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Soil-structure interaction for frame structures on shallow foundations

Interação solo-estrutura para sistemas estruturais reticulados sobre fundações rasas



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Abstract

This paper presents a program for consideration of the soil-structure interaction in the spatial analysis of frame structures. The method is based on the assumption of Winkler, which allows discrete adjacent springs to the shallow foundations simulating the influence of the settlements of support in three-dimensional structures. Although the model is in 3D and thus all the six degrees of freedom of each support may suffer displacements due to a settlement, in this paper, the analysis was made considering only the influence of the vertical translation of the support. The work consists in adding the methodology for consideration of flexible supports to the flowcharts presented by Gere and Weaver, Jr. [1], using the stiffness method, which is widely used for the frame structures analysis. Through the integrated analysis, contemplating parameters of infrastructure, superstructure and foundation ground, it was proved that the deformability of the soil has significant influence on the efforts redistribution and the entailment between the soil and the structure that best describes the physical behavior of a building and flexible supports condition.

Keywords: soil-structure; frame structures, shallow foundations.

Resumo

Neste artigo apresenta-se um programa para consideração da interação solo-estrutura na análise espacial de estruturas reticulares. O método empregado baseia-se na hipótese de Winkler, que admite molas discretas adjacentes as fundações rasas, simulando a influência dos recalques de apoio em estruturas tridimensionais. Embora o modelo seja 3D e, portanto, todos os seis graus de liberdade de cada apoio possam sofrer deslocamentos devidos a um recalque, neste artigo, a análise foi feita considerando apenas a influência da translação vertical dos apoios. O trabalho consiste em incorporar a metodologia para consideração de apoios flexíveis aos fluxogramas apresentados por Gere e Weaver Jr. [1], utilizando o Método da Rigidez, o qual é amplamente empregado para análise de estruturas reticulares. Através da análise integrada, contemplando parâmetros da infraestrutura, supraestrutura e do terreno de fundação, comprovou-se que a deformabilidade do solo tem influência significativa na redistribuição dos esforços e a vinculação entre solo e a estrutura que melhor descreve o comportamento físico de uma edificação é condição de apoios flexíveis.

Palavras-chave: solo-estrutura; estruturas reticulares; fundações rasas.

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1. Introduction

In the past, according to reports by Gusmão [2], it was common to consider all fully rigid support, even for acceptable displacement situations, such as foundations. This assumption was a necessary simplification for the technology offered at that time, being justified due to the extreme difficulty in manually analyzing buildings on flexible supports. However, the choice of a rigid model is a true "gap" between the prototype and the reality according to the author.
According to Velloso and Lopes [3], the analysis of the soil-structure system is essential and aims to provide the building translation and allow the study of the structural elements behavior, to guarantee the project quality. The proposal for the interaction consideration between the interfaces of the soil-structure system has as objective approach the theory to the reality, in order to assure the durability, stability and functionality of the work during its life.
The evolution of technology led to the development of faster computers, and as result, the advent of more sophisticated computer programs, enabling more realistic analyzes, which take into consideration the deformability of the adjacent soil to the foundation. Despite of the facilities caused by technological advancement it is possible to observe the use, by structural engineers, of the same simplified model of the past in the structural calculation of the current buildings. Based on the principle mentioned by Gusmão [2], the behavior of a building is the result of the interaction among infrastructure, superstructure and foundation ground, it becomes necessary the study of the interaction among these components. According to Colares [4] "There are several cases of buildings that had some deformity due to unanticipated changes in the mechanical behavior idealized in structural analysis." Among the defects we can highlight the incidence of major pathologies, such as gaps in beams and slabs or even columns crushing.
In this context, a program was developed in Visual Basic (VB), using Microsoft Excel ® platform, which considers the deformability of the adjacent soil to shallow foundations. The method is based on the assumption of Winkler, using discrete adjacent springs along the base of the foundations. This rheological model allows simulating the structure settlements and analyze the effects of the soil-structure interaction.

2. Soil-structure interaction

In Brazil the first contextualization on this subject were made by Chamecki [5], which the main idea of the work was to establish a relationship between the stiffness of the structure and the foundation settlements. In the author words "Solidarity among the elements of the structure, gives the same considerable stiffness, which makes the differential settlement becomes less accented than the calculated [...]" (CHAMECKI, 1954, pg. 37). Based on this conception we realize that the efficiency of the project depends on the analysis of the interaction between the soil and the structure.
The structural project idealized on a rigid base, without any displacement possibility, allows subdividing the building in three parts: superstructure, infrastructure and ground foundation. We note, through studies by Silva [6], this division is still making part of the structure analysis, in which the foundations are considered as elements infinitely rigid. This hypothesis is interpreted as independence between the parts, making the structural analysis not effective, because it limits the study of each subsystem in an isolation way [2].
Reis [7] emphasizes that this kind of analysis the superstructure calculator is worried only with the part above the ground, and the foundation engineer only with foundations elements and the adjacent soils to themselves. However, the behavior of the building is related to the interaction between the interfaces of the model components (superstructure, infrastructure, soil mass), and this interference is defined as the mechanical phenomenon of soil-structure iteration (SSI).
Several authors have demonstrated the importance on structural analysis incorporated to the study of settlements, according to Velloso, Santa Maria and Lopes [8], this study "[...] aims to provide the real displacements of the foundation - and also of the structure if it is included in the analysis - and its internal efforts". Therefore, it is essential the correct evaluation of the model support to become the construction project more realistic, taking into consideration the factors of interference between soil and structure.

