The Influence of the Foundation Settlements on the Column Loads of a Building

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Abstract. The loads on columns related to foundation settlements of a building localized in the city of Rio de Janeiro have been analysed. Settlements and strains in some columns have been measured from the beginning of construction. The structural behaviour was simulated with the finite element method with a model for each building stage related to the measurements. The loads evaluated considering no foundation settlements have been compared to the loads obtained with the measured settlements as prescribed displacements. The loadings thus obtained were also compared to those estimated by the columns strains.

Key words: foundation, soil-structure interaction, settlement measurement, strain measurement.

1. Introduction

The design of the structure and the foundation of a building are generally independently performed (*e.g.* Gusmão, 1990). Therefore, the soil-structure interaction is not considered. In general, there is a load transfer from the columns that have the trend to have higher settlements to those with smaller settlements. Thus there is a trend of uniformization of the settlements. This subject has been studied by a number of researchers, *e.g.* Meyerhof (1953), Chamecki (1954), Goschy (1978), Gusmão (1990), Gusmão and Gusmão Filho (1994a, 1994b), Gusmão Filho (1998), Moura (1995), Aoki (1997), Danziger *et al.* (1997), Santa Maria *et al.* (1999) and Soares (2004).

The present paper analyses the column loads of a building considering two situations. In the first one the foundations are assumed to have no settlements, which is the usual assumption in the design of a structure. The second one takes into account the settlements that have been measured from the beginning of construction. In both cases the structure was analysed with the use of the finite element method. The analysed models correspond to each available set of measurements.

Since the strains in columns have also been measured from the beginning of construction, a comparison between the loads estimated from the strains and from the finite element analysis is also made.

The analysed building is one out of nine instrumented buildings included in a research cooperation among COPPE/UFRJ, UFF and building contractor Construtora Ben.

2. The Building

2.1. General characteristics

The analysed building, designated SFA, is situated in Recreio dos Bandeirantes, west zone of the Rio de Janeiro city, and it is typical from this huge area where the city of Rio de Janeiro is growing towards. It is a reinforced concrete building, with one access floor, two similar floors, the penthouse, as well as an elevated water tank. Verandas in cantilever are present in the front of the building (Fig. 1).

There are 21 columns arriving at the ground level, and design loads vary from 220 kN to 1960 kN. Footings have been used, installed at a depth 1.5 m below the ground level. An average allowable soil stress of 200 kPa was

Figure 1 - View of SFA building.

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adopted. Fig. 2 shows a plan with the columns location and loads.

2.2. Soil characteristics

Standard penetration tests (SPT's) have been the only soil test used to characterize the soil, and the obtained profiles are presented in Fig. 3. Fine to medium sand, from loose to dense, mostly grey but in some layers brown, is found from the ground surface to 20 m depth. Organic clay layers, grey, soft and medium, as well as a hard silty clay layer are found interbedded with sand layers in the range 20 m - 26 m depth. The characteristics of those sand layers are similar to the layers found in the upper part of the profile.

3. Settlements and Strains Measurements

3.1. Procedure used to measure the settlements

Settlements have been measured from March 1993 until February 1996. Corrosion-free seats have been installed in some columns, at a height of around 1 m above the ground level.

Due to the high cost to install a bench-mark, corrosion-free seats have been installed outside the building, in places where the influence of the building is assumed to be negligible. Optical leveling has been used to evaluate the settlements. In every set of measurements the obtained values have been checked against different external reference values. Other details, both from the procedure and the equipment used, can be found in Danziger *et al*. (1995, 1997, 2000) and Gonçalves (2004).

3.2. Procedure used to measure the strains

Strains have been measured in a shorter period than the settlements, from March 1993 until January 1994. The experience associated with the procedure used comes from the 1970's, when Soares (1978) and Soares and Carim (1978) used it to evaluate the strut loads in the Rio de Janeiro Subway. Two pins, 250 mm vertically apart, have been installed in the middle section of the columns. Dents have been punched in the pins, in order to provide the "perfect" suitable fitting for a mechanical extensometer. The Huggenberg extensometer, which consists of an internal rod moving inside a tube coupled with an extensometer able to measure 0.001 mm, has been used (Fig. 4). The extensometer measures the length variation between the 2 reference pins. From this measurement, the strain values are obtained. Values of strain have been obtained in the four faces of columns C11 and C17, as well as in two parallel faces of columns C10 and C15. Further details can be obtained in Gonçalves (2004).

