Technical University of Civil Engineering Bucharest
Bd. Lacul Tei nr. 122 - 124, cod 020396, Sector 2, București
Fundamental field: Engineering sciences
Doctoral Field: Civil Engineerind and Installations
Specialization: Civil Constructioon
Doctoral School – Year II, Semester II: 2016-2017

Report 1

--- Aspects regarding the modeling of bearings yield in the study of the behavior of structures---

Scientific leader,
Professor Ramiro Sofronie
UNESCO CHAIR #177, Bucharest

Phd student,
Ing. Cristian Mihai IRIMIA

Bucharest
2016-2017

Contents:

Cap.1 Introduction

The objective of this research is to investigate the behavior of reinforced concrete structures in which the effect of differentiated settlement occurs. Differentiated settlement in the structure is important because the relative pressing of the foundation often leads to its damage. The impact of differentiated settlement in structure has been widely investigated since the late 1940s (Meyerhof, 1947, Chamecki, 1956, Skempton and Macdonald, 1956, Polshin and Tokar, 1957, Jennings and Kerrich, 1962, Brown, 1969a, 1969b, Grant *et al.*, 1974, Burland and Wroth, 1975, Burland *et al.*, 1977, Jardine *et al.*, 1986, Boscarding and Cording, 1989, Boone, 1996, Potts and Addenbrooke, 1997, Potts *et al.*, 1998, Burland *et al.*, 2001).

Although much researched the problem of relative settlement, progress has been hampered by:

- Lack of rigorous methods of describing the foundation's
- Lack of rigorous methods of describing the type of structure,
- Lack of rigorous methods of description of structural damage.
- The different methods used in estimating relative settlement.

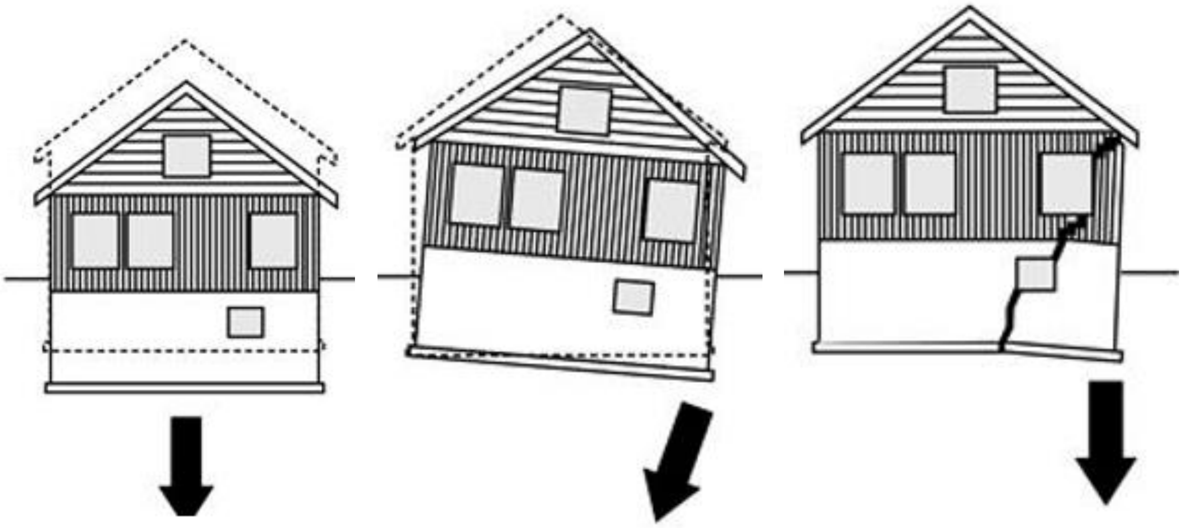
The following discussion points will be:

- Description of foundation deformation
- Type of structure
- Description of damage
- Methods used to estimate settlement.

To define the movement of the Terzaghi Foundation (1935) stated that for an accurate description of the movement of the foundation it takes a minimum of 15 points arranged over the entire area occupied by the structure in which to make measurements. This will result in a 3D representation of the deformation of the structure. Skempton and MacDonald (1956) used the simplified 2D approach and defined angular distortion as the ratio between differentiated settlement and the distance between two points outside the area of influence of the settlement. Polshin and Tokar (1957) defined the slope as the difference between two points relative to the distance between them. In 1974 Burland and Wroth suggested a set of definitions to define relative settlement in the 2D model. These definitions are limited because they only describe plane movement (2D) and are useful for describing 2D frame behavior and 3D building behavior with minimal lateral deformation. However, structures with a regular shape in the process of relatively self-tapping will have important deformations in the corners, which are difficult to describe in 2D. Research has shown that a description in the plan is not sufficient, and a spatial description is also required. The type of structure influences the response of the differentiated settlement.

There are 3 types of settlement as follows:

- Uniform settlement in which the settlements under the foundation will be uniform. Uniform settlement will not produce efforts in the structure;
- Relative settlement in which the settlements will differ from point to point. Relative settlement can produce major cracks in the structure;
- The tilt of the foundation. It occurs when the structure is rigid, and the deformation is a rigid body. Tilting the foundation often will not produce cracks in the structure;



a) Uniform settlement b) Tilt of the foundation c) Relative settlement

Figure 1. 1. Types of construction
Source: www.civilengineeringdiscoveries.wordpress.com

Cap.2 Ground-structure interaction

Side and gravitational actions, applied to the structure, lead to a state of tension and deformation in the ground. At a certain distance from the foundation, the deformations of the land will be very small (fig.3,1). The land in the active area will not influence the behavior of the structure as a whole, it can be modeled in calculations as rigid.

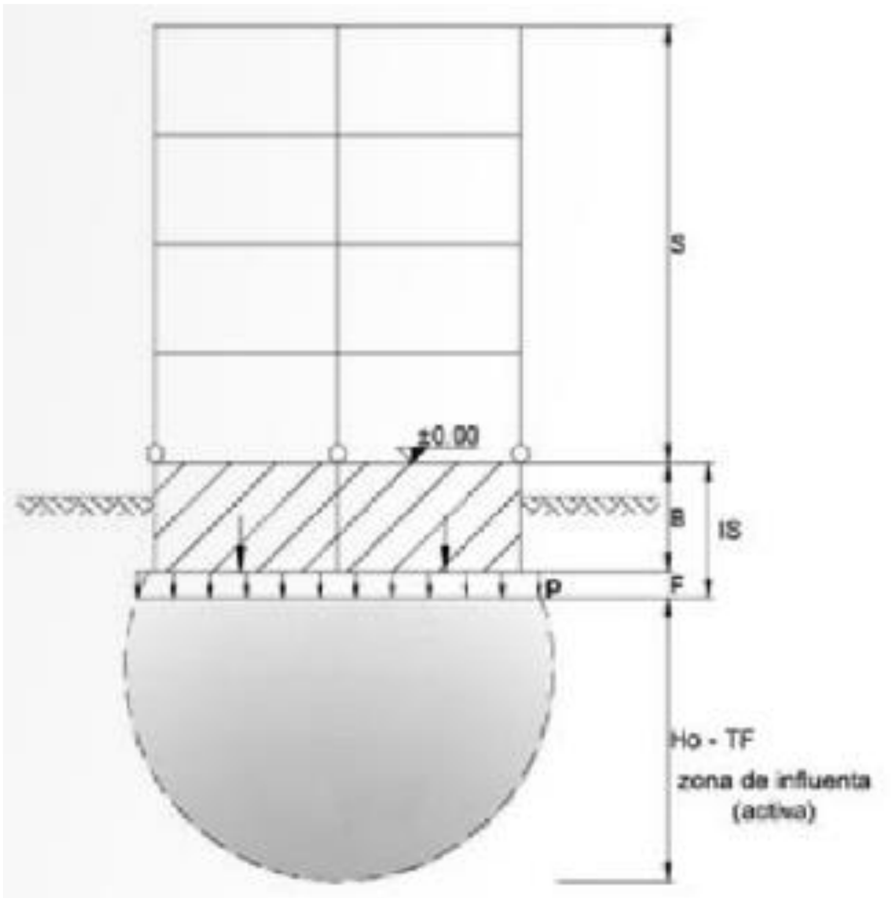


Figure 3. 1. Active area in the foundation ground
Source: http://issuu.com/revistaconstructiilor/docs/rc_nr_99_decembrie_2013

In Figure 3.1 the deformable part of the land under the foundation and the region nearby can be idealized as rigid, called A and B. the boundary between the two regions can be determined based on an analysis of efforts, region A can be modeled as part of the structural system.

During the action of external loads, the following phenomena occur in the earth:

- reciprocal movements of aggregates or earth granules;
- destruction of the aggregate structure or particles;
- removal of water from the pores of the earth;
- deformation of the adsorbed water film;
- compression and partial dissolution of closed air bubbles in the pores of the earth;
- deformations of the earth's particles.

Some simplified hypotheses are made for the modeling and calculation of the earth.

One of the hypotheses relates to the behavior of the earth and concerns its continuity. Depending on this assumption, a relationship may be established between the burdens on the earth and the movements that occur. Based on the relationship, the deformation module of the entire earth mass is highlighted.

The interaction between the ground and the construction is materialized by writing the contact conditions between the two subsystems. The displacement of a point of construction shall be equal to the displacement of the point of contact of the land, throughout the deformation phenomenon.

Models can be categorized into two categories, based on the knowledge of the literature:

· models that take into account the distribution property of earth deformations (e.g. linear-deformable semi space);

· models that do not take into account deformation distribution properties (e.g. Winkler model).

2.1 Winkler model

The Winkler model **(also called Fuss-Winkler)** is the first model used in the ground.

The model considers the land to be made up of a set of non-linked resorts. The compression of the springs increases in proportion to the size of the intensity of the loads applied. According to the hypothesis, the springs are introduced only to the load applied, the land in the vicinity of the loaded area does not take part in the deformability phenomenon. The result is a flat distribution of reactive pressures, the hypothesis is closer to reality only in the case of rigid foundations.

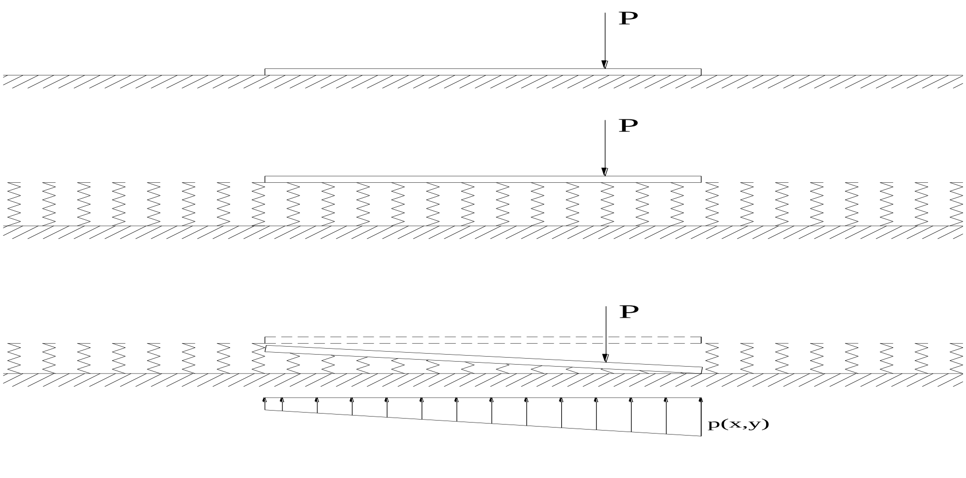


Figure 2.1. 1 Winkler Model
Source: Constantin Ionescu, Teoria sistemelor de rezemare pe medii deformabile

If the foundation is not rigid, then its deformation cannot be neglected compared to the deformation of the foundation ground.