3. Analysis of the soil deformability

To understand the effects of soil-structure interaction is necessary the comprehension of how the soil behaves when it is subjected the loads of edification, as well as, their physical behavior during the loading process. During this process, in the understanding of Cintra, Aoki and Albiero [9], inevitably vertical displacements occur, downwards, usually in order of centimeters, and in exceptional cases may reach hundreds of centimeters. This deformation in relation to rigid is called settlement.
According to Simons and Menzies [10] the foundation settlements may be considered as: immediate settlement (*wi*); primary consolidation settlement (*wc*) and secondary consolidation settlement (*ws*), $w = w_i + w_c + w_s$. The immediate settlement is the predominant portion of consolidation in sandy soils and is time independent. It results in the deformation almost instantaneous when the load is applied to the soil, without the occurrence of the reduction of the void ratio in soil mass. While soil is not an elastic material, because the settlements are not recovered by unload, the immediate settlement is calculated using the theory of elasticity due to the initial volume deformation be constant in the soil mass [6].
In low permeability soils, as in the case of saturated clays, great part of the foundation settlement is due to the consolidation of the subjacent layer. In the case of settlement by consolidation, both the primary and the secondary are time-dependent and result due to the reduction of the void ratio. The primary settlement occurs because of the dissipation of excess of neutral pressure present in the solid after loading, while the secondary settlement modifies the structure of the soil without having an increase in load, in other words, without increasing the effective stress. But despite the nomenclature used to differentiate them, it does not mean that they happen at different times [10].

3.1 Winkler model

Predict the mechanical behavior of a soil mass is a complex task due to the heterogeneity of the material, which varies from clay particles to boulders. While the soil is not an elastic material because it does not recover the original volume after the unloading of itself, in conventional analyzes the actuating stress is limited at the base of the foundation, until the admissible stress of the soil. Under these conditions it is possible the application of the Winkler model.

This soil behavior model admits that the contact pressures are proportional to the displacement (w) of any point on the surface of the ground when loaded. For the case of vertical strain, the stress is given by Equation (1):

$$\sigma = k_s^v . w_i \quad (1)$$

Where:
σ is the average contact stress at the base of the foundation.
w_i is the vertical displacement (settlement).
k^v_s is the module vertical reaction. This value depending to the type of soil that form the bulk of the foundation.
These springs are represented by the coefficient of elastic support Ks (kN/m), which is directly proportional to the vertical reaction module Ki (kN/m³) and the loading area A_f (m²), according to equation (2).

$$K_i = \frac{K_s}{A_f} \quad (2)$$

According to Moraes [11] it is possible to admit that the foundation base keeps rigid after the elastic deformation of the soil, which allows considering a linear variation of stress. Under these conditions, it is possible to calculate the displacements from the elastic support coefficients Ks (kN / m), according to the equation (3).

$$w = \frac{N}{K_V} = \frac{F}{K_s^v . A_f} \quad (3)$$

Where:
N is an action in the foundation base.
F is é a normal force to the analyzed section.
The reaction module K^v_s is not a constant of the soil and depends on various factors, such as: shape and size of the foundation, type of construction and load changes (MORAES, 1976). In general, the coefficient K^v_s can be determined in three ways: plate tests, tables of typical values and through correlations with elastic modulus.
In the absence of appropriate tests Béton-Kalender (1962, apud MORAES [11]) recommends the use of the values in Table 3.1, for the vertical reaction module and values of the table 3.2, for the elasticity module (for soils subjected to a lower stress 1 MPa). These properties were obtained through the metal plate tests with a diameter of 45 cm.
The values proposed in the bibliography must be corrected, because according to Velloso and Lopes [3], vertical reaction modules does not derive only from soil properties but also from a loading system, so they must be corrected for this situation, considering the size and shape of the analyzed element. The authors propose a correlation using equation (4), assuming the soil as an elastic homogeneous and semi-infinite material to approach the value to the real situation.

$$K_B^v = K_b^v . \frac{b \, I_{s\,b}}{B \, I_{s\,B}} \quad (4)$$

Where:
K^v_B is the vertical reaction module of the foundation;
K^v_b is the vertical reaction module of the plate.
b is a smaller dimension of the plate;
B is a smaller dimension of the foundation;
I^s_B is the shape material of the foundation;
I^s_b is the shape factor of the plate.

Table 3.1 – Values of K_s^v

Type of soil	K^V_s (kN.m^{-3})
Light peat - Marshy ground	5,000 to 10,000
Heavy Peat - Marshy ground	10,000 to 15,000
Fine sand beach	10,000 to 15,000
Landfill silt, sand and gravel	10,000 to 20,000
Soaked clay	20,000 to 30,000
Wet clay	40,000 to50,000
Dry clay	60,000 to 80,000
Dry clay hardened	100,000
Compacted silt with sand and stone	80,000 to 100,000
Compacted sand and silt with many stones	100,000 to 120,000
Gravel with fine sand	80,000 to 120,000
Medium gravel with fine sand	100,000 to 120,000
Coarse gravel with coarse sand	120,000 to 150,000
Coarse gravel with little sand	150,000 to 200,000
Coarse gravel with little compacted sand	200,000 to 250,000

Table 3.2 – Values of E_o (endometrial elasticity module) and E (elasticity module)

Values of E_o and E	E_o (MPa)	E (MPa)
Peat	0.1 to 0.5	0.07 to 0.35
Soaked clay	1.5 to 4.0	0.99 to 2.2
Plastic clay	4.0 to 8.0	2.6 to 5.3
Hardened clay – plastic	8.0 to15.0	5.3 to 9.9
Loose sand	10.0 to 20.0	6.6 to13.2
Compact sand	50.0 to 80.0	33.0 to 53.0

The shape factors, are recommended by Perloff (1975, apud VELLOSO; LOPES [3]), as shown in Table 3.3 [3]. In case of problems with thickness of finite compressible layer we use a similar table which can be obtained, in Velloso and Lopes [3].
The settlement (w) can also be obtained by a direct calculus, based on the theory of elasticity. According to Velloso and Lopes [3], this method is widely used in the analysis of SSI, and it is always associated with simplified models of the soil behavior. The authors present for the settlement prediction, in footings under centered load, the equation (5).

$$w=q\,.B\,.\,\frac{1-v^2}{E}\,.I_s\,.I_d.I_h \qquad (5)$$

Where:
w is the direct settlement;
q is the medium applied pressure;
B is the smallest dimension of the foundation;
υ is Poisson coefficient;
E is the elasticity module;
I_s is is the shape factor;
I_d is the embedded factor;
I_h is the thickness factor of the compressible layer.