4. The Evaluation of Column Loads from the Strain Values

The strain values have been measured aiming at the evaluation of the column loads in different stages of the

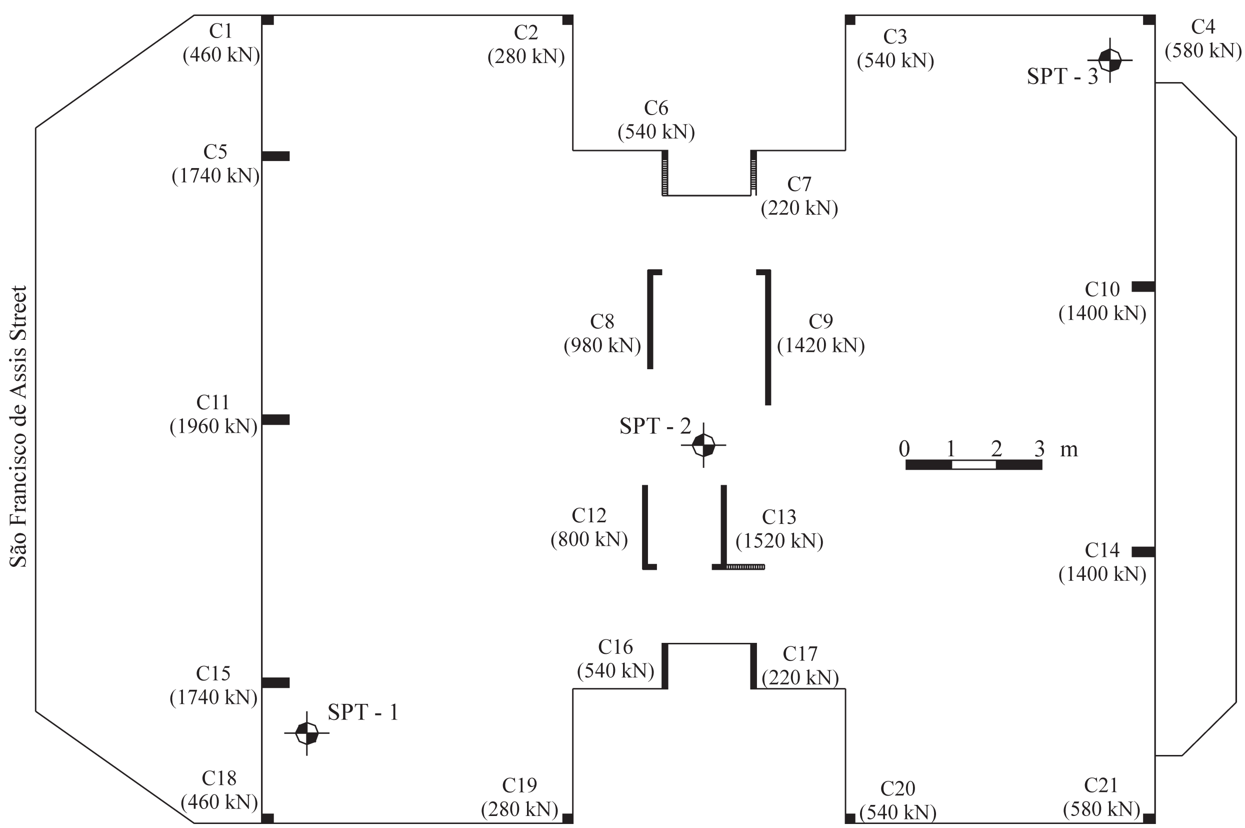

Figure 2 - Location of columns and SPT's performed (Gonçalves *et al*., 2004).