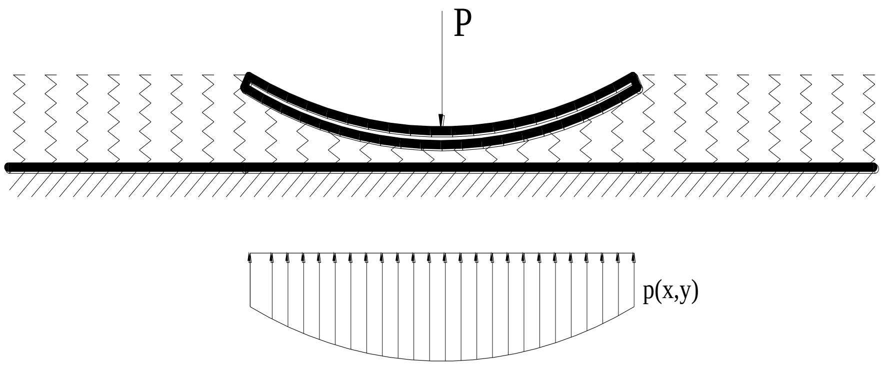


Figure 2.1. 2 Case of elastic foundations placed on a Winkler medium
Source: Constantin Ionescu, Teoria sistemelor de rezemare pe medii deformabile

The hypothesis of proportionality between reaction and settlement was made by N. Fuss in 1798. The model obtained was first used by Winkler in 1867 to calculate road infrastructure.

The model assimilates the foundation ground with a continuous, elastic, and homogeneous environment, and the reaction at any point of the field, in the loaded area, is proportional to the settlement.

If the reaction that occurs in the field and the grounding is noted with p(x,y) then the fundamental equation of Winkler mode is as follows:

$$p(x,y) = k \cdot s(x,y)$$

where:

k - represents a constant of proportionality between reaction and settlement, also called the bed coefficient

The bed coefficient also called the reaction mode or the settlement coefficient is defined as the ratio of the pressure developing in an elastic medium (earth) in a particular section of a building element based on that medium and the corresponding grounding in that section

$$k = \frac{p(x,y)}{s(x,y)}$$

The differential equation of the beam leaning on such a model has the following form:

$$EI_z \frac{d^4 s(x)}{dx^4} + ks(x) = q(x)$$

Table 3.1.1 shows indicative values of the bed coefficient, established for a plate with a side of 30 cm.

Tabel2.1.1 Indicative values of the bed coefficient

Land layer name	K_s(daN/cm^3)
Loose sand	1-2
Medium-thickness sand	4-8
Sand stuffed	10-20
Various plastic clay	15-3
Hard clay	5-10

Source: Lungu Irina, A. Stanciu, *Fundații*, Editura Tehnică, București, 2006

The shortcomings of this model are as follows:

The bed coefficient (k_s) does not have a physical meaning.

Research has shown that a constant coefficient value cannot be established for land.

This coefficient is influenced by:

- the physical properties of the earth;

• the shape and size of the surface of the load plate;

• the size of the load.

The model does not possess the distribution properties of the loads applied in the field. Observations made on actual constructions, in situ, and laboratory experiments have shown that land settlement also depends on the tasks applied at neighboring points. The land suffers from damage not only at the loaded points, according to the hypothesis made, but also in the surrounding areas. It follows that the model cannot take into account the influence of lateral overloads on the distribution of the reagents under the base of the foundation.

According to the hypothesis made, in the case of continuous, uniformly loaded foundations, the reactions equal to the loads result, which leads to confusion that the construction is not required to bend. Experiences have proved otherwise.

The advantages of the Winkler are as follows:

1. in the case of sandy land the model is fair enough; sand has a small distribution capacity; the damping of deformations, beyond the loaded area, is done faster than the semi space model indicates;
2. the simplicity and clarity of the model;
3. small relative influence on the final result, since the bed coefficient does not represent a constant of the earth.

The ways to improve this model, as highlighted in the literature, are:

- the most accurate determination of the bed coefficient, either by experimentation at the site of the future construction or by using the model of the elastic, homogeneous and isotropic semi-space;
- adding to the model a distribution capacity by introducing elements of interaction between the model's springs.

2.2 Linear-deformable semispace model (Boussinesq model)

The dimensions of the linear - deformable semi-space model are infinite. It is limited to the top with a plane and extends down and sideways to infinity. The material in semi-space is considered elastic, homogeneous, isotropic, and continuous. The model thus constituted is referred to, in the literature, as linear-deformable semispace. The elastic properties of the model are represented by two parameters:

• deformation module, "E";

• Poisson's coefficient

Considered a continuous environment, under loads, within the model, tensions and deformations arise, which are found in a linear dependence. It is considered that for the model of linear-deformable semi space, the methods of the Theory of Elasticity are valid. The deviations of the properties of the earth from those of the ideal-elastic body are blurred by an adequate determination, by experimental measurements in situ, of the deformation module "E". The application of the Theory of Elasticity, for the model of linear-deformable semi space, allows the calculation of the constructions based on the ground, taking into account all the main factors that define its behavior, namely:

• variation of the deformation module with depth;

• the influence of new buildings in the vicinity;

• influence of the depth of the foundation;

• the influence of the base rock on which the compressible upper layers re-re-

The use of this calculation model, for the foundation ground, could explain the degradation of many constructions calculated based on the bed coefficient hypothesis.

Due to the scientific accuracy of this model and the considerations outlined above, it has now become widely applicable in the calculation of beams, plates, and structures based on the deformable medium, although mathematically many complications occur.

The shortcomings of this model are as follows:

• greatly idealizes the behavior of the foundation ground, which is neither elastic nor isotropic;

• neglects the non-linear nature of the terrain deformations;

• does not take into account the fact that the depreciation of deformations is greater, as the point of application of the task is removed than results from the application of the Methods of elasticity theory.

These deficiencies have led to the emergence among specialists of two distinct scientific currents:

- the first category of specialists are the advocates of improving the linear-deformable semi-space model by taking into account the negative factors exposed above;
- the second category of specialists, based on the fact that the Winkler model possesses a great simplicity and mathematical elasticity, proposes to determine the bed coefficient by methods of the Theory of Elasticity or by considering the variable bed coefficient under the base of the foundation.

Applying known methods, an analysis of the field structural interaction, as well as the sizing and detailing of the infrastructure, can be carried out. While these methods may be useful, practical difficulties arise in principle that prevents their application. In particular, concrete non-linear behavior and actual behavior over time add to the complexity of the problem. In addition, the parameters of the terrain determined in advance can be limited. Any investigation of the land is limited in scope and size, and the results obtained are difficult to interpret. Finally, the residual tension in the field is not very well known and thus their influence on the behavior of the structure cannot be appreciated. Looking at these issues, we cannot believe that any analysis could lead to an accurate representation of the tensions and distortions that have arisen throughout the structural system. simplified approaches should therefore be adopted. These approaches relate to how the resulting N, T, M from the base of the superstructure can be transmitted to the ground through the foundation. Different approaches lead to different distributions on the base of the foundation. Classically the transmission of the basic T cutting force can be seen as a secondary problem, compared to the transmission of gravitational forces N and M moments.

2.3 Determination of the bed coefficient

In the literature, the bed coefficient is also called the mode of reaction or coefficient of settlement. This is defined as the ratio of pressure developing in an elastic environment in a given section of a building element based on that medium and the corresponding grounding in that section.

It is thus proposed that the bed coefficient be calculated by the Winkler Method, refined at the base of the foundation system, which consists in the dependence of the bed coefficient (k_S)on the compressibility of the land (E) as well as the size of the foundation (B, L).

$$p(x,y) = k_S \cdot s(x,y)$$

where:

k_S -represents a constant of proportionality between reaction and settlement, also called the bed coefficient

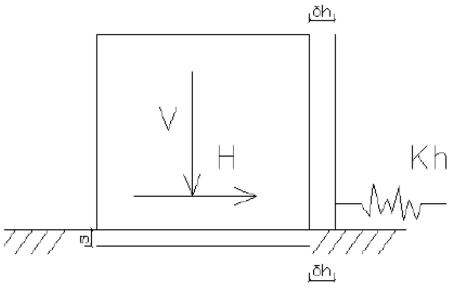


Figure 2.1. Rigidity calculation model

Using the hypothesis of elastic semi-space, characterized by parameters:

- E = elasticity mode; o = Poisson coefficient (α = 0,5) is accepted.
- H - represents the basic cutting force resulting from the special grouping;
- V - represents the weight of the building specific to the grouping of long-lasting vertical loads;
- ρh - lateral displacement of the structure in the earth massif;
- s - vertical settlement;
- L and B - the plane dimensions of the eraser or foundation system.

Thus determining the coefficient of stiffness of the grounding at lateral forces.

$$\delta_h = s \cdot \frac{H}{V} \cdot [1 + \frac{0.6}{1 + \sqrt{\frac{L}{B}}}]$$

Settlement according to Boussinesq's resolution (admitting o = 0.5) becomes:

$$S = \frac{\omega \cdot p \cdot B}{E} \cdot (1 - \nu^2) = \frac{0.8 \cdot V \cdot B}{E \cdot B \cdot L} \cdot (1 - 0.5^2) = \frac{0.6 \cdot V}{E \cdot L}$$

$$\delta_h = \frac{0.6 \cdot V}{E \cdot L} \cdot \frac{H}{V} \cdot [1 + \frac{0.6}{1 + \sqrt{\frac{L}{B}}}]$$

The transverse stiffness coefficient of the Kh sole shall be determined as follows:

$$\mathrm{K}_h = \frac{H}{\delta_h} = \frac{E \cdot L \cdot H}{0.6 \cdot H \cdot [1 + \frac{0.6}{1 + \sqrt{\frac{L}{B}}}]} = \frac{1.67 \cdot E \cdot L}{[1 + \frac{0.6}{1 + \sqrt{\frac{L}{B}}}]}$$

It is noted that by assuming maintenance in the elastic field H=V, the horizontal displacement is:

$$\delta_h = s \cdot [1 + \frac{0.6}{1 + \sqrt{\frac{L}{B}}}]$$

Consequently, an average transverse 'bed coefficient' may be considered (if H = V →

$$\mathrm{K}_h = \frac{\tau}{\delta_h} = \frac{\tau}{s \cdot [1 + \frac{0.6}{1 + \sqrt{\frac{L}{B}}}]} = \frac{P}{s} \cdot \frac{1}{1 + \frac{0.6}{1 + \sqrt{\frac{L}{B}}}}$$

$$\mathrm{K}_h = \mathrm{K}_s \cdot \frac{1}{1 + \frac{0.6}{1 + \sqrt{\frac{L}{B}}}}$$

And the overall coefficient of transverse stiffness (on the entire sole of the foundation) is:

$$K_h = k_s \cdot B \cdot L$$

In the study of this paper, the transverse rigidity for the land made of clay and for the land made of sand and gravel is determined. The plane dimensions of the discrete model erasers are B x L, i.e. 30.00 x 30.00 m.

For the comparative study, the values of Kh slightly covering compared to those resulting from the measurements in the site were taken into account in the calculation models analyzed:

It is then proposed to determine the coefficient of horizontal reaction to model the ground-structure interaction on the entire surface of the sidewalls of the infrastructure.

During the completion of the excavation, the enclosure wall tends to move towards the inside of the excavation due to the imbalance of pressures. After reaching the active thrust, the resulting contact pressure shall be kept constant regardless of the increase in lateral displacement.

If the enclosure wall moves towards the earth mass, the contact pressure will increase with the displacement, until the pressure (passive resistance) is reached.

δ_a , δ_p - *movements causing active or passive disposal* shall be determined on small-scale models or by in situ measurements.