The coefficient Is is a function of the footing shape and its stiffness. In the flexible case depends on the position of the point on the footing (center, edge, etc.) to which is desired the estimate of the immediate settlement. Thus, equation (5) can be used for rigid and flexible footing, with appropriate values of Is, presented in Table 3.3. The dimension characteristic of the footing, B, is taken by convention, as the diameter of the circular footing or as the length of the shorter side of a rectangular footing.
The shape factors (Is) are usually tabulated for certain values I_d and I_h. The Table 3.3 shows these values for the case of loading on the surface of a medium infinite thickness, where I_d and I_h are equal 1.0. From the settlement it becomes possible the determination of the spring coefficient (Kv), applying the deformation obtained in the equation (3).

4. Methodological procedures

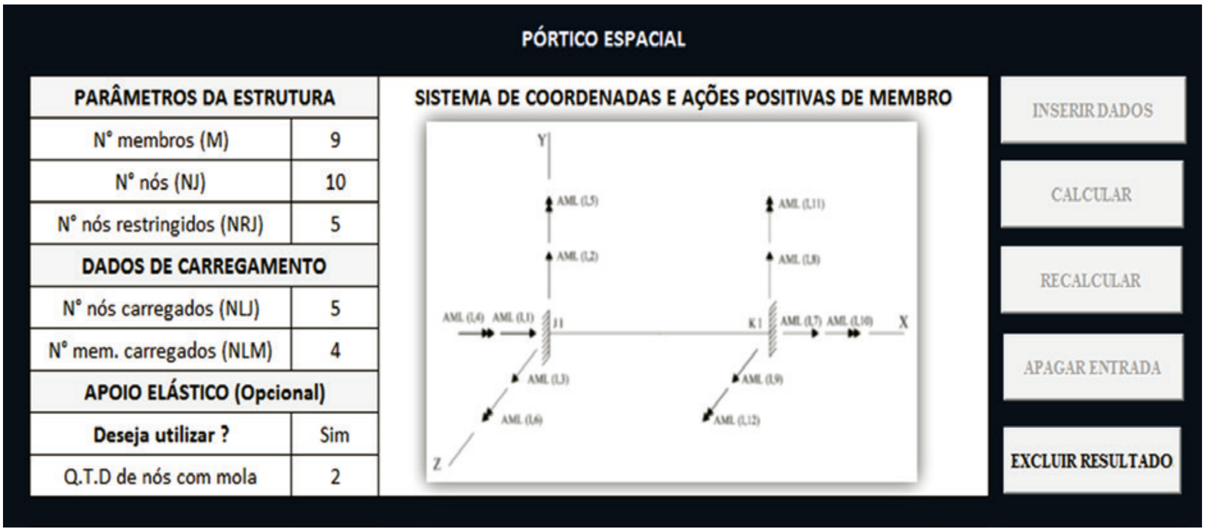
4.1 Program development

The initial step was the formalization of the computational program, based on the flow charts presented by the authors Gere and Weaver Jr. [1], for space frame, using the stiffness method for the loads and displacements determination in frame structures. It was used the programming language of Visual Basic (VB) Microsoft

Table 3.3 – Shape factors Is, for loadings on the surface in an infinite thickness way (12)

Shape	Flexible			Rigid
	Center	Edge	Average	
Circle	1.00	0.64	0.85	0.79
Square	1.12	0.56	0.95	0.99
Rectangle	–	–	–	–
L/B = 1,5	1.36	0.67	1.15	–
2	1.52	0.76	1.30	–
3	1.78	0.88	1.52	–
5	2.10	1.05	1.83	–
10	2.53	1.26	2.25	–
100	4.00	2,00	3.70	–
1000	5.47	2.75	5.15	–
10000	6.90	3.50	6.60	–

Figure 4.1 – Partial view of the data entry program



Excel ® platform. The main view of the developed software can be seen in Figure 4.1.

However, as the key requirement to validate the computational model is that it produces satisfactory results when compared to the results from the literature corresponding to the computational flowchart. Therefore, through an example of space frame, example I (5.1.1) available in the book Frame Structural Analysis by the authors Gere and Weaver, Jr. [1], a numerical test was performed.

4.2 Methodology for the flexible support inclusion

This procedure is done by replacing the rigid support by a flexible one, through replacing the degrees of freedom by a defined stiffness spring. This spring, partially constraints the displacement of a particular joint, thus characterizing a condition of elastic support.

4.3 Methodology for the consideration of the soil-structure interaction

The technique consists in calculating the support reactions of the structure, with rigid support, and from these values estimate the dimensions of the foundation to later apply the equation (5), to get immediate settlement. The elastic coefficient, for the base of each column, can then be obtained from equation (3). In a new analysis, rigid supports are replaced by the springs coefficients, at this point new reactions, new settlements and new springs coefficients are obtained. As the spring coefficients derive specifically from the type of soil and foundation dimensions, at each iteration, the foundation elements should be resized. The process is iterative and reaches the end when the settlements or support reactions converge to the same value. This procedure is based on the methodology presented by Chamecki [5].

4.3.1 Spatial model

The spatial model, illustrated by the Figure 4.2, consists of sixty-nine members and forty joints, initially the joints 1-10 will be constraints, then these vertical constraints will be replaced by relative stiffness coefficients, calculated as the equation (3) according to the elastic settlement of the points that rest on solid ground.

Taking into consideration the loads, the spatial model uses only uniform loads Q, acting in the negative Y direction, applied to all horizontal structural members and all members are subject to actions due to their own weight (γ_c = 25 kN/m³). The structural elements are reinforced concrete with compressive strength (f_{ck}) de 25 MPa.

At the beginning, the model was submitted to the SSI analysis in situations of homogeneous soil, in the specific case, sand and clay. Then (in a characterized combination as a landfill performed in a sideband in the transverse direction (Z) of the structure), the P01 columns (member 01) and P06 (member 06) were submitted to spring coefficients of lower stiffness than for the other columns.