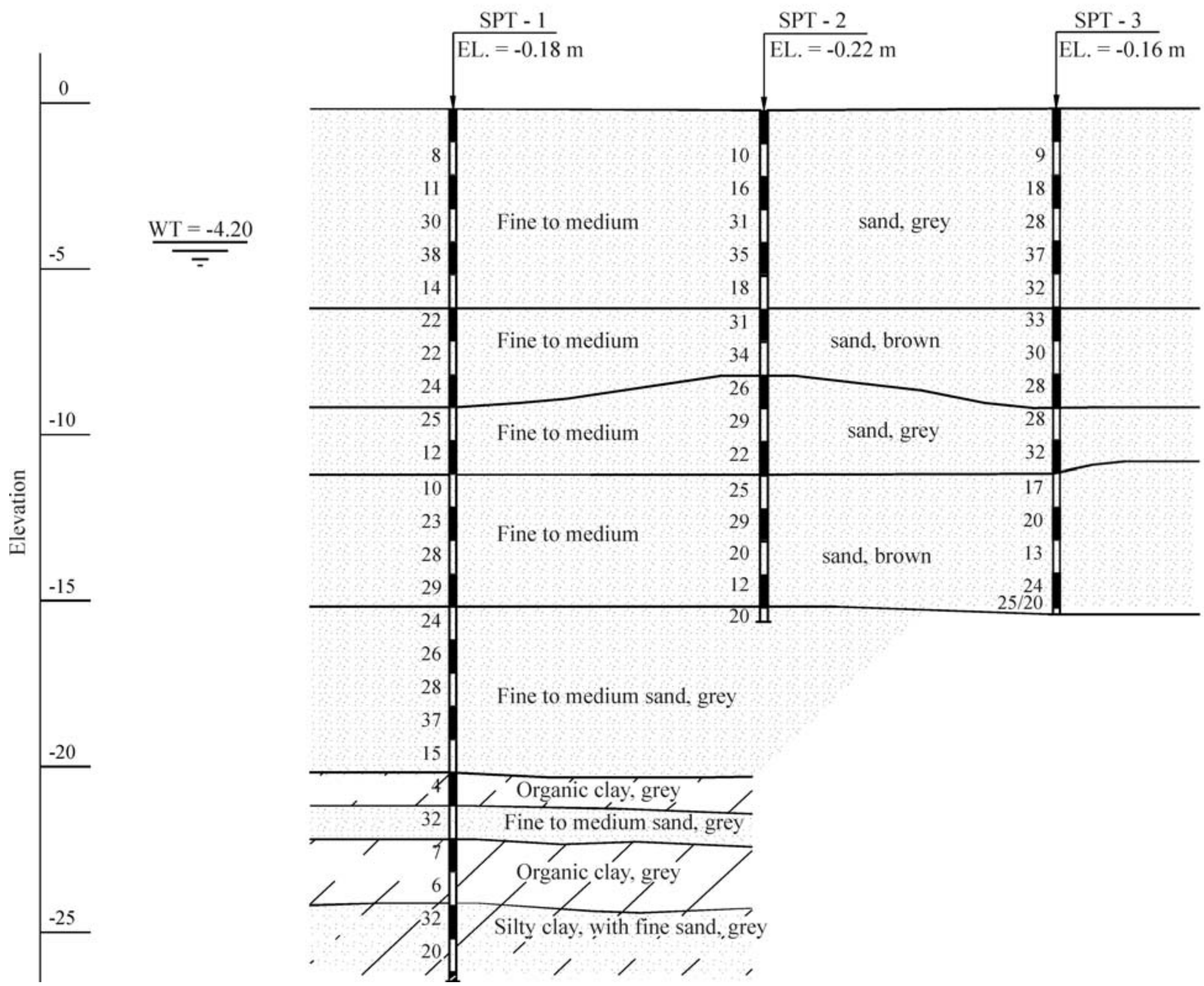

Figure 3 - Soil profile.

building construction. However, such evaluation is not straightforward, since the strains are influenced not only by the column loads, but also by concrete creep, shrinkage and thermal strain. Such values must therefore be estimated.

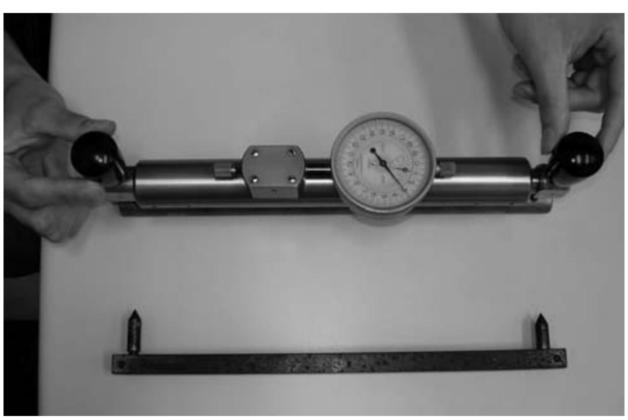

Figure 4 - Mechanical extensometer Huggenberg used to measure the strains in the columns.

According to the CEB-FIP Model Code (1990), the total strain at time t, $\varepsilon_c(t)$, of a concrete member, uniaxially loaded at time t_0 with a constant stress $\sigma_c(t_0)$, may be expressed as

$$\varepsilon_c(t) = \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t) + \varepsilon_{cs}(t) + \varepsilon_{cT}(t) \tag{1}$$

or

$$\varepsilon_c(t) = \varepsilon_{c\sigma}(t) + \varepsilon_{cn}(t) \tag{2}$$

where $\varepsilon_{ci}(t_0)$ = initial strain at loading; $\varepsilon_{cc}(t)$ = creep strain at time $t > t_0$; $\varepsilon_{cs}(t)$ = shrinkage strain; $\varepsilon_{cT}(t)$ = thermal strain; $\varepsilon_{c\sigma}(t) = \varepsilon_{ci}(t) + \varepsilon_{cc}(t)$, stress dependent strain; $\varepsilon_{cn}(t) = \varepsilon_{cs}(t) + \varepsilon_{cT}(t)$, stress independent strain.

For stresses and strains varying with time, assuming the validity of the superposition principle, one can obtain

$$\varepsilon_c(t) = \sigma_c(t_0) J(t, t_0) + \int_{t_0}^{t} J(t, \tau) \frac{\partial \sigma_c(\tau)}{\partial \tau} d\tau + \varepsilon_{cn}(t) \tag{3}$$

where $\sigma_c(t_0)$ = initial stress; $J(t, \tau)$ = creep function or creep compliance: $J(t,\tau)=\left[\frac{1}{E(\tau)}+\frac{\phi(t,\tau)}{E_{ci}}\right]$; $E(\tau)$ = modulus of elasticity of the concrete at the time of application of the load increase; τ = dummy time variable; $\phi(t, \tau)$ = creep coefficient; E_{ci} = modulus of elasticity of the concrete at the age of 28 days; $\frac{\partial\sigma_c(\tau)}{\partial\tau}d\tau$ = infinitesimal increment of stress.