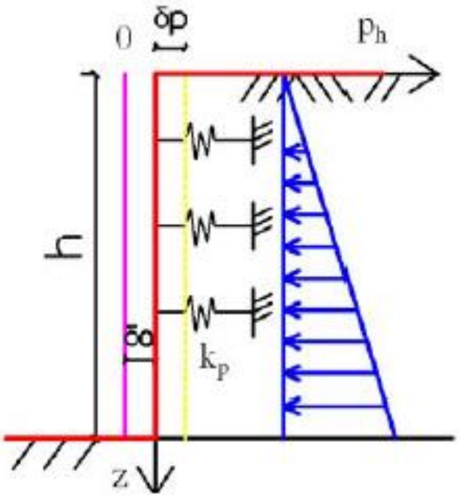


Figure 2.2. Vertical wall co-operation scheme – land infrastructure

ÎThe figure below shows the schematic diagram, established from the experiments carried out, which represents the relationship between the contact pressure associated with active and passive pushing and the lateral movements of the enclosure wall.

in which: p_a – active pressure; p_p – passive pressure; p_0 – the pressure at rest.

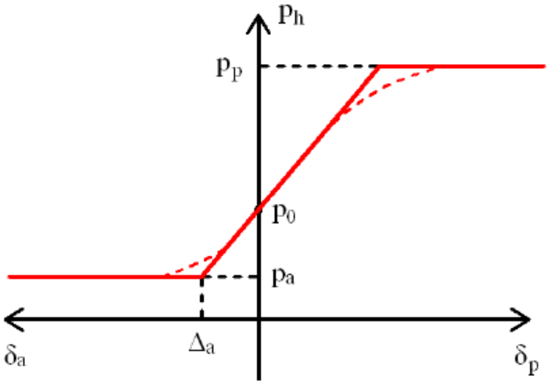


Figure 2.3. Relationship between contact pressure and lateral displacement of the support wall

The relationship (δ, ph) in the linear (elastic) field has been established experimentally and is recommended in European rules. The horizontal reaction coefficient is, therefore:

$$k_p = \frac{P_h}{\delta}$$

The following parameters were taken into account in the particular study of this paper: *δ = ρa = 0.0005h* for stuffed and *δ = ρa = 0.002h* for deep earth.

$$k_p = \frac{\Delta P_h}{\Delta a} = \frac{P_0 - P_a}{\Delta a} = \frac{\gamma_z \cdot (K_0 - K_a)}{\Delta a}$$

$$k_0 = 1 - sin\theta$$

$$k_a = tg^2(45^o - \frac{\theta}{2})$$

2.4 Admissible and tolerable settlement

2.4.1 Possible movements and deformations

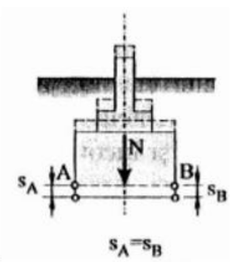
To achieve the classification of structures within the permissible, permissible, or tolerable limits, the following types of the settlement have been defined:

- Absolute settlement (fig.2.5.1.1.a and b) is the vertical displacement of a point of the foundation, usually the center of gravity
- Probable mean settlement of construction – the arithmetic mean of at least three isolated foundations of the construction, characteristic by the dimensions in the plane and loads. This can also be calculated as the weighted average to the areas (Ai) of the soles of the foundations:

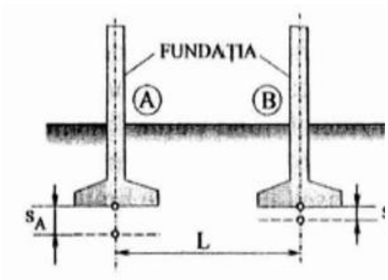
$$S_{med} = \frac{\sum_1^n s_i \cdot A_i}{\sum_1^n A_i}$$

- Probable relative settlement (fig.2.5.1.1.c and d) – defined as the ratio between the tamping of two neighboring foundations (S_A-S_B) or two adjacent points belonging to the same foundations (δ_{ij}) and the distance between them (L) taking into account the most unfavorable loading case:

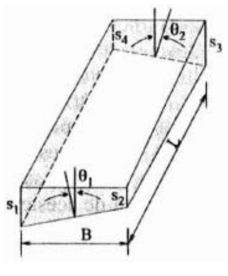
$$S_{rel} = \frac{S_A - S_B}{L} = \frac{S_{AB}}{l_{AB}}$$



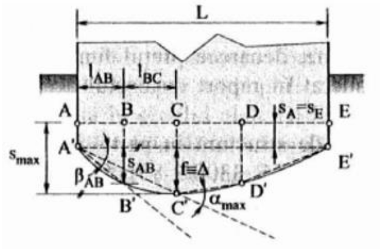
a) Settlement



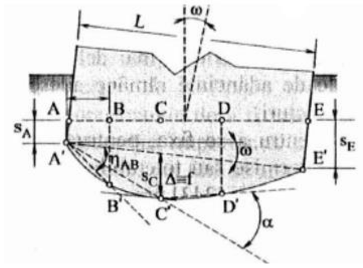
b) Settlement



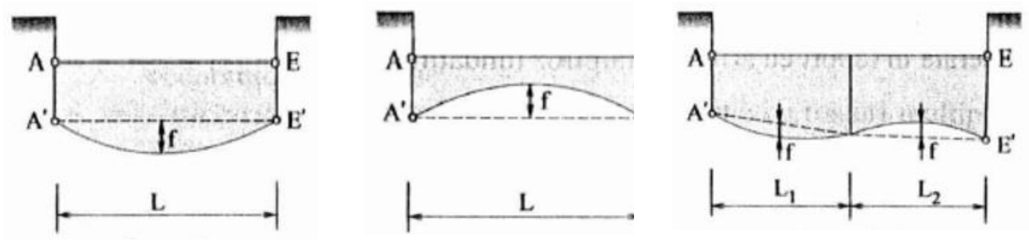
c) the relative torsion angle of the construction grounded on the eraser



d) uneven tilt-free



e) Uneven tilting



e) covet f) saddle g) saddle and covet

Figure 2.4Scheme for defining types of settlement and their elements
Source: Lungu Irina, A. Stanciu, *Fundații*, Editura Tehnică, București 2006

- The inclination of the foundation (Figure 2.4.e), tgρ, represents the difference between the settlement of two extreme points of the foundation, relative to the distance between them.

$$tg\theta = \frac{\Delta}{L}$$

or

$$tg\theta = \frac{S_{E-}S_A}{L} \qquad fig.2.4.e$$

- Arrow (f) or deflection (ρ) is the maximum vertical distance measured between the tangent at the settlement profile, parallel to the tilt line (AE) and this (Figure 2.4.d and e)
- Relative bending or deflection ratio is the ratio between the arrow(f)/deflection and the length of the part of the construction that bends without a change of sign (Figure 2.4.f and g)

$$i = \frac{f}{L} = \frac{\Delta}{L} \cong \frac{2 \cdot S_C - S_A - S_E}{L}$$

- Differentiated settlement δ_{ij} as the difference between the settlement of two neighboring points belonging to the same foundations (Figure 2.4.d)

$$\delta_{ij} = \delta_{AB} = S_{AB} = S_A - S_B$$

- Angular distortion (ηij) or relative rotation (βij), (Figure 2.4.e and d) as the ratio between the differentiated settlement (δij) and the associated length from which the inclination is subtracted:

$$\eta_{AB} = \frac{\delta_{AB}}{l_{AB}} - \omega$$

or

$$\beta_{AB} = \frac{\delta_{AB}}{l_{AB}}$$

Angular deformation:

$$\alpha = \frac{\Delta S_{AC}}{l_{AC}} + \frac{\Delta S_{CD}}{l_{CD}}$$

where:

ΔS_{AC} ;ΔS_{CD}– presents the differentiated settlement between the AC and CD points respectively;

- Complex deformations resulting from overlapping several types of deformations of the type shown in figure (Figure 2.4.d and e) showing both inclinations and relative bending;
- The relative torsion angle of the construction (Figure 2.4.c) which attempts to characterize the spatial behavior of the construction, the foundations of the eraser type, based on those for settlement recorded in the corners of the foundation:

$$\theta_1 \cong -\frac{S_1 - S_2}{B}$$

$$\theta_2 \cong tg\theta_2 = -\frac{S_3 - S_1}{B}$$

$$\chi = \frac{\theta_1 + \theta_2}{L}$$

It can be noted that the elastic foundation (Figure 2.4.f,g, and h) may have variable profiles, in concave form, of the saddle, or combined according to the distribution of loads (P_i) on the foundation, their relative values (ρP_i), but also on the distribution of the compressibility of the land along with the foundations. To avoid the limit states S.L.D.E.N. and S.L.D.U., respectively, the question arises of setting limit values for settlement or their components.

The first approach to this problem was made by Skempton and Mc. Donald (1956) who pursued 98 buildings with structure made of load-bearing masonry, steel and reinforced concrete consisting of 40 of them having architectural and even structural degradations.

Based on these findings, limit thresholds for angular distortion, differentiated settlement, and maximum settlement in the case of clay or sand foundations (Table 2.1) were proposed.

Tabel 2.1. Construction limits tass and recommended maximum values in mm

criterion	Type of earth	Isolated Foundation	Radier Foundation
Angular distorsis ρ/L	Architectural degradations	1/300	
	Structural degradations	1/150	
Maximum differentiated settlement δ_{ij}	clay	45(40)	
	sand	30(25)	
Maximum settlement	clay	75(65)	75-135(60-100)
	sand	50(40)	50-75(40-60)

Source: Lungu Irina, A. Stanciu, *Fundații*, Editura Tehnică, București, 2006

SREN 1997-1-2004 recommends for structures in open frames, for frames in frames with fillers, for walls of load-bearing masonry to be the same, but they are between 1/2000 – 1/300, to prevent the achievement of a limit state of normal operation in the structure. A maximum relative rotation of 1/500 is acceptable for many structures. The relative rotation for which it is likely to be the last limit state is about 1/500.

For ordinary constructions with insulation foundations, total settlements of up to 50 mm are often acceptable. Higher total and differentiated settlement values may be allowed if the permissible rotations remain within acceptable limits and if the settlement does do not create deficiencies in the networks entering the construction and does not cause inclinations.

Terzaghi and Peck (1948() suggest that values limit a maximum sand value of 25 mm and a differentiated value of 0.75 of the maximum settlement.

Polsh and Takor (1957) indicate angular distortion values η or β (ρ/L) of 1/500 for architectural degradation and 1/250 for structural degradations. Meyerhof reached a similar set of conclusions.

Bjerrum (1963) showed that no damage was recorded in buildings on a general clay-based eraser, for differentiated settlements of less than 125 mm and a total settlement of 250 mm.

Damage was recorded in buildings with insulated clay foundations for differentiated tasses greater than 50 mm and totals of more than 150 mm.

Finally, it indicated the following limit angular deformations at which degradation may occur (Table 2. 2).

Table 2.2 Angular deformations to which construction degradation is expected

Angular deformation (radians)	The behavior of the construction
η=1/750 (0.0013)	The limit to which difficulties may arise in the functionality of machines sensitive to settlement
η=1/600 (0.00167)	Limit of the danger of degradation of diagonal frames
η=1/500 (0.002)	The safety limit for buildings where cracks are not allowed
η=1/300 (0.0033)	The limit to which the first cracks in the partition walls are expected
η=1/300 (0.0033)	The limit to which the first cracks in the partition walls are expected
η=1/250 (0.004)	The limit at which the inclination of high rigid constructions becomes visible
η=1/150 (0.0067)	Appreciable cracks in partition walls and brick masonry
η=1/150 (0.0067)	The safety limit for flexible brick walls to which h/l<1/4
η=1/150 (0.0067)	The limit at which general degradations of the construction structure may occur.

Source: Lungu Irina, A. Stanciu, *Fundații*, Editura Tehnică, București, 2006

Burland and Wroth (1975), indicate for the deflection ratio (ρ_{max}/L), for load-bearing masonry structures the following limit values:

$$\Delta_{max}/L = 2 \cdot 10^{-4} \quad \text{for } L/H=1$$

$$\Delta_{max}/L = 4 \cdot 10^{-4} \quad \text{for } L/H=5$$

Grand et al.e.g. (1974) took over the study of Skempton and McDonald, adding in addition to 98 other 95 buildings, a much more recent date (some even after 1950). They concluded that the limit of 1/300 is exceeded and buildings record some degradation, proposing that these simple, easy-to-use relationships be viewed with caution, the 1/300 limit should be regarded as a broad limit, which does not take into account the specific conditions of each site, the characteristics of each building.