All the beams have dimensions of 15x70 cm, base and height respectively, while the columns have a section of 20x20 cm. The other physical parameters for the resolution since structural model are listed in Table 4.1. The numeration, location and nomenclature of the members and foundations can be seen in the Figure 4.2.

Two types of mass soils are used in the analysis, a soil with less strength and other more strength capacity. The soil with less stiff-

Table 4.1 – General physical parameters for the spatial structural model

Q (kN/m)	E (kN/m²)	G (kN/m²)
30.0	28,000,000.00	11,666,666.70
L_x (m)	L_y (m)	L_z (m)
5.0	3.0	8.0

Figure 4.2 – Nomenclature for the spatial structural model members

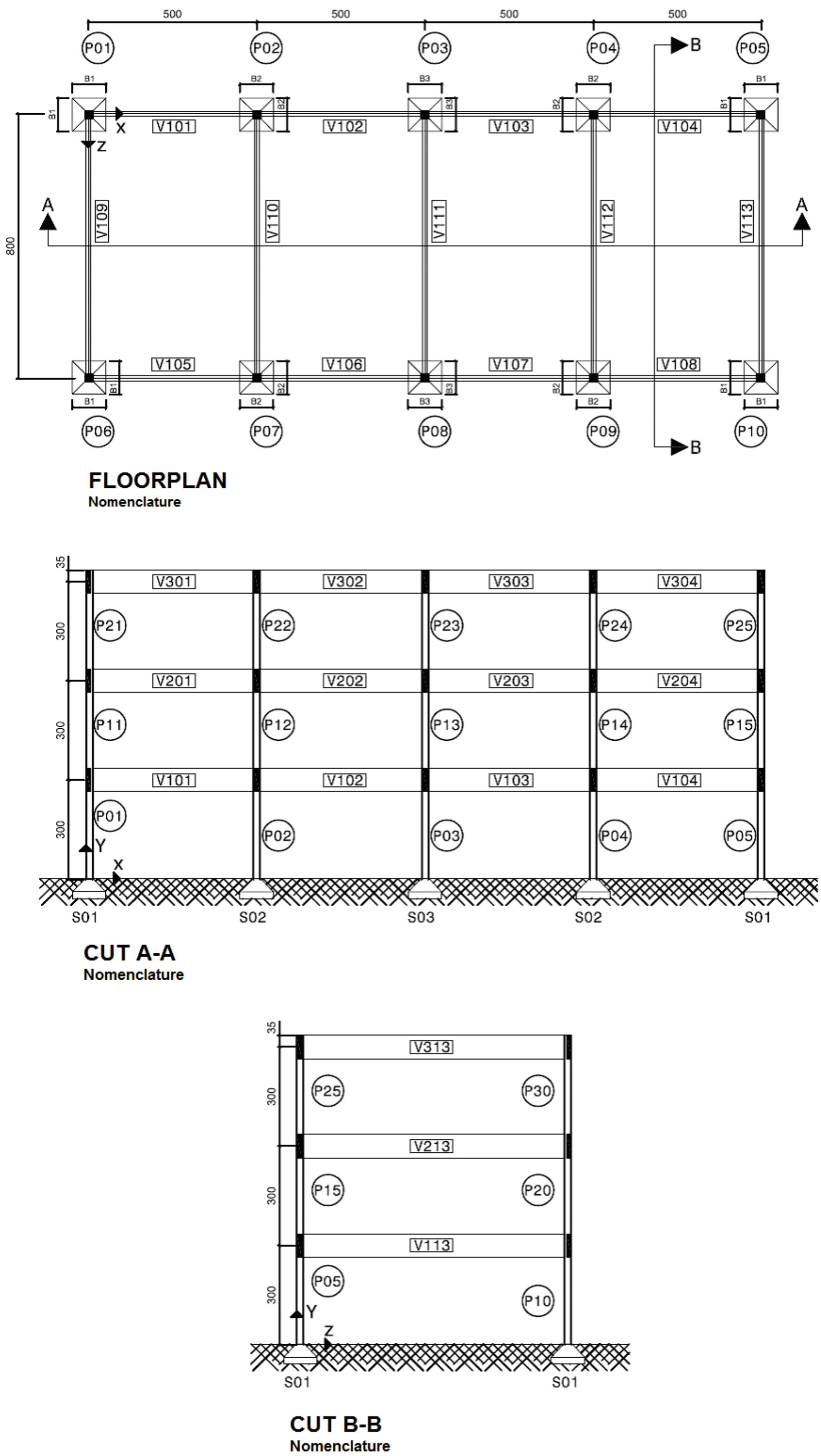
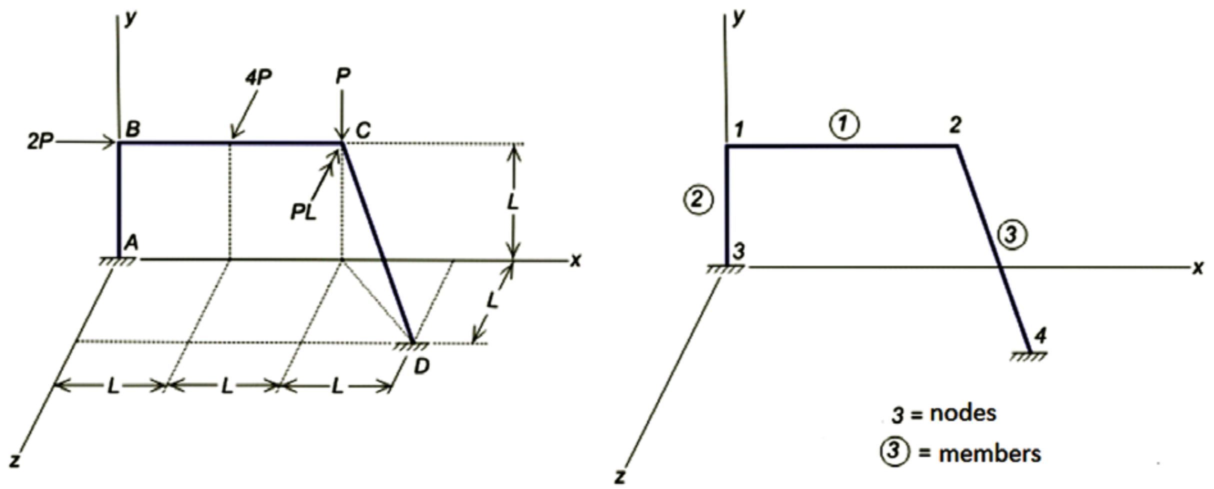


Figure 5.1 – Example I



ness is clay with elasticity module (Es) of 35 MPa and Poisson's coefficient equal to 0.3, with a resultant basic stress of 0.2 MPa. The most resistant soil is sand with elasticity module (Es) of 70 MPa and Poisson's coefficient of 0.4, with a basic stress of 0.4 MPa. For purposes of calculus the footing will be considered on the ground surface and hinged, limiting the analysis only to the vertical displacements of each support.