4.1. Estimating the concrete creep, shrinkage and thermal strain according to the CEB-FIP Model Code (1990)

The equations used to estimate the concrete creep, shrinkage and thermal strain, according to the CEB-FIP Model Code (1990), are valid for concrete structures (12 MPa < f_{ck} ≤ 80 MPa) subjected to a compression stress $|\sigma_c| < 0.4 f_{cm}(t_0)$ at age of loading t_0 and exposed to mean relative humidity between 40 and 100% and mean temperature between 5 and 30 °C, where f_{ck} = characteristic compressive strength of concrete; f_{cm} = mean compressive strength of concrete at the age of 28 days.

The thermal strain has not been considered in the analysis because the concrete temperature was not measured. It is believed that this strain has not been significant with respect to the others, due to the geometric configuration of the columns in the building. Russo Neto (2005), in a similar analysis for a precast concrete structure with precast concrete piles in the city of Curitiba, did obtain a significant influence of the temperature on the measured strain values. However, the geometric positioning of the columns in this building was rather different from the one analysed in the present paper. Moreover, the column cross-section of the Russo Neto (2005) building was different from the ones analysed herein.

4.1.1. Concrete creep

The creep coefficient can be estimated from Eq. (4),

$$\phi(t,\tau)=\phi_0\,\beta_c(t-\tau) \tag{4}$$

where ϕ_0 = notional creep coefficient, depending on the relative humidity of the ambient environment, on the section homogenized, on its perimeter, and on the mean compressive concrete strength at the age of 28 days; $\beta_c(t - \tau)$ = coefficient describing the development of creep with time after loading; t = concrete age (days) at the moment considered; τ = concrete age (days) at loading.

The CEB-FIP Model Code (1990) presents the equations necessary to estimate ϕ_0 and $\beta_c(t - \tau)$.

The creep strain occurring in a reinforced concrete member is smaller than the one in a concrete member, since in a reinforced concrete member there is a load transfer from the concrete to the steel throughout the time. Because of that, the concrete strain in the present analysis was corrected according to an analysis carried out by Santa Maria (1997), since the homogenized area had been used in the calculations.

4.1.2. Concrete shrinkage

The strain due to shrinkage can be estimated from Eq. (5)

$$\varepsilon_{cs}(t,t_s)=\varepsilon_{cso}\beta_s(t-t_s) \tag{5}$$

where ε_{cso} = notional shrinkage coefficient, depending on the type of cement, the mean compressive concrete strength at the age of 28 days, and on the relative humidity of the ambient environment; $\beta_s(t - t_s)$ = coefficient describing the development of shrinkage with time; t = concrete age (days) at the time considered; t_s = concrete age (days) at the beginning of shrinkage.

The CEB-FIP Model Code (1990) presents the equations necessary to estimate ε_{cso} and $\beta_s(t - t_s)$.

5. Soil-Structure Interaction

5.1. Numerical model of the building structure

Five 3D finite element models of the framed structure, presented in Figs. 5 to 7, have been developed, corresponding to each available series of settlement and strain measurements (named stages), as shown in Table 1. Figure 7 represents the 3 last series of measurements (stages), for which the differences are related to the applied loads after the completion of the concrete structure.

Frame elements have been used to discretize beams and columns. However, the central wall-columns C8, C9, C12 and C13 have been simulated as shell elements due to their high stiffness. Shell elements have also been used to discretize the slabs. An elastic behaviour was assumed for

Figure 5 - Numerical model corresponding to 1st stage (Gonçalves, 2004).

Figure 6 - Numerical model corresponding to 2nd stage (Gonçalves, 2004).

Figure 7 - Numerical model corresponding to 3rd, 4th and 5th stages (Gonçalves, 2004).

the whole structure. All analyses have been performed with the program SAP 2000 (1996).

5.2. Column loads in different hypotheses

The column loads have been estimated for two situations regarding the foundation settlements. In the first one the foundations are assumed to present no settlements, which is the usual assumption in the design of a structure. The second one takes into account the settlements that have been measured from the beginning of construction. Once the measurement of settlements have not been performed in all columns, settlement values have been adopted considering the symmetry observed in the structure with respect to an axis at right angle to the street. Moreover, the soil has been assumed constituted by homogeneous layers. Measured and adopted settlement values are included in Table 2.

Table 1 - Numerical models and building stages (Gonçalves, 2004).