As differentiated settlements (ρ), i.e. angular distortions /relative rotations (η;β), are relatively difficult to assess, solutions have been sought to find correlations between maximum total settlement and these sizes.

The maximum differentiated settlement shall be expressed based on field observations as representing:

$\Delta=0.20S_{max}$ - For rigid superstructures on clays;
$\Delta=0.40S_{max}$ - For flexible superstructures on clays;
$\Delta=S_{max}$ - For sands;

In other words, it is estimated that the differentiated settlement would represent 1/2 of the maximum or even 3/4 settlement.

Grant and others indicate a correlation of the following type:

$$S_{max} = R \cdot \frac{\Delta}{L}$$

Between the maximum settlement and the relative deflection, i.e. the ratio of deflections (ρ/L) to the values of R in Table 2.3.

Table 2.3. Correlation of the values of the maximum permissible settlements (ρ=Smax) relative deflection (ρ/L).

Type of land	Value	Type of foundation	
		Insulated	eraser
clay	R	22500	30000
	Smax (mm)	75	100
sand	R	1500	1800
	Smax (mm)	50	60

Source: Lungu Irina, A. Stanciu, *Fundații*, Editura Tehnică, București, 2006

Egorov indicates the following limit values for settlement with the rigidity of the structure (Table 2.4)

Table 2.4. The maximum permitted construction cuts, tilts, arrows, and bends

Type of construction	Uniform settlement (cm)	Relative sizes	
		tilts	Arrows and bends
Perfectly rigid	25-50	0.005	-
Relatively rigid	5-10	0.002	0.001
non-rigid	15-25	0.002	0.002

Source: Lungu Irina, A. Stanciu, *Fundații*, Editura Tehnică, București, 2006

NP 123 indicates the following limit settlement values (Table 2.5)

Tabel 2.5. Tab limit values

Type of construction		Deformations		Movements (settlement)	
		Type of deformation	Limit value [-]	Type of movement	Limit value [mm]
1	Civil and industrial constructions with resistance structure in frames: a) Frames of reinforced concrete without a wall or panels	relative settlement	0,002	absolute settlement maximum,sma	80
	b) Steel frames without masonry filling or panels	relative settlement	0,004	absolute settlement maximum,smax	120
	c) Concrete frames reinforced with masonry filling	relative settlement	0,001	Absolute settlement maximum,smax	80
	d) Steel frames with masonry filler or panels	relative settlement	0,002	absolute settlement maximum,smax	120
2	Constructions in structure not to appear additional efforts due to Uneven	relative settlement	0,006	absolute settlement maximum,smax	150
3	Multi-story constructions with load-bearing walls of a) large panels	relative bending, f	0,0007	the medium settlement, sm	100
	b) masonry of blocks or brick, without arming	relative bending, f	0,001	the medium settlement, sm	100
	c) masonry of blocks or reinforced brick	relative bending, f	0,0012	the medium settlement, sm	150
	d) independent of the material of the walls	transverse tilt $tg\theta_{tr}$	0,005	-	-
4	High, rigid constructions (a) Reinforced concrete silos: - the tower of elevators and the groups of cells monolith concrete and support on the same continuous eraser	longitudinal tilt or transverse $tg\theta$	0,003	the medium settlement, sm	400
	- the tower of elevators and the groups of cells are b.a.p. and support on the same eraser	longitudinal tilt or transverse $tg\theta$	0,003	medium settlement, sm	300
	- the tower of elevators leaning on an independent eraser	transverse tilt $tg\theta_{tr}$	0,003	the medium settlement, sm	250
		longitudinal tilt $tg\theta_{l}$	0,004	medium settlement, sm	250

	- groups of monolith concrete cells leaning on an independent eraser	longitudinal tilt or transverse $tg\theta$	0,004	medium settlement, sm	400
	- groups of b.a.p. cells leaning on an independent eraser	longitudinal tilt or transverse $tg\theta$	0,004	medium settlement, sm	300
	b) Chimneys with height H[m]:				
	H < 100 m	Tilt, $tg\theta$	0,005	the medium settlement, sm	400
	$100 \le H \le 200\,m$	Tilt , $tg\theta$	1 / 2H	medium settlement, sm	300
	$200 < H \le 300\,m$	Tilt, $tg\theta$		the medium settlement, sm	200
	H > 300 m	Tilt , $tg\theta$		the medium settlement, sm	100
	c) High, rigid constructions, H < 100 m	Tilt , $tg\theta$	0,004	medium settlement, sm	200

Cap.3 Case study on the determination of the structures in reinforced concrete frames.

3.1 Description of the model analyzed

For example, a structure located in the city of Cluj Napoca was chosen. The structure is characterized by reinforced concrete frames whose elements have been sized according to the current design codes. The type of frame structure has been chosen to analyze the degree of damage.

Description:

- Height regime: P+7E (H_{level}=3.00m. H_{total}=24.00m);
- Geometry in plane: 4 opens 7.00m, 4.00m respectively and 2 travails of 6.00m;
- Structure type: reinforced concrete frames;
- Type of foundations: direct foundation – insulated foundations with/without balancing beams;

Materials:

- Concrete reinforced C25/30 (E=31000kN/m^2)
- Concrete steel BST 500S

Dimensions of structural elements:

- Reinforced concrete columns with dimensions 50x50 cm and 60x60 cm respectively;
- 30x60 reinforced concrete beams;
- Reinforced concrete floor 15 cm.

Assessment of loads:

1. Distributed on slab:
   - Useful 1,5 kN/m2
   - Quasipermanent 3,5 kN/m2

Seismic force:

- Behavior factor q=4.725
- (Class M of ductility)
- Seismic coefficient c=0.044
- Building weight 27865 kN
- FTB=1253 kN

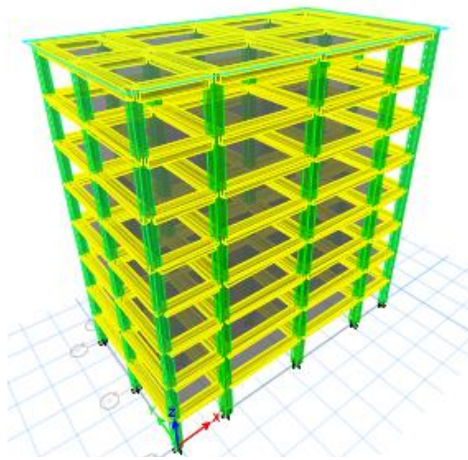


Figure 3. 1 Reinforced concrete building model

The building model was subjected to a seismic action determined by calculation. The structure has been designed by the current design codes resulting in the following types of reinforcements for the elements:

The foundations were dimensioned based on conventional pressure Pconv. A bed coefficient of 16800 kN/m3was obtained after the ironing was carried out under each foundation. Figure 3.2 shows a comparison of foundation stakes.

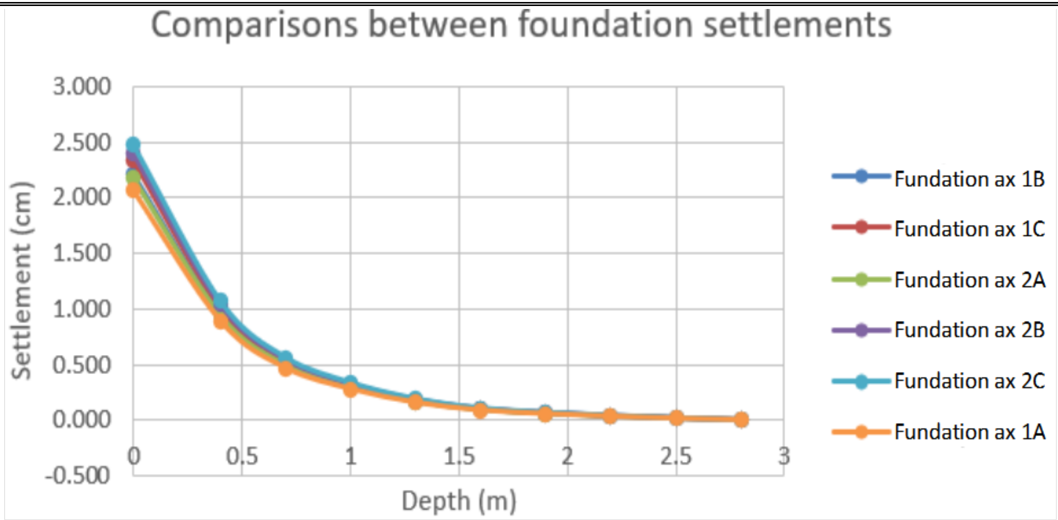


Figure 3. 2. Comparisons between foundation sands

Modeling land-structure interaction.

Knowing the value of the bed coefficient, the modeling of the interaction between the land and the structure was achieved by inserting some solutions under each foundation.

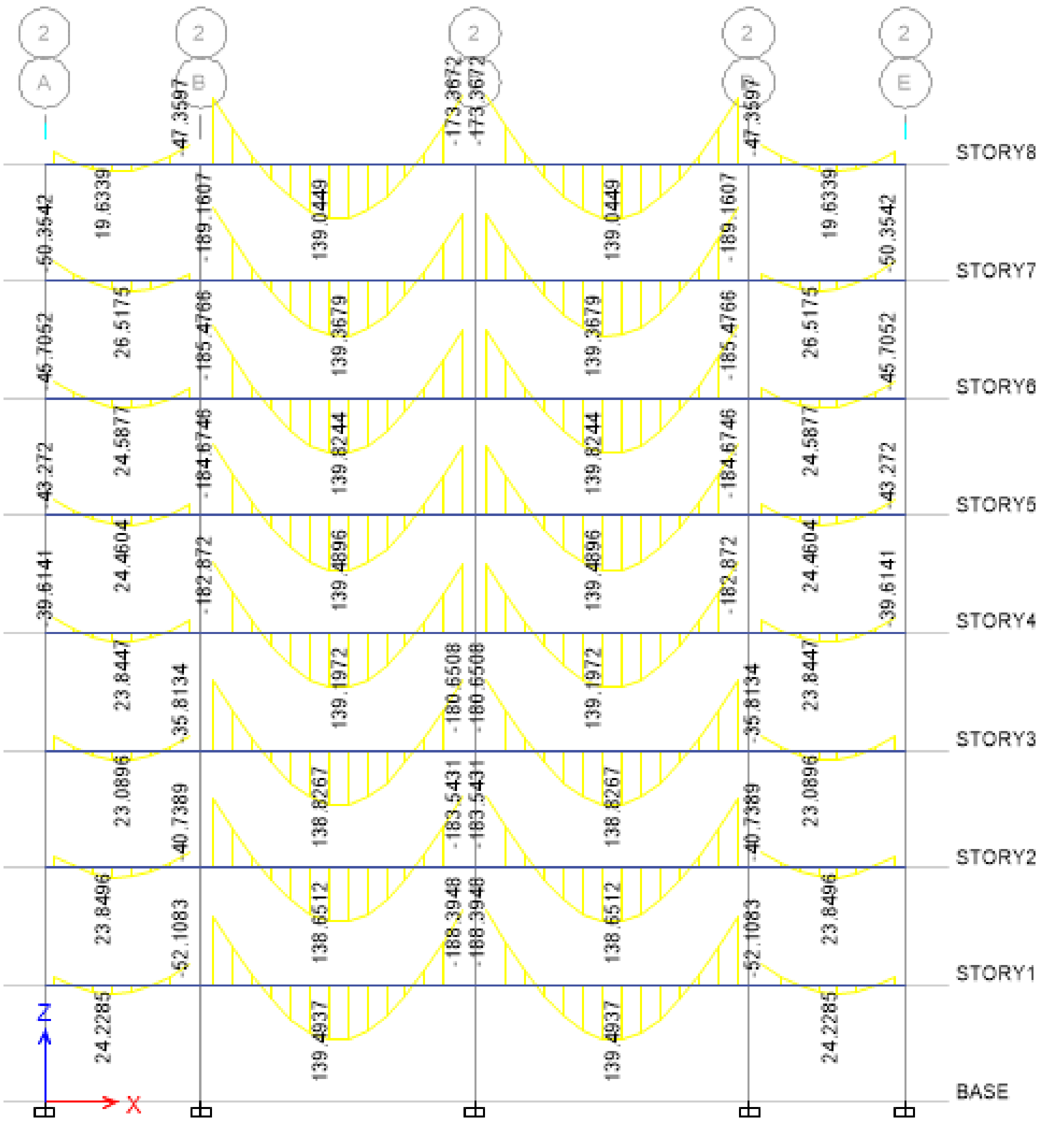


Figure 3. 3. Effort diagram in the fundamental grouping for the central longitudinal frame.