5. Presentation and discussion of the results

5.1 Program validation

5.1.1 Example I

The first example, shown in Figure 5.1 [1], consists of three members and four joints, where two are completely constrained (A and D), resulting in twelve support constraints, the rest are free getting twelve degrees of freedom to the structure (six in each of the joints B and C). The joint loads consist of: 2P in the positive direction of the X axis, in point B; P in the negative direction of the Y axis, at point C, and a torque PL in the negative direction of the Z axis at the point C. Further, on BC member, there is a load of 4P in the positive Z direction, applied at the gap of each member.

The physical parameters for this structural model resolution are shown in Table 5.1 (Gere and Weaver Jr. [1]).

The results generated by the software, for the displacements and reactions of the example I, are shown in table 5.2.

The results for the displacements and reactions of the example I, according to Gere and Weaver Jr. (1987, p. 366), can be observed in Table 5.3 [1].

It is possible to observe the compatibility between the results presented by Gere and Weaver Jr. (1987, p. 366) and the results obtained with the developed software. It is important to highlight that other examples were analyzed and all the results are compatible with the literature, not being added to the present work, for not writing too much text.

Table 5.1 – Example data I

E (MPa)	P (kN)	A_x (m^2)	I_y (m^4)
206,842.71	4.45	0.0071	$2.33.10^{-5}$
G (MPa)	**L (m)**	**I_x (m^4)**	**I_z (m^4)**
82,737.08	3.5	$3.45.10^{-5}$	$2.33.10^{-5}$

5.2 Results for the spatial model

From the support reactions obtained from the spatial model, for each combination under rigid and flexible base, considering the described methodology for the consideration of the soil-structure interaction (item 4.3), tables for each iteration and to each column were developed. The convergence process, for the lateral landfill combination (P01/P06), can be seen in Table 5.4.

Applying to the methodology again it was created the table 5.5, which presents a summary table of the final reactions for all the cases proposed on item 4.3.1.

The structure behavior can be analyzed from the variation percentage of the three combinations, for the column reactions, in relation to the efforts obtained in the situation of rigid support. Figure 5.2 has as objective demonstrate the effort migration according to the elastic base (P01/P06 combination simulates a lateral landfill according to the item 4.3.1.)

Table 5.2 – Displacements and reactions of the example I

Joint, displacements (m e rad/m) and Reactions (kN e kN.m)						
Joint	Trans. X Load X	Trans. Y Load Y	Trans. Z Load Z	Rot. X Bending X	Rot. Y Bending Y	Rot. Z Bending Z
1	-0.00388112 0	0.00000618 0	0.01590856 0	0.0001914 0	-0.0001387 0	0.0000679 0
2	-0.00391674 0	0.01158629 0	0.01559181 0	0.0000910 0	0.0001459 0	-0.0000686 0
3	0 -0.396	0 -2.980	0 -9.038	0 -25.694	0 5.123	0 -3.628
4	0 -8.762	0 7.43	0 -8.762	0 -2.903	0 -5.032	0 3.50

Table 5.3 – Displacements and reactions of the example I, according to the bibliography

Joint, displacements (m e rad/m) and Reactions (kN e kN.m)						
Joint	Trans. X Load X	Trans. Y Load Y	Trans. Z Load Z	Rot. X Bending X	Rot. Y Bending Y	Rot. Z Bending Z
1	-0.0038811 0	0.0000062 0	0.0159085 0	0.000191 0	-0.000139 0	0.000068 0
2	-0.0039167 0	0.0115863 0	0.0155918 0	0.000091 0	0.000146 0	-0.000069 0
3	0 -0.4	0 -2.98	0 -9.04	0 -25.69	0 5.12	0 -3.63
4	0 -8.76	0 7.43	0 -8.76	0 -2.90	0 -5.03	0 3.50

Table 5.4 – Stiffness, coefficients, reactions and foundation elements

Combination – lateral landfill - P01/P06									
Column	Rigid support			Flexible support					
				Iteration 1			Iteration 2		
	K_v ($kN.m^{-1}$)	Rv (kN)	Footing (m)	K_v ($kN.m^{-1}$)	Rv (tf)	Footing (m)	K_v ($kN.m^{-1}$)	Rv (kN)	Footing (m)
P1	∞	608,87	1.85x1.85x0.55	71872.6	603.41	1.85x1.85x0.55	71872.6	601.84	1.85x1.85x0.55
P2	∞	927.48	1.60x1.60x0.47	134680.1	935.87	1.65x1.65x0.48	138888.9	939.83	1.65x1.65x0.48
P3	∞	887.30	1.60x1.60x0.47	134680.1	893.04	1.60x1.60x0.47	134680.1	890.76	1.60x1.60x0.47
P4	∞	927.48	1.60x1.60x0.47	134680.1	912.67	1.60x1.60x0.47	134680.1	911.61	1.60x1.60x0.47
P5	∞	608.87	1.30x1.30x0.37	109427.6	615.01	1.35x1.35x0.38	113636.4	615.95	1.35x1.35x0.38
P6	∞	608.87	1.85x1.85x0.55	71872.6	603.41	1.85x1.85x0.55	71872.6	601.84	1.85x1.85x0.55
P7	∞	927.48	1.60x1.60x0.47	134680.1	935.87	1.65x1.65x0.48	138888.9	939.83	1.65x1.65x0.48
P8	∞	887.30	1.60x1.60x0.47	134680.1	893.04	1.60x1.60x0.47	134680.1	890.76	1.60x1.60x0.47
P9	∞	927.48	1.60x1.60x0.47	134680.1	912.67	1.60x1.60x0.47	134680.1	911.61	1.60x1.60x0.47
P10	∞	608.87	1.30x1.30x0.37	109427.6	615.01	1.35x1.35x0.38	113636.4	615.95	1.35x1.35x0.38