Model	Date*	Building stage
1st stage	17/5/1993	1st floor structure concreted without the front cantilevers
2nd stage	17/8/1993	2nd floor structure concreted
3rd stage	26/1/1994	Structure and brick walls concluded
4th stage	3/8/1995	Whole structure
5th stage	7/2/1996	Building in use

*Reference (zero) readings taken in 31/3/1993.

6. Analysis of the Results

6.1. Comparison of column loads obtained for the hypotheses of no settlements and measured settlements

The column loads obtained from the finite element analysis with the no settlements hypothesis are presented in Table 3, for each construction stage. The ratio between the loads obtained for the 5th stage and the design loads is also included in the table. The design loads have been obtained by calculating separately slabs, beams and columns, disregarding the actual interaction among these members. The lower part of the table contains the ratio of load (in percentage) in each stage with respect to the 5th stage.

The ratio total load (including the loads of all columns) of 5th stage and the design load is 89%, *i.e.* did not reach 100%. Minor simplifications in the model (the non-consideration of the water load in the elevated water tank, as well as the load of the elevators machinery) do not justify such difference.

The last column of Table 3 presents the difference (in percentage), for each column, between the ratio 5th stage load/design load and 0.89, the average ratio for all columns. It can be observed that the differences have been quite significant. In fact, a value as high as 30% was obtained, which was attributed to the non conventional building structure. Thus, this kind of structure suggests the need for using of more refined design methods (like e.g. the finite element method) than the commonly used. High differences between design loads obtained from the usual method and the loads obtained by the finite element method (in the range +58% to -45%) have been obtained by Costa (2003) for a similar structure.

The influence of the settlements on the column loads is illustrated in the comparison included in Table 4. The column loads in Table 4 have been obtained in both hypotheses (no settlements and measured settlements) from the analyses performed with the finite element method. The differences between the column loads for both hypotheses (in percentage), which can be considered an indication of

Table 2 - Measured and adopted settlement values (Gonçalves, 2004).

Column	Settlement (mm)				
	1st stage 17/5/1993	2nd stage 17/8/1993	3rd stage 26/1/1994	4th stage 3/8/1995	5th stage 7/2/1996
C1	0.36*	1.02*	1.94	3.23*	5.10*
C2	0.45	1.08	2.05	3.49	4.89
C3	0.58*	1.27*	2.41	4.58*	6.41
C4	0.65	1.72	3.00	6.63	7.65
C5	0.73	1.71	2.96	4.41	6.43
C6	0.50	1.20	2.28	3.88	5.00
C7	0.38	0.91	1.73	2.94	3.75
C8	0.72*	1.59*	3.03*	5.26*	7.21
C9	0.48*	1.40*	3.12*	5.21*	7.14
C10	0.98*	2.32*	3.67*	6.02*	7.56*
C11	0.73*	1.75*	3.12*	4.94*	6.95*
C12	0.56*	1.21*	2.64*	4.60	7.21
C13	0.48	1.40	3.12	5.21	7.14
C14	0.98	2.32	3.67	6.02	7.56
C15	0.73*	1.71*	2.96*	4.41*	6.43*
C16	0.50	1.20	2.28	3.88	5.00
C17	0.38	0.91	1.73	2.94	3.75
C18	0.36	1.02	1.94	3.23	5.10
C19	0.45	1.08	2.05	3.49	4.89
C20	0.58	1.27	2.41	4.58	6.41
C21	0.65*	1.72*	3.00*	6.63*	7.65*

*Measured values.

the soil-structure interaction, are also included in the table. The averages of such differences are shown in the lower part of the table.

Figures 8 and 9 present the mentioned difference as a function of time, where the date of reference (zero) readings (31st March 1993) has been considered as time equal to 0. Figure 8 contains the columns that presented a load increase, and Fig. 9 a load decrease at least for some period, with respect to the no settlement hypothesis. It is worth emphasizing that since the structure has been modelled as an elastic structure, time is only associated with load variation.

From Table 4 and Figs. 8 and 9 it can be observed that 11 columns have presented small load differences (smaller than 5%) with respect to the no settlements hypothesis. These are C2, C3, C4, C5, C10, C11, C14, C15, C19, C20 and C21. The columns C2, C3, C4, C5 and C10 are symmetrically located with regard to columns C19, C20, C21, C15 and C14, respectively. The columns C5, C11 and C15 are located in the frontal part of the building, very much influenced by the cantilever (5 m), and have the highest design loads: C5 and C15, 1740 kN, and C11, 1960 kN. All these columns, despite of the particular situation of C5, C11 and C15 are peripheral columns.

The column C13 could have been included in the same previous situation (with respect to the no settlements hypothesis smaller than 5%), except for the 2nd stage, where a difference of 9% was obtained. This value is discussed afterwards.