Model isolated foundations.

The foundations were modeled with a spring under each pillar, with the calculated bed coefficient. The following in Figure 3.4 is presented the actual moments of the fundamental grouping taking into account also the effect of the settlement.

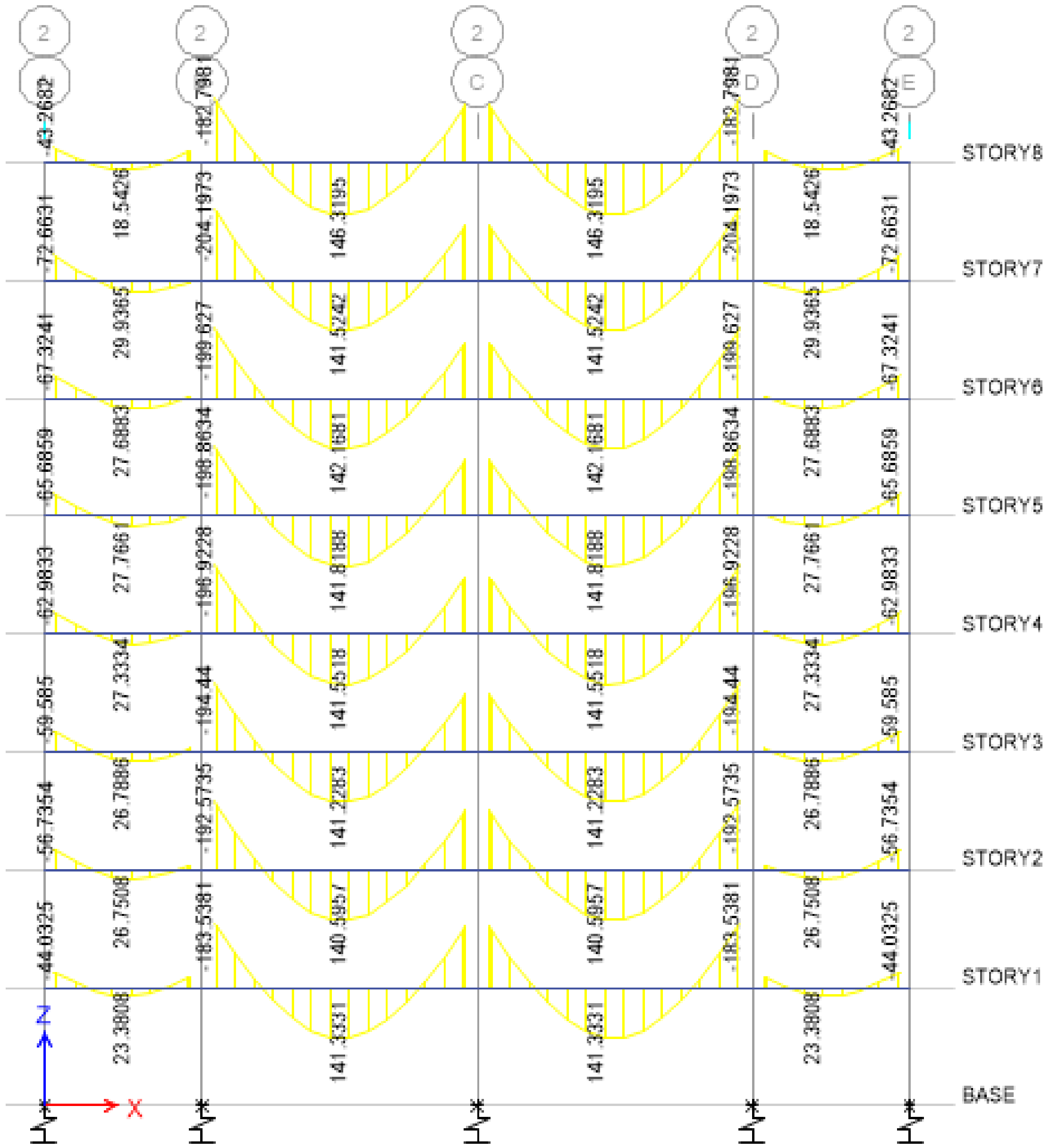


Figure 3. 4 Effort diagram of the fundamental grouping for the central longitudinal frame isolated foundation model.

To see if additional efforts occur in the beams, calculate the ratio of beam moments from the GF+type and GF grouping in the element sizing model.

	2A	FIELD	2B	2B	FIELD	2C	2C	FIELD	2D	2D	FIELD	2E
Floor 7	1.49		0.73	1.10		0.95	0.95		1.10	0.73		1.49
		1.08			1.06			1.06			1.08	
Floor 6	1.44		-0.25	1.08		0.90	0.90		1.08	-0.25		1.44
		1.13			1.02			1.02			1.13	
Floor 5	1.47		0.14	1.08		0.91	0.91		1.08	0.14		1.47
		1.13			1.02			1.02			1.13	
Floor 4	1.52		0.17	1.08		0.91	0.91		1.08	0.17		1.52
		1.14			1.02			1.02			1.14	
Floor 3	1.59		0.24	1.08		0.91	0.91		1.08	0.24		1.59
		1.12			1.02			1.02			1.12	
Floor 2	1.70		0.30	1.08		0.91	0.91		1.08	0.30		1.70
		1.16			1.02			1.02			1.16	
Floor 1	1.90		0.32	1.08		0.91	0.91		1.08	0.32		1.90
		1.19			1.02			1.02			1.19	
Ground floor	2.13		0.55	1.06		0.92	0.92		1.06	0.55		2.13
		1.17			1.02			1.02			1.17	

Figure 3. 5. The ratio of the actual moments in the fundamental grouping+settlement and the moments in the fundamental grouping model isolated the foundation

It is noted that due to the settlement of the central pillars with a value greater than that of the marginal pillars, the actual moments in the central axis are smaller than the moments in the recessed model at the base, and the moments in the marginal axes are much greater than the moments in the embedded model at the base.

Following static ironing, it is observed that the absolute settlement will be equal to 2.13 cm, which is less than the limit imposed by the design code (8 cm).

Figure 3.6 shows that the relative settlement is within the limits imposed by the design code. In this situation, it is expected that no degradations of the partition walls and no plastic joints in the resistance structure.

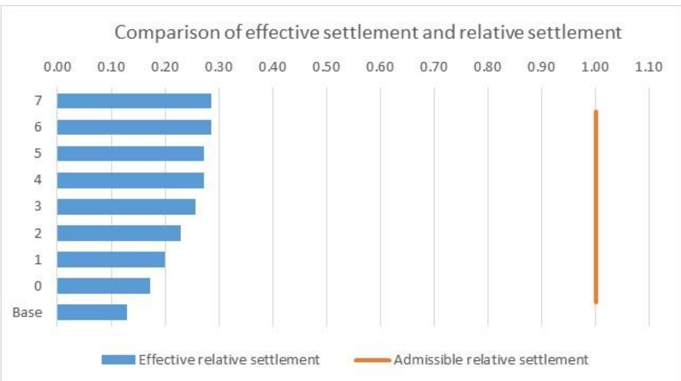


Figure 3. 6. Diagram of relative settlement for each isolated foundation model level

Model insulated foundations ks shrunk for the central foundation.

To capture the unevenness of the land, a bed coefficient with a value less than the bed coefficient resulting from the calculation shall be considered. Thus the bed coefficient is reduced by the ratio between P_{pl} and P_{conv}.

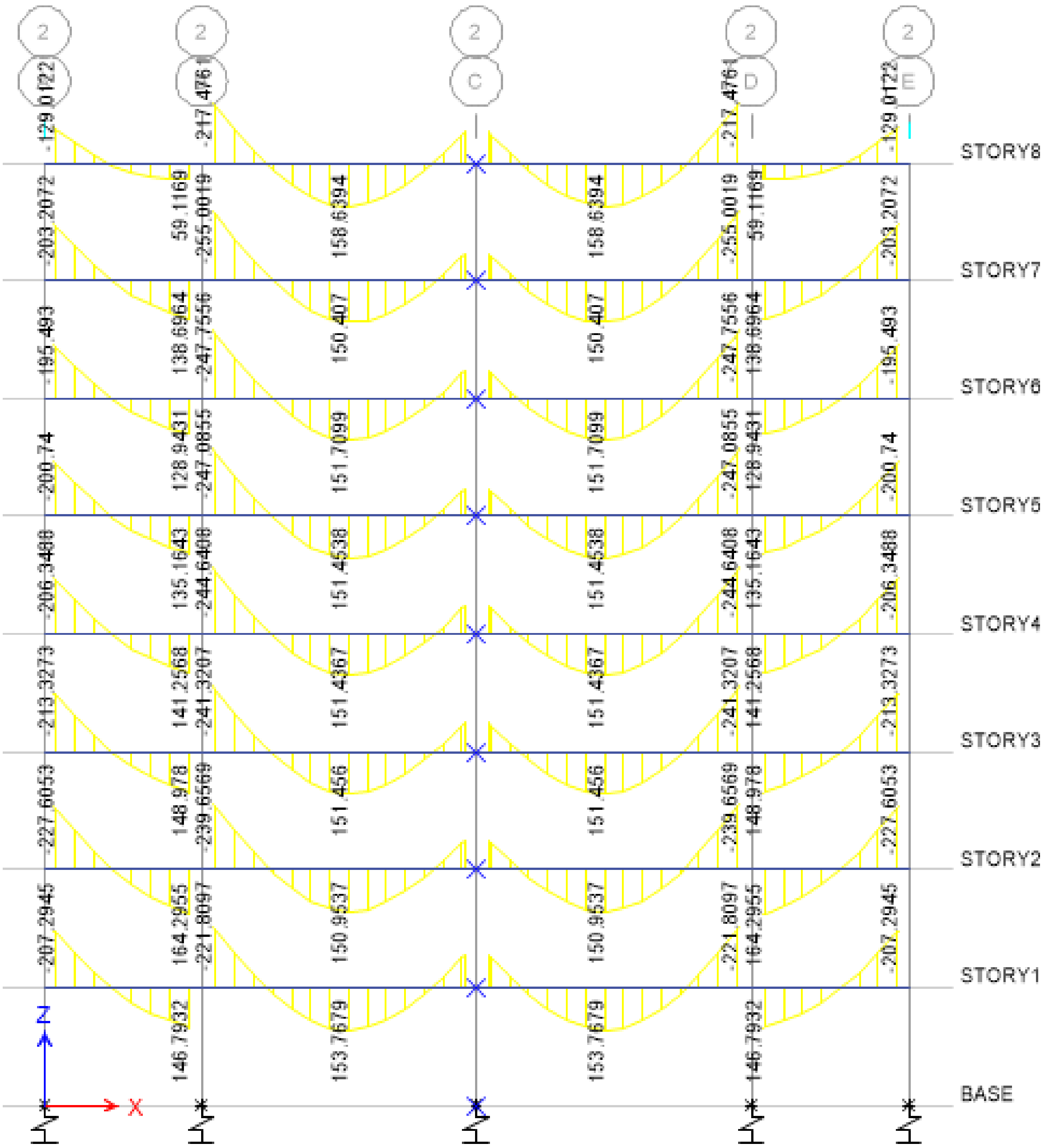


Figure 3. 7. Effort diagram of the fundamental grouping for the central longitudinal frame isolated foundation model ks shrunk.