Table 5.5 – Reactions and convergences

Column	Spatial model – axial columns: Rigid support – Infinite stiffness	Flexible suport – Sand	Flexible suport – Clay	Flexible suport – Lanfill P01/P06
P1	608.87 kN	616.17 kN	619.50 kN	601.84 kN
P2	927.48 kN	912.91 kN	907.44 kN	939.83 kN
P3	887.30 kN	901.82 kN	906.12 kN	890.76 kN
P4	927.48 kN	912.91 kN	907.44 kN	911.61 kN
P5	608.87 kN	616.17 kN	619.50 kN	615.95 kN
P6	608.87 kN	616.17 kN	619.50 kN	601.84 kN
P7	927.48 kN	912.91 kN	907.44 kN	939.83 kN
P8	887.30 kN	901.82 kN	906.12 kN	890.76 kN
P9	927.48 kN	912.91 kN	907.44 kN	911.61 kN
P10	608.87 kN	616.17 kN	619.50 kN	615.95 kN

The variations of the results show a trend of uniformity of the loads. It is possible to see that the soil with lower reaction coefficient causes greater effort redistribution. However, it should be noticed that the magnitude of the effort redistribution was not so significant, due to the large foundation dimensions and, consequently, more rigid. The variations reached a maximum of 2.08% for increases and a minimum of -2.21% for alleviations, both while the model was seated on the clay.
In the combination between two types of soil, where P01 and P06 columns are submitted to a lower stiffness coefficient compared to the other, it is seen the influence of the structure rotation in its behavior, causing load alleviations in order of 1.17% in P01 and P06 columns and overloading the neighboring columns, P02 and P08, in the order of 1.31%.

The behavior of the efforts, according to the floor increase, can be observed through P01-P21 vertical lines, symmetrical to the vertical lines P06-P26. Normal and bending efforts acting on the base of the columns, analyzed in module due to the symmetry of the reforced arrangement, are listed respectively in tables 5.6.
On the other hand the discrete variation in normal efforts of the columns, the moments had a significant increase, in relation to rigid situation, reaching up to 39.087% on the clay base. Attenuating moments in the transversal direction of the spatial model showed no change compared to the efforts obtained with the model of fixed supports, justified by the absence of rotation in transversal members.
The vertical lines behavior P01-P21, through each floor, can be seen in the graphs presented in the figures 5.3 and 5.4.

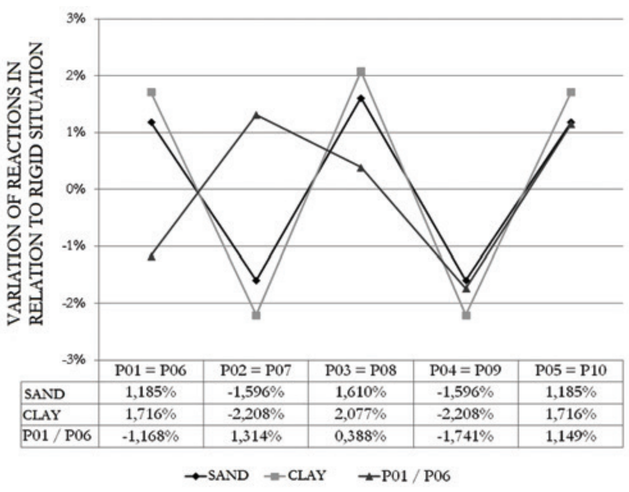
Table 5.6 – Acting efforts in the vertical lines members P01-P21

Spatial model – vertical lines efforts P01-P21

Column	Rigid support – Infinite stiffness N	MY	MZ	Flexible support – Sand N	MY	MZ
P01 = P06	608.87 kN	0.00 kN.m	0.00 kN.m	616.17 kN	0.00 kN.m	0.00 kN.m
P11 = P16	407.03 kN	30.56 kN.m	7.01 kN.m	411.73 kN	30.56 kN.m	8.82 kN.m
P21 = P26	202.93 kN	30.54 kN.m	7.84 kN.m	204.98 kN	30.54 kN.m	9.57 kN.m

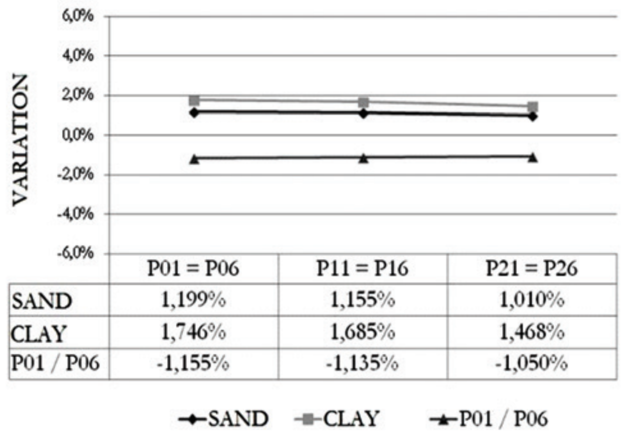
Column	Flexible support – Clay N	MY	MZ	Flexible support – P01 / P06 N	MY	MZ
P01 = P06	619.50 kN	0.00 kN.m	0.00 kN.m	601.84 kN	0.00 kN.m	0.00 kN.m
P11 = P16	413.89 kN	30.56 kN.m	9.75 kN.m	402.41 kN	30.56 kN.m	5.75 kN.m
P21 = P26	205.91 kN	30.54 kN.m	10.48 kN.m	200.80 kN	30.54 kN.m	6.62 kN.m

