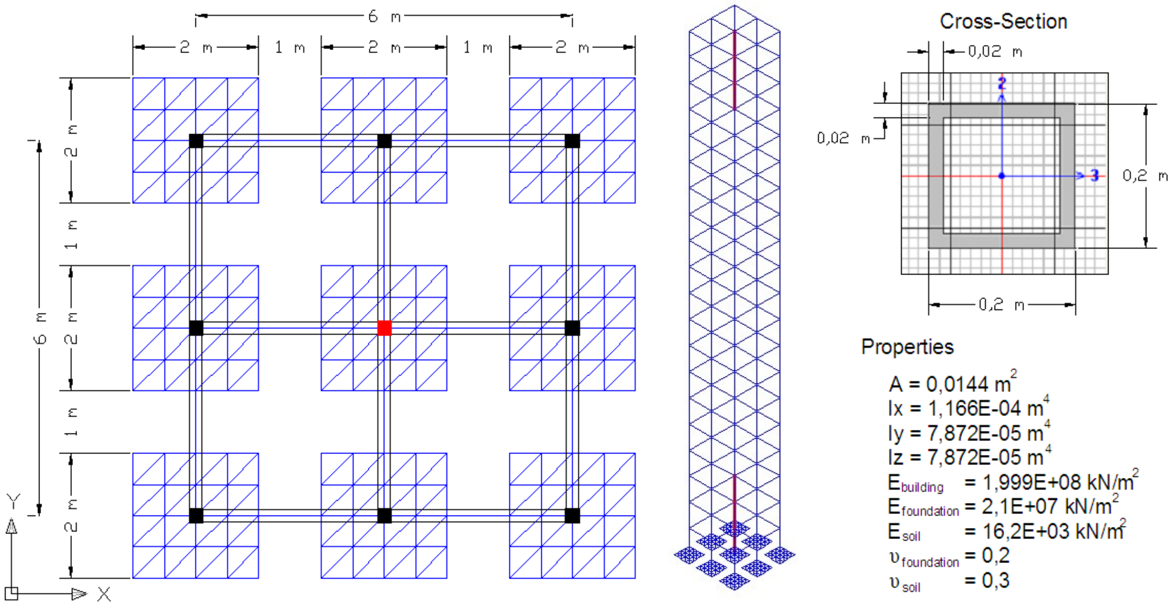

Figure 5: Physics and Geometrical Properties

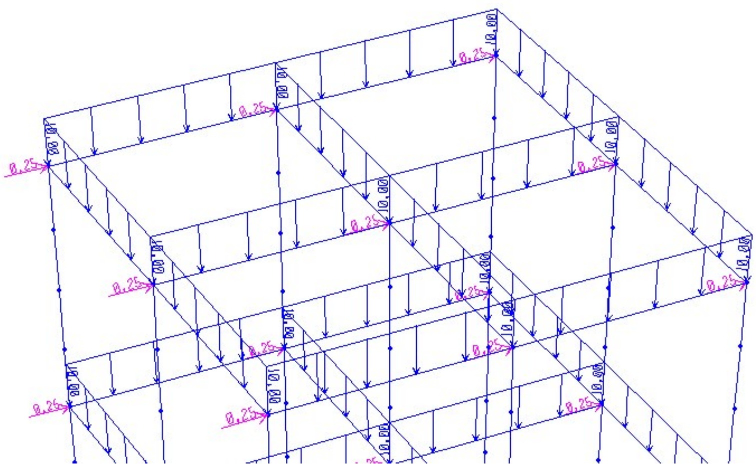

Figure 6: Loads distribution

Through estimated vertical soil deformation described in Poulos and Davis (1974) we can determine the vertical spring coefficient, which bases on the deformation of the soil by an unitary pressure applied on a rectangular area. In this paper we are assuming that the horizontal spring coefficient is 20% of the vertical.

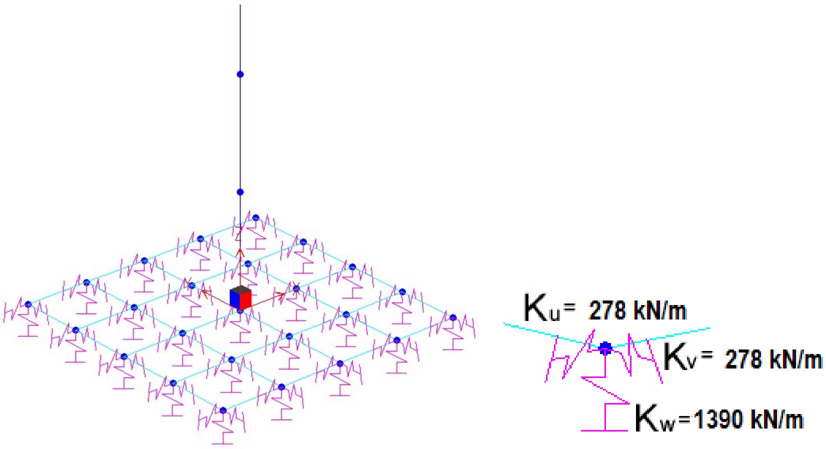

Figure 7: Foundation and soil (Winkler Model)