Figure 3.7 shows the actual moments in the fundamental grouping taking into account the effect of the settlement. It is noted that the moments in axis 2C are smaller than in axis 2B and 2D, indicating that the pillar in axis 2C is not entirely a joint point, which is more than the rest of the pillars.

To see if additional efforts occur in the beams, calculate the ratio of beam moments from the GF+type and GF grouping in the element sizing model.

	2A	FIELD	2B		FIELD	2C		FIELD	2D		FIELD	2E
Floor 7	4.45		-1.25	1.31		0.65	0.65		1.31	-1.25		4.45
		2.03			1.15			1.15			2.03	
Floor 6	4.04		-7.57	1.35		0.55	0.55		1.35	-7.57		4.04
		1.89			1.08			1.08			1.89	
Floor 5	4.28		-5.12	1.34		0.57	0.57		1.34	-5.12		4.28
		1.95			1.09			1.09			1.95	
Floor 4	4.64		-4.96	1.34		0.57	0.57		1.34	-4.96		4.64
		2.02			1.09			1.09			2.02	
Floor 3	5.21		-4.55	1.34		0.57	0.57		1.34	-4.55		5.21
		2.17			1.09			1.09			2.17	
Floor 2	6.09		-4.16	1.34		0.58	0.58		1.34	-4.16		6.09
		2.24			1.09			1.09			2.24	
Floor 1	7.63		-4.03	1.34		0.59	0.59		1.34	-4.03		7.63
		2.41			1.09			1.09			2.41	
Ground floor	10.05		-2.82	1.28		0.62	0.62		1.28	-2.82		10.05
		2.66			1.11			1.11			2.66	

Figure 3. 8. The ratio of the actual moments in the fundamental grouping+settlement sithes and the moments in the fundamental group model isolated foundation ks shrunk

As anticipated, the moments of the beams in the 2C axis are about 40% smaller than the moments of the beams in the recessed model at the base. For the joint of the axis 2B and 2D, there is even a change of sign of the moment, this being positive. For axis 2A and 2E, the moments are even 10 times higher.

Following static calculation, it is observed that the absolute settlement will be equal to 3,43 cm, which is less than the limit imposed by the design code (8 cm).

Figure 3.9 shows that the relative settlement is within the limits imposed by the design code. In this situation, it is expected that no degradations of the partition walls and no plastic joints in the resistance structure.

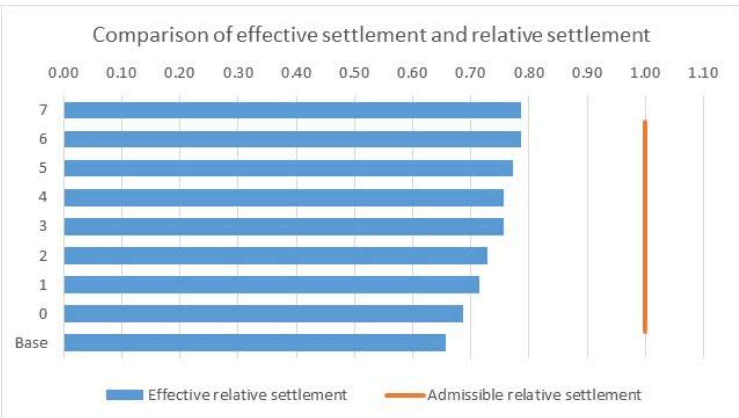


Figure 3. 9. Diagram of relative settlement solders for each model level insulated foundation ks shrunk

Figure 3.10 shows the distribution of plastic joints in the structure. The capable moments of the beams were calculated using the average resistors. Even if the relative settlement falls within the limits imposed by the code it is observed that in the resistance structure will appear plastic joints.

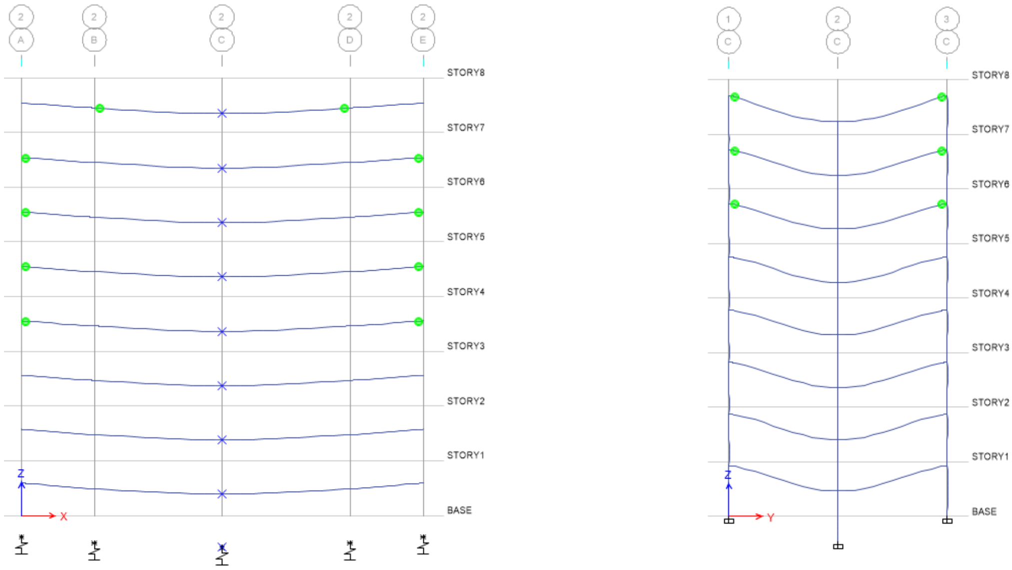


Figure 3. 10. The appearance of plastic joints insulated foundation and shrank ks

<u>Model insulated foundation with joint cedar.</u>

For the pillar in axis 2C, a displacement was imposed so that the relative settlement was 1.1%.

Figure 3.11 shows the actual moments in the fundamental grouping taking into account the effect of the settlement. It is noted that the moments in axis 2C are smaller than in axis 2B and 2D, indicating that the pillar in axis 2C is not entirely a joint point, which is more than the rest of the pillars.

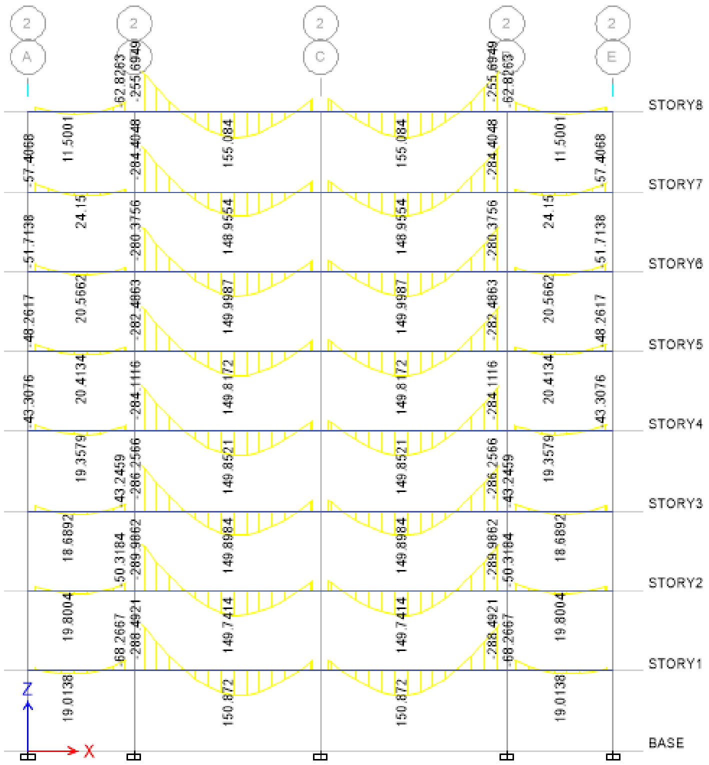


Figure 3. 11. Effort diagram of the fundamental grouping for the central longitudinal framework model isolated foundation with joint

To see if additional efforts occur in the beams, calculate the ratio of beam moments from the GF+type and GF grouping in the element sizing model.

	2A	FIELD	2B		FIELD	2C		FIELD	2D		FIELD	2E
Floor 7	1.09		1.33	1.54		0.50	0.50		1.54	1.33		1.09
		0.43			1.12			1.12			0.43	
Floor 6	1.14		0.94	1.50		0.42	0.42		1.50	0.94		1.14
		0.91			1.07			1.07			0.91	
Floor 5	1.13		1.10	1.51		0.43	0.43		1.51	1.10		1.13
		0.84			1.07			1.07			0.84	
Floor 4	1.12		1.13	1.53		0.41	0.41		1.53	1.13		1.12
		0.83			1.07			1.07			0.83	
Floor 3	1.09		1.17	1.55		0.40	0.40		1.55	1.17		1.09
		0.76			1.08			1.08			0.76	
Floor 2	1.06		1.21	1.58		0.38	0.38		1.58	1.21		1.06
		0.78			1.08			1.08			0.78	
Floor 1	1.00		1.24	1.62		0.36	0.36		1.62	1.24		1.00
		0.76			1.08			1.08			0.76	
Ground floor	0.85		1.31	1.67		0.35	0.35		1.67	1.31		0.85
		0.61			1.09			1.09			0.61	

Figure 3. 12. The ratio between the actual moments in the fundamental grouping+tats and the moments in the fundamental grouping model isolated foundation and the joint.

The moments in the 2C axis are even 65% smaller than the moments in the bottom-encased model, and in the rest of the elements, they increased by up to 60%.

Following static ironing it is observed that the absolute settlement will be equal to 7mm, this being less than the limit imposed by the design code (8 cm).

Figure 3.13 shows that the relative settlement is within the limits imposed by the design code.

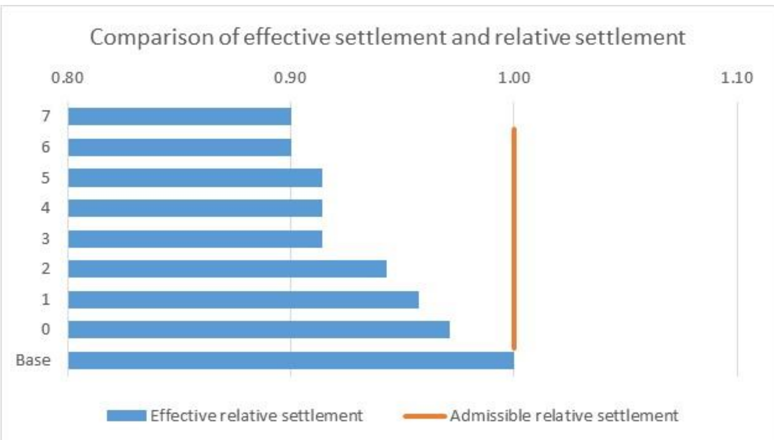


Figure 3. 13. Diagram of relative settlement for each model level isolated foundation and joint

Figure 3.14 shows the distribution of plastic joints in the structure. The capable moments of the beams were calculated using the average resistors. Even if the relative settlement falls within the limits imposed by the code it is observed that in the resistance structure will appear plastic joints.