Figure 5.2 – Graphical analysis of the percentage change of the vertical reactions of the spatial structural model



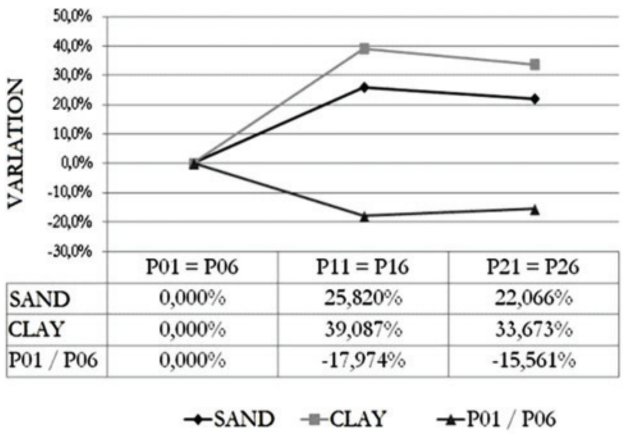
	P01 = P06	P02 = P07	P03 = P08	P04 = P09	P05 = P10
SAND	1,185%	-1,596%	1,610%	-1,596%	1,185%
CLAY	1,716%	-2,208%	2,077%	-2,208%	1,716%
P01 / P06	-1,168%	1,314%	0,388%	-1,741%	1,149%

Figure 5.3 – Axial Effort variation for the vertical line P01-P21



	P01 = P06	P11 = P16	P21 = P26
SAND	1,199%	1,155%	1,010%
CLAY	1,746%	1,685%	1,468%
P01 / P06	-1,155%	-1,135%	-1,050%

Figure 5.4 – Variation of the bending moment (MZ) for the vertical line P01-P21



	P01 = P06	P11 = P16	P21 = P26
SAND	0,000%	25,820%	22,066%
CLAY	0,000%	39,087%	33,673%
P01 / P06	0,000%	-17,974%	-15,561%

According to the variations, gotten through each floor, it is possible to observe that independently of the combination or the vertical line analyzed, the variations are bigger in the members closer to the foundations. It happens because of the increase on the stiffness structure, proportionally to the floor increase, which causes lower rotations.

To analyze the beams behavior was selected longitudinal reference beams (X-axis). The beams are V101-V301, symmetric the beams V305-V105, in the longitudinal direction. The shearing efforts are related in Table 5.7 and its variations are represented by the graph in figure 5.5.

The bending moments can be seen in table 5.8 and its variations are presented in Figure 5.6.

Analyzing the figures 5.5 and 5.6 it is possible to see that the shearing efforts did not have significant variation percentage. However, the high bending moment variation is due to the direct influence of the rotation of the ends, which is higher in the case of flexible support. It is also notable the reduction variation with the

Table 5.7 – Shear effort for the beams V101,V201 and V301

Spatial model – shearing					
Beam		Rigid support	Flexible support		
		Infinite stiffness	Sand	Clay	Lanfill P01/P06
V101	P01	68.3 kN	70.9 kN	72.1 kN	65.9 kN
	P02	94.8 kN	92.2 kN	91.0 kN	97.2 kN
V201	P11	70.6 kN	73.3 kN	74.5 kN	68.1 kN
	P12	92.5 kN	89.9 kN	88.7 kN	95.0 kN
V301	P21	69.4 kN	71.5 kN	72.4 kN	67.3 kN
	P22	93.7 kN	91.6 kN	90.7 kN	95.8 kN

Figure 5.5 – Shearing effort variation for the beams V101, V201 e V301

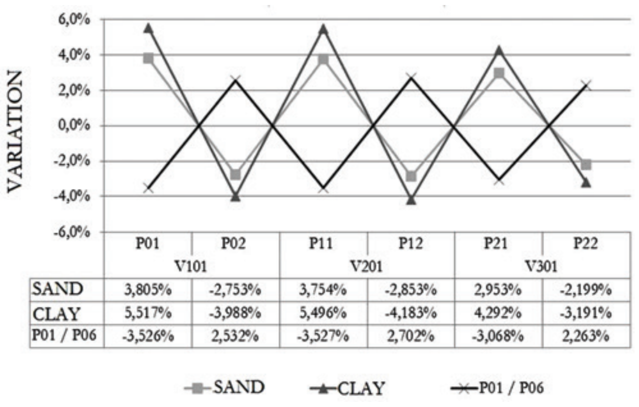
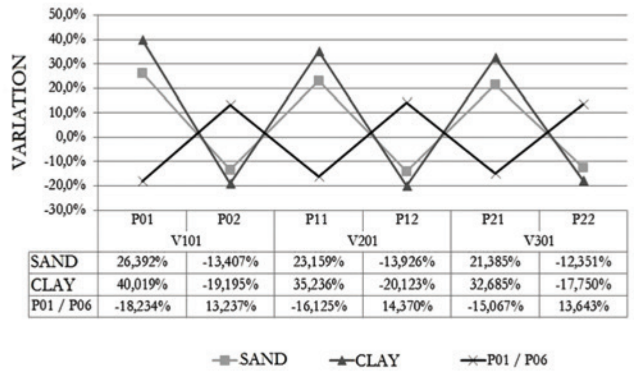


Figure 5.6 – Bending moment variation for the beams V101, V201 e V301



increasing floors, that is justified by the convergence of displacements between the rigid and flexible analysis, making variations become negligible according to the analysis of the structure moves away from the interface with the foundations.
Performing the immediate settlements analysis of the foundations model it also realizes the uniformity of settlements to the flexible support situations. The settlements values are related between flexible and rigid support, for sand, clay and soil combination of both, according to figures 5.7, 5.8 and 5.9. The base for foundation is regarded as 0.00 m quota.
The uniformity of settlements caused by the compatibility between soil and structure is observed that the differential settlements become smaller. The differential settlements obtained for the spatial model do not exceed distortions larger than 0.0352% (maximum differential settlement between P01 and P02, respective to the flexible support situation on clay).