The other columns have presented higher differences with respect to the no settlement hypothesis. The columns C1 and C18, which are symmetrically located in the frontal part of the building (at the corners), have always shown differences higher than 5% (C1 higher than 10%), as it would be expected. In fact, it is usual a load transfer from the internal columns to the external columns, or in other words, a load increase in the external columns and a load decrease in the internal columns with respect to the no settlements hypothesis.

The columns C8, C9 and C12 have shown a load increase with respect to the no settlement hypothesis, differently from the expected behaviour. Besides, all those columns have shown a trend of an increase of the soil-structure interaction with time. It is worth mentioning that settle-

Table 3 - Column loads for the hypothesis of no foundation settlements (Gonçalves, 2004).

Column	Design load (kN)	1st stage (kN)	2nd stage (kN)	3rd stage (kN)	4th stage (kN)	5th stage (kN)	Load 5th stage/ Design load	Difference* with respect to 89%
C1	460	19	50	189	253	274	0.60	0.29
C2	280	30	38	161	203	212	0.76	0.13
C3	540	53	97	346	445	501	0.93	-0.04
C4	580	52	89	304	384	432	0.74	0.15
C5	1740	110	407	1089	1441	1672	0.96	-0.07
C6	540	55	87	304	390	438	0.81	0.08
C7	220	19	42	172	219	244	1.11	-0.22
C8	980	108	166	554	708	816	0.83	0.06
C9	1420	139	254	771	963	1137	0.80	0.09
C10	1400	166	322	817	1025	1221	0.87	0.02
C11	1960	158	504	1372	1861	2169	1.11	-0.22
C12	800	92	161	489	628	717	0.90	-0.01
C13	1520	110	178	865	1103	1275	0.84	0.05
C14	1400	164	319	799	1006	1199	0.86	0.03
C15	1740	122	415	1081	1459	1686	0.97	-0.08
C16	540	79	113	343	446	498	0.92	-0.03
C17	220	25	41	195	235	262	1.19	-0.30
C18	460	19	53	200	270	293	0.64	0.25
C19	280	32	40	169	219	230	0.82	0.07
C20	540	44	88	330	427	481	0.89	0.00
C21	580	52	90	307	390	439	0.76	0.13
Σ	18200	1648	3554	10857	14075	16196	average	0.89
Percentage with respect to the 5th stage		10	22	67	87	100		

*The negative sign indicates that the value corresponding to the 5th stage was greater than 89% of the design load.

ments have not been measured in the column C12, which is the one presenting the highest load increase, especially for the 4th and 5th stages. The adopted settlement values may have been overestimated, since they have been obtained from the increase rate of settlements of column C8, due to its similarity with C12.

The columns C6, C7, C16 and C17 have shown load decrease with time with respect to the no settlement hypothesis, and this trend increased with time, also depicting the soil-structure interaction influence with time.

It can be observed that in the inner part of the building the columns C6, C7, C8, C9 C12, C16 and C17 have been the most affected by the structure stiffness increasing with time.

This behaviour has been attributed to the particular characteristics of the structure, which has different floors, central columns with high stiffness and, especially, large cantilevers (5 m) corresponding to the veranda, which have produced higher loads in the frontal columns, mainly C5, C11 and C15, than the internal loads, differently from regular buildings, where higher loads are found in the central columns.

Some columns (C1, C2, C6, C8, C12, C13, C16, C18 and C19) have presented a significant variation of their behaviour in the 2nd measurement with respect to the other series of measurements. This is probably related to a special construction aspect, the removal of the shoring of the cantilever slab from the first to the second stages.

It was also found that the load redistribution throughout the time, which can be represented by the average of load redistributions of all columns, was small (3%) only in the first series of measurements (see Table 4). In the others stages, this value was about 7%. In other words, the first series of measurements would be the only one showing a stiffness smaller than the others.

It is worth emphasizing that the differences of column loads for the two structural models analysed (the procedure disregarding the interaction among the structural members and the finite element method) were higher than the load differences obtained when the hypotheses of no settlements

Table 4 - Column loads for two hypotheses: no settlements and measured settlements (Gonçalves, 2004).