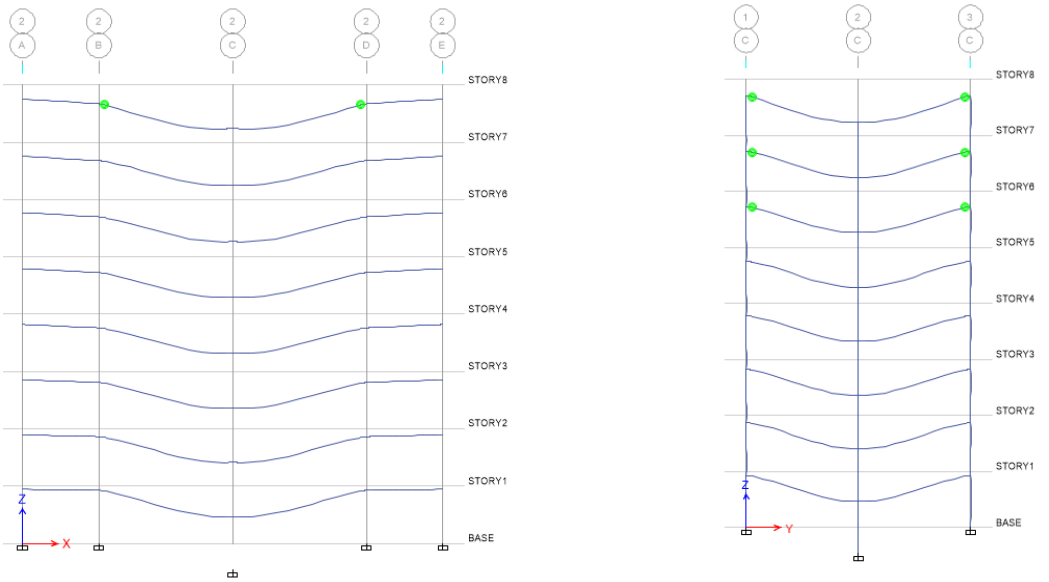


Figure 3. 14. The appearance of plastic joints model insulated foundation and joint

Model foundation beams.

Maintaining the same hypothesis will be considered that the land is uneven under construction. The following in Figure 3.15 is presented the actual moments of the fundamental grouping taking into account also the effect of the settlement.

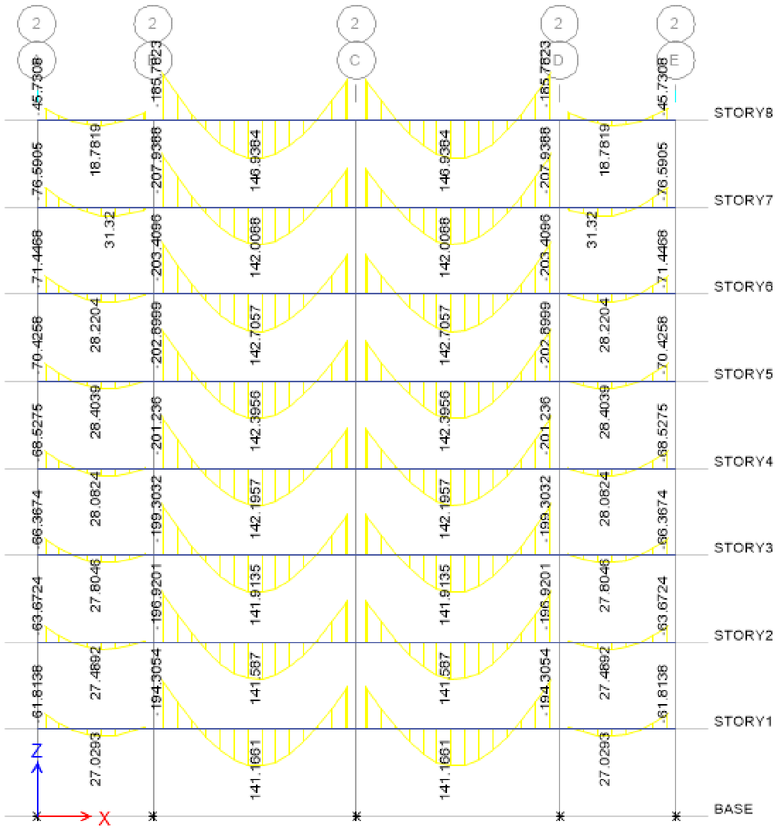


Figure 3. 15. Effort diagram of the fundamental grouping for the central longitudinal frame model foundation beam

To see if additional efforts occur in the beams, calculate the ratio of beam moments from the GF+type and GF grouping in the element sizing model.

	2A	FIELD	2B		FIELD	2C		FIELD	2D		FIELD	2E
Floor 7	1.58		0.68	1.12		0.93	0.93		1.12	0.68		1.58
		1.09			1.06			1.06			1.09	
Floor 6	1.52		-0.46	1.10		0.88	0.88		1.10	-0.46		1.52
		1.15			1.02			1.02			1.15	
Floor 5	1.56		-0.02	1.10		0.88	0.88		1.10	-0.02		1.56
		1.15			1.02			1.02			1.15	
Floor 4	1.63		0.00	1.10		0.88	0.88		1.10	0.00		1.63
		1.16			1.02			1.02			1.16	
Floor 3	1.73		0.06	1.10		0.88	0.88		1.10	0.06		1.73
		1.17			1.02			1.02			1.17	
Floor 2	1.89		0.11	1.10		0.88	0.88		1.10	0.11		1.89
		1.20			1.02			1.02			1.20	
Floor 1	2.13		0.16	1.10		0.88	0.88		1.10	0.16		2.13
		1.22			1.02			1.02			1.22	
Ground floor	3.00		0.17	1.12		0.87	0.87		1.12	0.17		3.00
		1.35			1.02			1.02			1.35	

Figure 3. 16. The ratio of the actual moments in the fundamental grouping+settlement and the moments in the fundamental grouping of the foundation beam model

Following static calculation, it is observed that the absolute settlement will be equal to 2,15 cm, which is less than the limit imposed by the design code (8 cm).

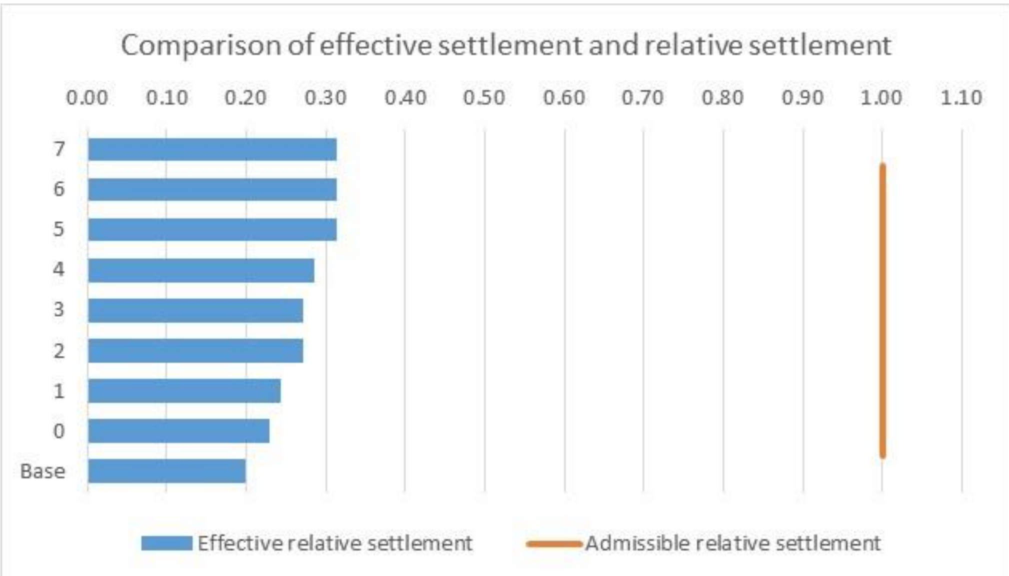


Figure 3. 17. Diagram of relative settlement for each foundation beam model level

<u>Model foundation beams cedar of joint</u>.

For the pillar in axis 2C, a displacement was imposed so that the relative settlement was 1.1%.

Figure 3.18 shows the actual moments in the fundamental grouping taking into account the effect of the settlement. It is noted that the moments in axis 2C are smaller than in axis 2B and 2D, indicating that the pillar in axis 2C is not entirely a joint point, which is more than the rest of the pillars.

.

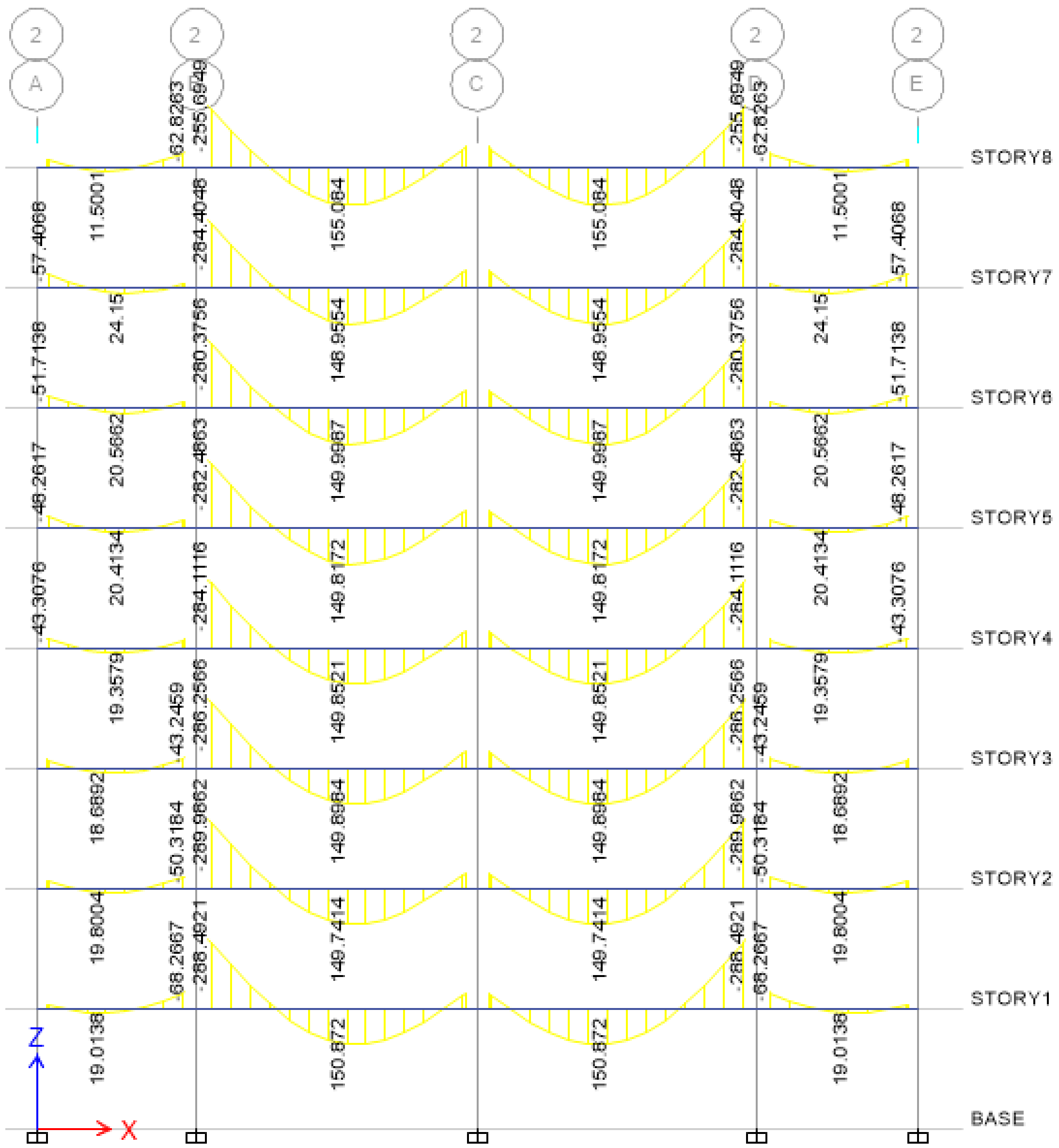


Figure 3. 18. Effort diagram of the fundamental grouping for the central longitudinal frame foundation beam and joint

To see if additional efforts occur in the beams, calculate the ratio of beam moments from the GF+type and GF grouping in the element sizing model.