6. Conclusions

Winkler solution, used for getting stiffness coefficients for foundations, admits the soil as an elastic, homogeneous, semi-infinite material, which responds elastically to the loading. However, it is known that the soil does not recover the original volume in the unloading of itself, due to permanent deformation of the structure. However, limiting the stress at the base of the foundation to admissible stress it is possible to consider an elastic soil response. At this contest, this rheological model was presented as a relatively simple and practical solution, due to the convergence of the results in a few iterations.
The comparative analysis of the results showed that the efforts redistribution is proportional to the rotations suffered by elements of the model.
In general, soils with lower reaction coefficient cause larger effort redistributions, forcing its compatibility according to the stiffness of the springs placed at the base of each column. However, the elastic constants used to simulate the deformation of the soil, neglect the interaction between adjacent springs, so the errors tend to grow on soft soils.
The solidarity between the structural elements was observed by uniformity of the settlements caused by the compatibility between the soil and the structure deformation, making smaller the differential settlements. Even the settlements values, do not show large differences, because they depend directly on the state of stress, which undergoes the soil, caused considerable variations in the beams and column efforts. However, to the lo-

Table 5.8 – Bending moments for the beams V101, V201 e V301

Spatial model – bending moments					
Beam		Rigid support	Flexible support		
		Infinite stiffness	Sand	Clay	Landfill P01/P06
V101	P01	10.4 kN.m	13.2 kN.m	14.6 kN.m	8.50kN.m
	P02	-76.5 kN.m	-66.3 kN.m	-61.8 kN.m	-86.7 kN.m
V201	P11	15.1 kN.m	18.6 kN.m	20.4 kN.m	12.6 kN.m
	P12	-69.9 kN.m	-60.1 kN.m	-55.8 kN.m	-79.9 kN.m
V301	P21	8.20kN.m	10.0 kN.m	10.9 kN.m	7.00kN.m
	P22	-68.9 kN.m	-60.4 kN.m	-56.7 kN.m	-78.3 kN.m

ads redistribution happen, there is a need to occur differential settlement at the supports, rotating the beams and causing the migration of the load to the neighboring columns, with smaller settlements, which would not happen if the supports had identical settlements.
It is clear that the redistribution effects are more accented at the ends of the beams than in the columns. The stiffness influence of the horizontal elements is also notable in the load redistribution, since the efforts transfer occurs through the same, so as high the stiffness of the beams is as near the structure behavior will be of a rigid block.
The variations in the efforts are higher for the members closer to the foundations, independently of the combination. It happens because of the increase of the stiffness structure with the increase on the floors, therefore a factor to be considered in the beams transition project.
In this context, it was proved that there is variation in the efforts acting on frame structures based on shallow foundations, due to the interaction between the soil and the structure, warning the importance of considering this phenomenon in situations with high concentration of normal effort, which would cause high differentials settlements, whose effects would be neglected in a conventional analysis. Therefore, even with the use of a simplified model, it is concluded that disregard the influence of the support settlements can conduct to non-realistic efforts able to harm the safety and the durability of the buildings.

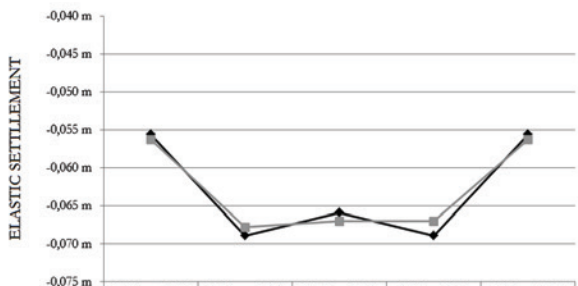


Figure 5.7 – Elastic settlement for sand

	P01 = P06	P02 = P07	P03 = P08	P04 = P09	P05 = P10
RIGID	-0,0556 m	-0,0689 m	-0,0659 m	-0,0689 m	-0,0556 m
FLEXIBLE	-0,0563 m	-0,0678 m	-0,0670 m	-0,0670 m	-0,0563 m

RIGID FLEXIBLE

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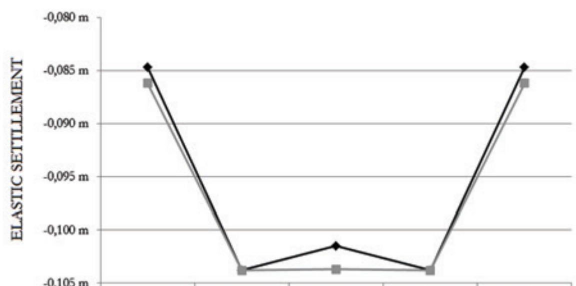

Figure 5.8 – Elastic settlement for clay

	P01 = P06	P02 = P07	P03 = P08	P04 = P09	P05 = P10
RIGID	-0,0847 m	-0,1038 m	-0,1015 m	-0,1038 m	-0,0847 m
FLEXIBLE	-0,0862 m	-0,1038 m	-0,1037 m	-0,1038 m	-0,0862 m

RIGID FLEXIBLE

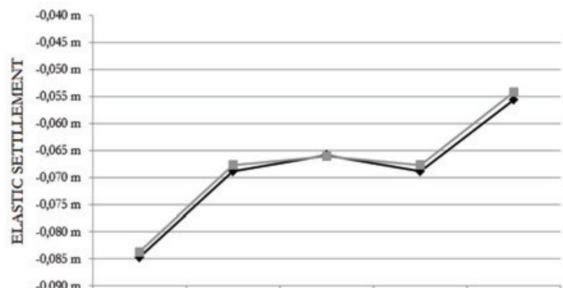

Figure 5.9 – Elastic settlement for sandy and clay soil combination

	P01 = P06	P02 = P07	P03 = P08	P04 = P09	P05 = P10
RIGID	-0,0847 m	-0,0689 m	-0,0659 m	-0,0689 m	-0,0556 m
FLEXIBLE	-0,0837 m	-0,0677 m	-0,0661 m	-0,0677 m	-0,0542 m

RIGID FLEXIBLE

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