Column	Design load (kN)	1st stage (kN) settlements	2nd stage (kN) settlements	3rd stage (kN) settlements	4th stage (kN) settlements	5th stage (kN) settlements
		meas. no	meas. no	meas. no	meas. no	meas. no
		Difference	Difference	Difference	Difference	Difference
C1*	460	21-19	62-50	213-189	279-253	303-274
		11%	24%	13%	10%	11%
C2	280	30-30	39-38	163-161	207-203	216-212
		0%	3%	1%	2%	2%
C3*	540	52-53	96-97	341-346	435-445	477-501
		-2%	-1%	-1%	-2%	-5%
C4	580	52-52	91-89	308-304	380-384	435-432
		0%	2%	1%	-1%	1%
C5	1740	107-110	391-407	1054-1089	1407-1441	1630-1672
		-3%	-4%	-3%	-2%	-3%
C6	540	53-55	78-87	274-304	338-390	374-438
		-4%	-10%	-10%	-13%	-15%
C7	220	17-19	37-42	153-172	191-219	214-244
		-11%	-12%	-11%	-13%	-12%
C8*	980	110-108	177-166	590-554	765-708	895-816
		2%	7%	6%	8%	10%
C9*	1420	142-139	267-254	818-771	1045-963	1237-1137
		2%	5%	6%	9%	9%
C10*	1400	165-166	317-322	805-817	1015-1025	1204-1221
		-1%	-2%	-1%	-1%	-1%
C11*	1960	158-158	503-504	1362-1372	1828-1861	2129-2169
		0%	0%	-1%	-2%	-2%
C12*	800	100-92	193-161	587-489	787-628	919-717
		9%	20%	20%	25%	28%
C13	1520	106-110	162-178	840-865	1067-1103	1223-1275
		-4%	-9%	-3%	-3%	-4%
C14	1400	164-164	315-319	788-799	995-1006	1182-1199
		0%	-1%	-1%	-1%	-1%
C15*	1740	120-122	403-415	1055-1081	1434-1459	1654-1686
		-2%	-3%	-2%	-2%	-2%
C16	540	75-79	97-113	301-343	372-446	409-498
		-5%	-14%	-12%	-17%	-18%
C17	220	25-25	40-41	168-195	193-235	215-262
		0%	-2%	-14%	-18%	-18%
C18	460	20-19	61-53	218-200	288-270	314-293
		5%	15%	9%	7%	7%
C19	280	32-32	42-40	173-169	226-219	238-230
		0%	5%	2%	3%	3%
C20	540	43-44	88-88	328-330	423-427	467-481
		-2%	0%	-1%	-1%	-3%
C21*	580	52-52	91-90	310-307	386-390	439-439
		0%	1%	1%	-1%	0%
Average**		3%	7%	6%	7%	7%

*Columns with settlement measurement; **Average of absolute values.

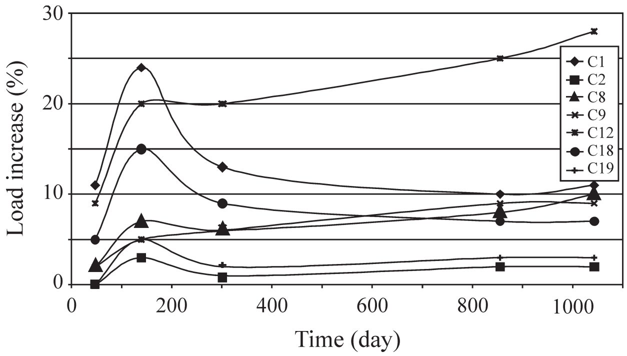

Figure 8 - Columns with load increase (%) with time.

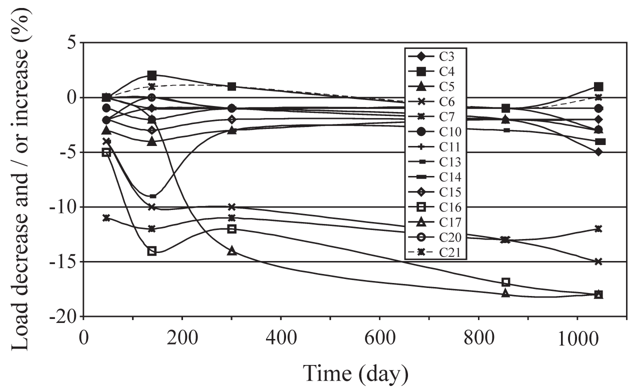

Figure 9 - Columns with load decrease (%), at least during a period, with time.

and measured settlements have been compared, but the same structural model (the finite element model) was used. This has been attributed to the particular features of the building, as previously mentioned, and also to the small measured settlements used in the analysis.

6.2. Comparison of the loads estimated from the strain measurements with the ones obtained from the finite element analysis

The average values of strain measured on the column faces are included in Table 5. The loads obtained from the strain values (*N*), taking into account the strains due to creep and shrinkage, as previously shown, are compared with the loads obtained from the finite element analysis, considering the structure subjected to the measured settlements (*Nprog*), in Table 6 and Fig. 10. The ratio between those values is also included in the table.

From both Table 6 and Fig. 10 one can observe that a good agreement between the loads is obtained only in the first stage of column C10. A trend of higher *N* values than

Table 5 - Strains measured (Gonçalves, 2004).

Dates	Days	Average strain			
		C10	C11	C15	C17
31/3/93 to 17/5/93	47	1.60 E -04	1.06 E -04	1.58 E -04	8.10 E -05
31/3/93 to 17/8/93	139	2.87 E -04	2.48 E -04	2.85 E -04	8.50 E -05
31/3/93 to 26/1/94	301	2.87 E -04	-	3.26 E -04	1.77 E -04

Table 6 - Column loads as obtained from the measured strain values (*N*) and from the finite element analysis (*Nprog*) (Gonçalves, 2004).