In Figure 8 is shown a comparison of the present work and SAP 2000 for the distribution of moments in the Y direction when considering global axes, clamped connection, linear and nonlinear analysis for the central column on the first's and lasted floors.

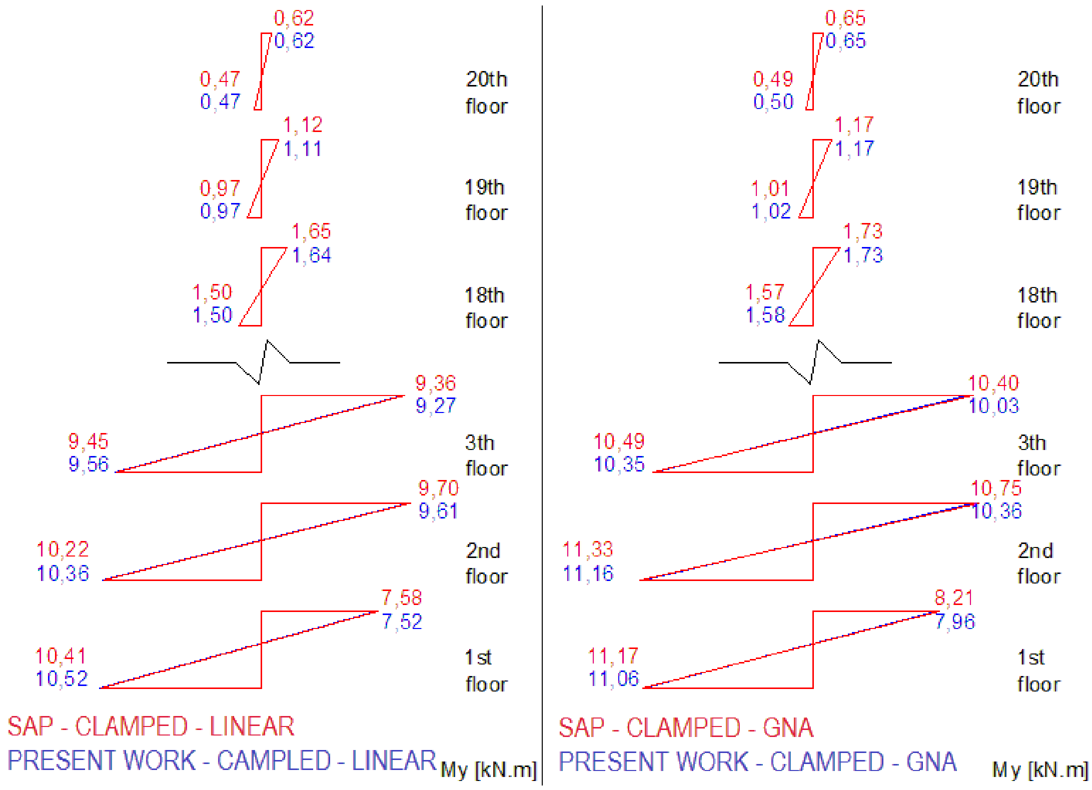

Figure 8: My - Clamped connection

In the linear and clamped connection analysis we got differences under 1.5% in the first's floor and in the geometrical nonlinear analysis the differences is under 3.7% in the first's floor. In the lasted floor we almost don't have differences.