	C10		
Model	*N* (kN)	*Nprog* (kN)	*N/Nprog*
1st stage	168	165	1.02
2nd stage	254	317	0.80
3rd stage	218	805	0.27
	C11		
Model	*N* (kN)	*Nprog* (kN)	*N/Nprog*
1st stage	130	158	0.82
2nd stage	344	503	0.68
	C15		
Model	*N* (kN)	*Nprog* (kN)	*N/Nprog*
1st stage	196	120	1.63
2nd stage	319	403	0.79
3rd stage	318	1055	0.30
	C17		
Model	*N* (kN)	*Nprog* (kN)	*N/Nprog*
1st stage	49	25	1.96
2nd stage	-	40	-
3rd stage	-	168	-

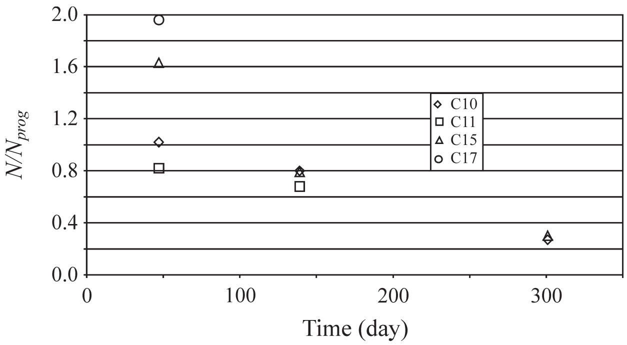

Figure 10 - Ratio between the column loads obtained from the measured strains and the loads from the finite element analysis versus time (Gonçalves, 2004).

Nprog values was obtained in the first stage while the opposite was found as time progresses.

For the second and third stages, although *Nprog* could have been overestimated by the finite element analysis, it is believed that the strain due to concrete shrinkage (or even the concrete creep) has been overestimated.

It was not possible to estimate the *N* values for the 2nd and 3rd stages in the case of C17, since the estimation of the strain due to concrete shrinkage was higher than the measured strain. This was due to the shape and perimeter of the column section (12 cm x 110 cm), which is very different from the other columns (C10, 20 cm x 50 cm, C15, 20 cm x 60 cm, C11, 20 cm x 70 cm), resulting in the estimation of high values of the strain due to shrinkage.

7. Conclusions

• The differences of column loads of the finite element analysis and the design loads have reached 30%, which was attributed to the non conventional building structure. Thus, this kind of structure suggests the need of use of more refined design methods (like *e.g.* the finite element method) than the commonly used to design building structures.

• The soil-structure interaction has been evaluated comparing two hypotheses: (i) no foundation settlements and (ii) measured settlements as input for the finite element analysis of the models developed. Eleven columns have shown differences between both hypotheses less than 5%. Some columns have shown differences higher than 5% differently from what it would be expected, the usual load transfer from the internal columns to the external columns, or in other words, a load increase in the external columns and a load decrease in the internal columns with respect to the no settlements hypothesis. This behaviour has been attributed to the particular characteristics of the structure, which has different floors, central columns with high stiffness and, especially, large cantilevers (5 m) corresponding to the veranda, which have produced higher loads in the frontal columns, especially C5, C11 and C15, than the internal loads, differently from regular buildings, where higher loads are found in the central columns.

• Some columns (C1, C2, C6, C8, C12, C13, C16, C18 and C19) have presented a significant variation of their behaviour in the 2nd stage with respect to the other stages. This has been attributed to a special construction aspect, the removal of the shoring of the cantilever slab from the first to the second stages.

• It was found that the load redistribution throughout the time, which can be represented by the average of load redistributions of all columns, was small (3%) only in the first stage. In the others stages, this value was about 7%. In other words, the first stage would be the only one showing a stiffness smaller than the others.

• The differences of column loads for the two structural models analysed (the procedure disregarding the interaction among the structural members and the finite element analysis) were higher than the load differences obtained when the hypotheses of no settlements and measured settlements have been compared, but the same structural model (the finite element model) was used. This has been attributed to the characteristics of the building and also to the small measured settlements used in the analysis.

• The strains in columns - aiming at the evaluation of the column loads - have been measured from the beginning of construction. The loads obtained from the strain values measured (*N*), taking into account the strains due to creep and shrinkage, have been compared with the loads obtained from the finite element analysis, considering the structure subjected to the measured settlements (*Nprog*). A good agreement between these loads is obtained only in the first stage of column C10. A trend of higher *N* values than *Nprog* values was obtained in the first stage while the opposite was found as time progresses. It is believed that the strain due to concrete shrinkage (or even the concrete creep) has been overestimated. There is an urgent need of measurements of strain in columns from the beginning of construction, as well as the improvement of the ability to predict the concrete strains due to creep and shrinkage in reinforced concrete columns of different shapes in order to properly evaluate the column loads in buildings.

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