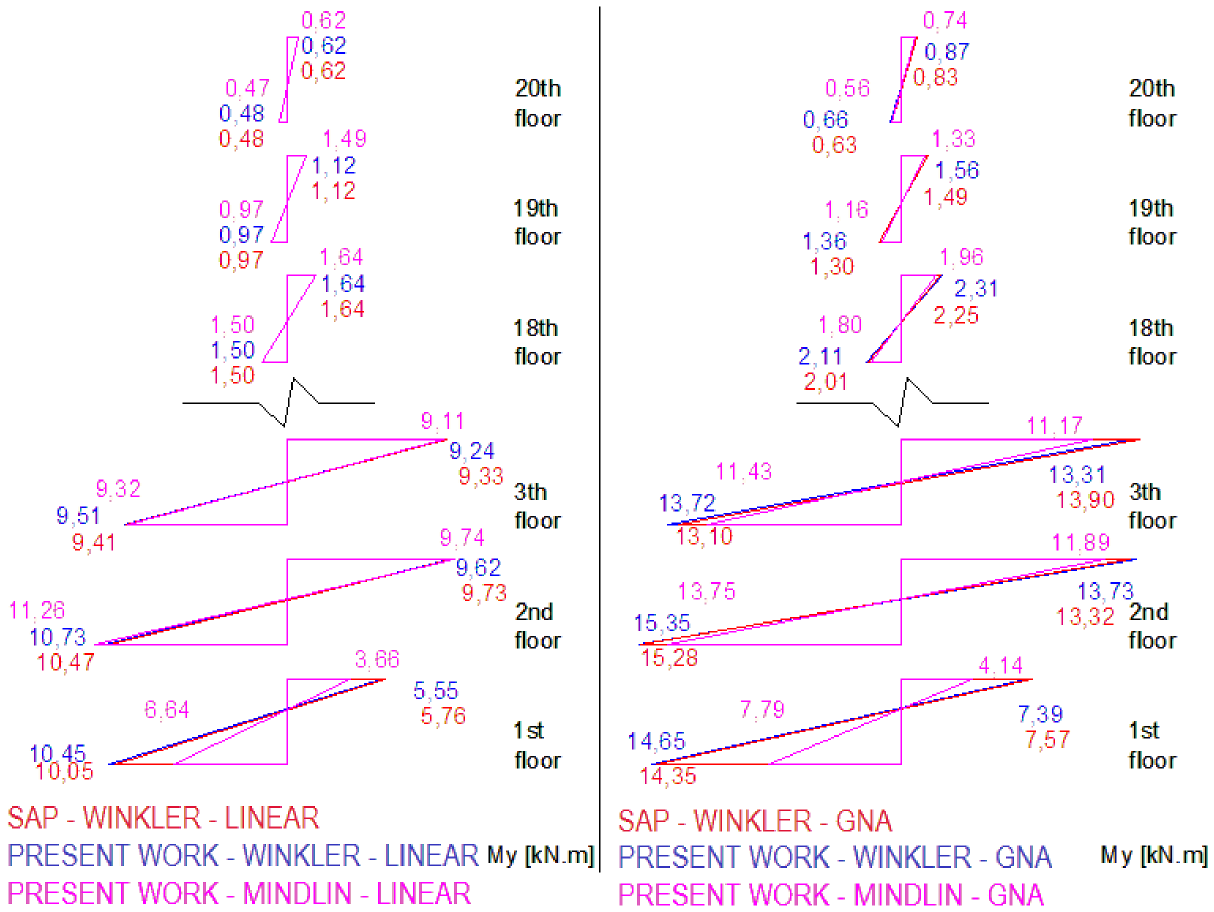

Figure 9: My – Interaction Soil-Structure

The difference of moment at the pillar base can be up to 2 times larger when comparing the analysis with soil-structure interaction and Mindlin´s model and the one using the clamped model.

Figure 9 indicates a significant difference of the efforts in pillars. The average difference is 40% when comparing the linear and nonlinear analyses.

It is important to emphasize that the structure behavior depend on the type of support. In this case, the differences get almost 50% especially in the first's floors according to established hypothesis.

6 CONCLUSION

The objective of this work was to verify the differences of structural behavior between the clamped model, which is usually adopted in the engineering practice, and other more refined models such as the Winkler´s discrete model and the Mindlin´s continuous model. A joint analysis between the infrastructure-superstructure and the soil massif is performed by these last two models.

The structural behavior depends on the applied model. Moreover, the occurrence of settlements is the main consequence of the analysis using soil-structure interaction. Differential settlement is the main cause of changes of the structure behavior.

In the Mindlin's model, which is considered to be a more refined analysis, the soil massif is represented as a continuous medium via BEM. However, for being a more refined analysis it requires longer processing time.

The Winkler's model also considers the soil-structure interaction. It replaces the soil by a set of springs and considers that there is no interaction among adjacent springs. The Winkler´s model is simpler, considered to perform a less refined analysis, and has a shorter processing time than the Mindlin's model.

The geometrically nonlinear analysis is iterative. It considers the structure in its deformed position. This analysis and the one that considers the soil-structure interaction lead to results that are closer to the real behavior of the structure.

The comparison of results from this work leads to the conclusion that the soil-structure interaction and the geometric not-linearity are very important considerations for the structural analysis of buildings.

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