	2A	FIELD	2B		FIELD	2C		FIELD	2D		FIELD	2E
Floor 7	1.58		0.68	1.12		0.93	0.93		1.12	0.68		1.58
		1.09			1.06			1.06			1.09	
Floor 6	1.52		-0.46	1.10		0.88	0.88		1.10	-0.46		1.52
		1.15			1.02			1.02			1.15	
Floor 5	1.56		-0.02	1.10		0.88	0.88		1.10	-0.02		1.56
		1.15			1.02			1.02			1.15	
Floor 4	1.63		0.00	1.10		0.88	0.88		1.10	0.00		1.63
		1.16			1.02			1.02			1.16	
Floor 3	1.73		0.06	1.10		0.88	0.88		1.10	0.06		1.73
		1.17			1.02			1.02			1.17	
Floor 2	1.89		0.11	1.10		0.88	0.88		1.10	0.11		1.89
		1.20			1.02			1.02			1.20	
Floor 1	2.13		0.16	1.10		0.88	0.88		1.10	0.16		2.13
		1.22			1.02			1.02			1.22	
Ground floor	3.00		0.17	1.12		0.87	0.87		1.12	0.17		3.00
		1.35			1.02			1.02			1.35	

Figure 3. 19. The ratio of the actual moments in the fundamental grouping+tats and the moments in the fundamental grouping model beam of foundation and joint

Following static calculation, it is observed that the absolute settlement will be equal to 2,15 cm, which is less than the limit imposed by the design code (8 cm).

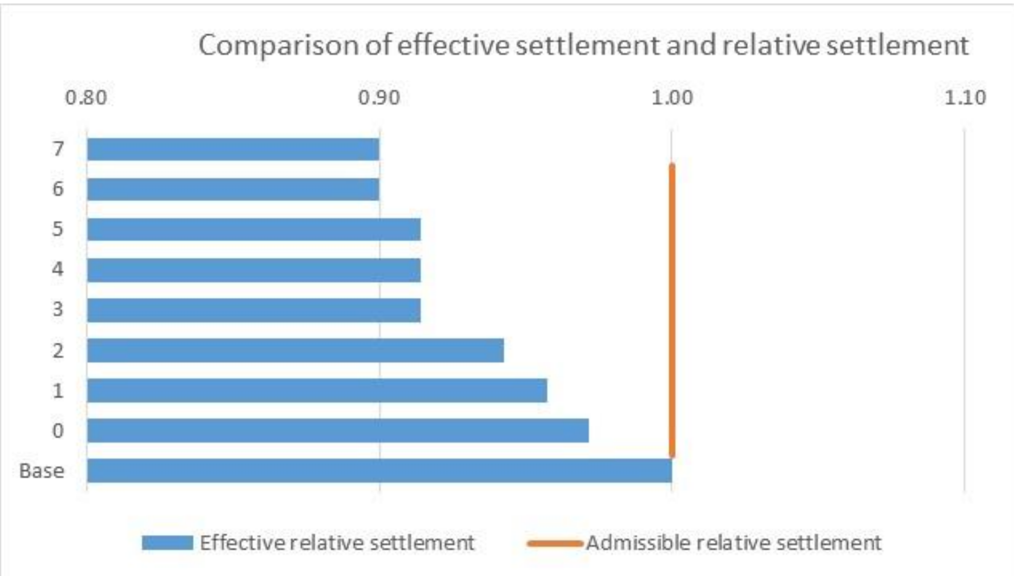


Figure 3. 20. Diagram of relative settlements for each level model foundation beam and joint

It is observed from Figure 3.21 that the plastic joints appeared in the structure even though the relative settlement is the limit imposed by the code.

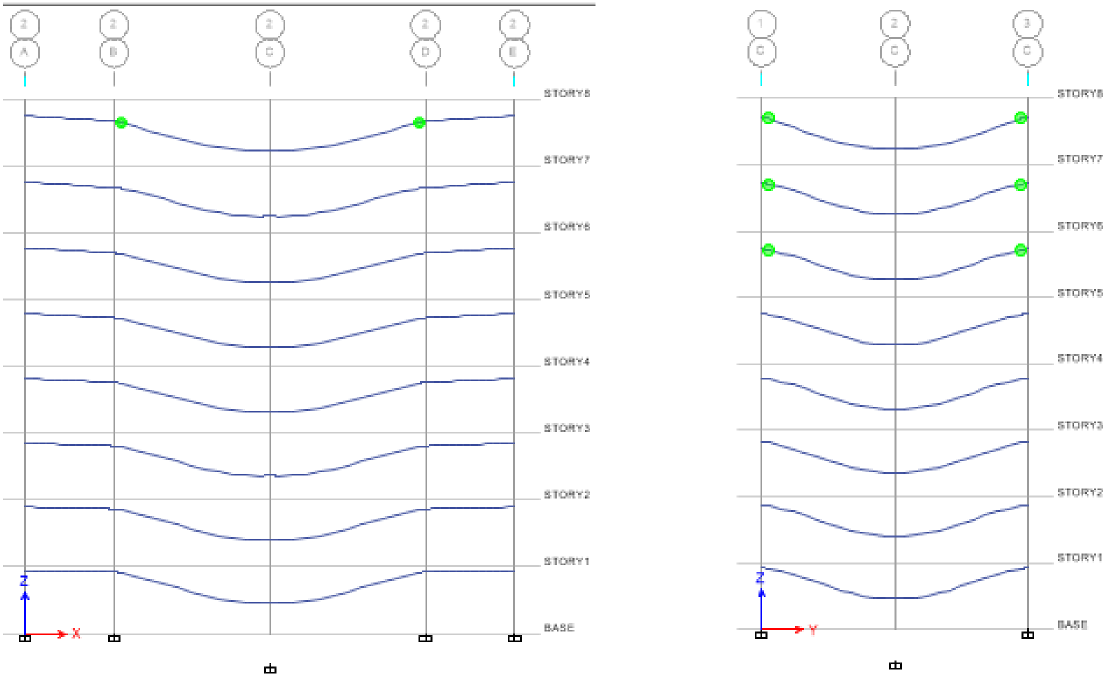


Figure 3. 21 Appearance of plastic joints pattern foundation beam and joint

Eraser.

The eraser was dimensioned according to the design normative of surface foundations, having a height of 80 cm. The hypothesis of the unevenness of the land under construction has been preserved. The interaction between the terrain and the structure was modeled by the resort, the interaction between the land and the structure was modeled by the resort, having an initial bed coefficient of 16800 kN/m^3. Figure 3.22is shows the diagram of moments from the fundamental grouping.

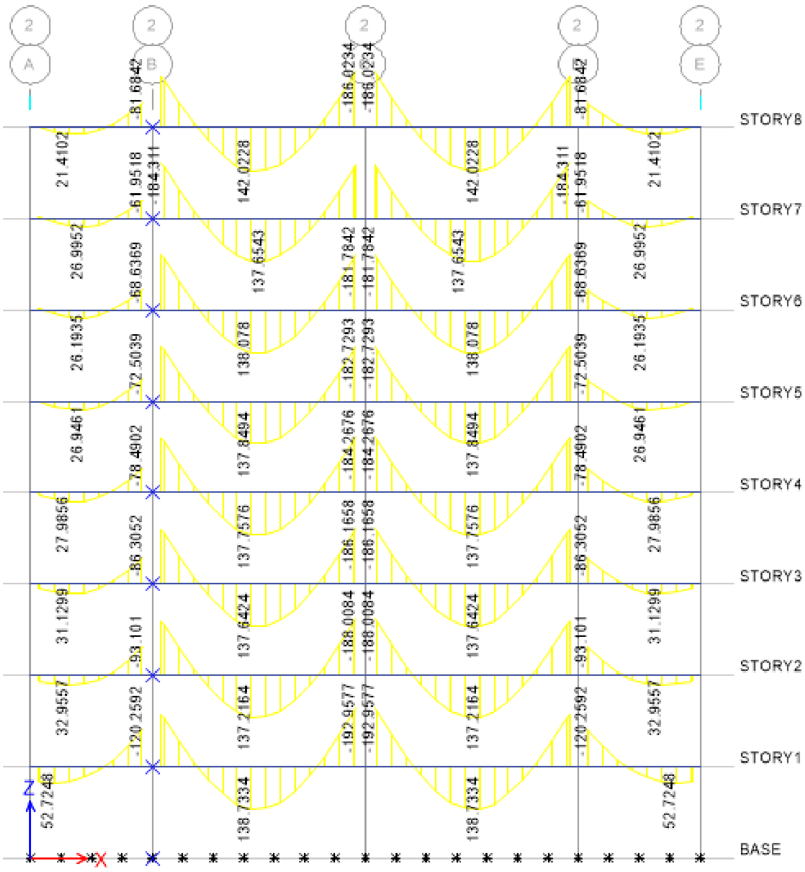


Figure 3. 22. Effort diagram of the fundamental grouping for the central longitudinal frame eraser model

To see if additional efforts occur in the beams, calculate the ratio of beam moments from the GF+type and GF grouping in the element sizing model.

From Figure 3.22. it is noted that this model is very close to the model embedded at the base.

	2A	FIELD	2B		FIELD	2C		FIELD	2D		FIELD	2E
Floor 7	0.03		1.72	1.02		1.07	1.07		1.02	1.72		0.03
		0.57			1.02			1.02			0.57	
Floor 6	0.17		3.38	0.97		1.04	1.04		0.97	3.38		0.17
		0.74			0.99			0.99			0.74	
Floor 5	0.10		2.72	0.98		1.04	1.04		0.98	2.72		0.10
		0.71			0.99			0.99			0.71	
Floor 4	0.01		2.66	0.98		1.04	1.04		0.98	2.66		0.01
		0.70			0.99			0.99			0.70	
Floor 3	-0.13		2.53	0.98		1.03	1.03		0.98	2.53		-0.13
		0.63			0.99			0.99			0.63	
Floor 2	-0.36		2.41	0.99		1.03	1.03		0.99	2.41		-0.36
		0.65			0.99			0.99			0.65	
Floor 1	-0.61		2.29	0.99		1.02	1.02		0.99	2.29		-0.61
		0.59			0.99			0.99			0.59	
Ground floor	-2.47		2.31	0.99		1.02	1.02		0.99	2.31		-2.47
		0.60			0.99			0.99			0.60	

Figure 3. 23. The ratio of actual moments in the fundamental grouping+settlements to the moments in the fundamental eraser model group

Following static calculation, it is observed that the absolute settlement will be equal to 1.3 cm, which is less than the limit imposed by the design code (8 cm). Figure 3.24 notes that the relative settlement falls within the limit imposed by the code.

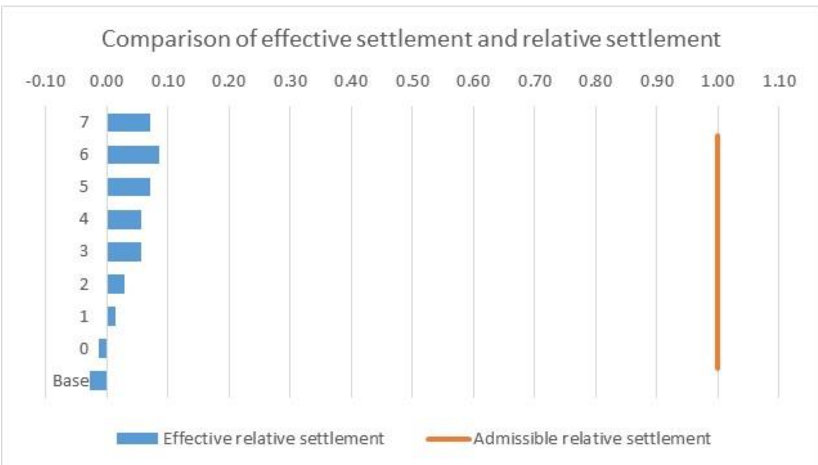


Figure 3. 24. Diagram of relative settlement for each eraser model level

Figure 3.25 shows a comparison between actual relative settlement and allowable relative settlement for different types of foundation solutions.

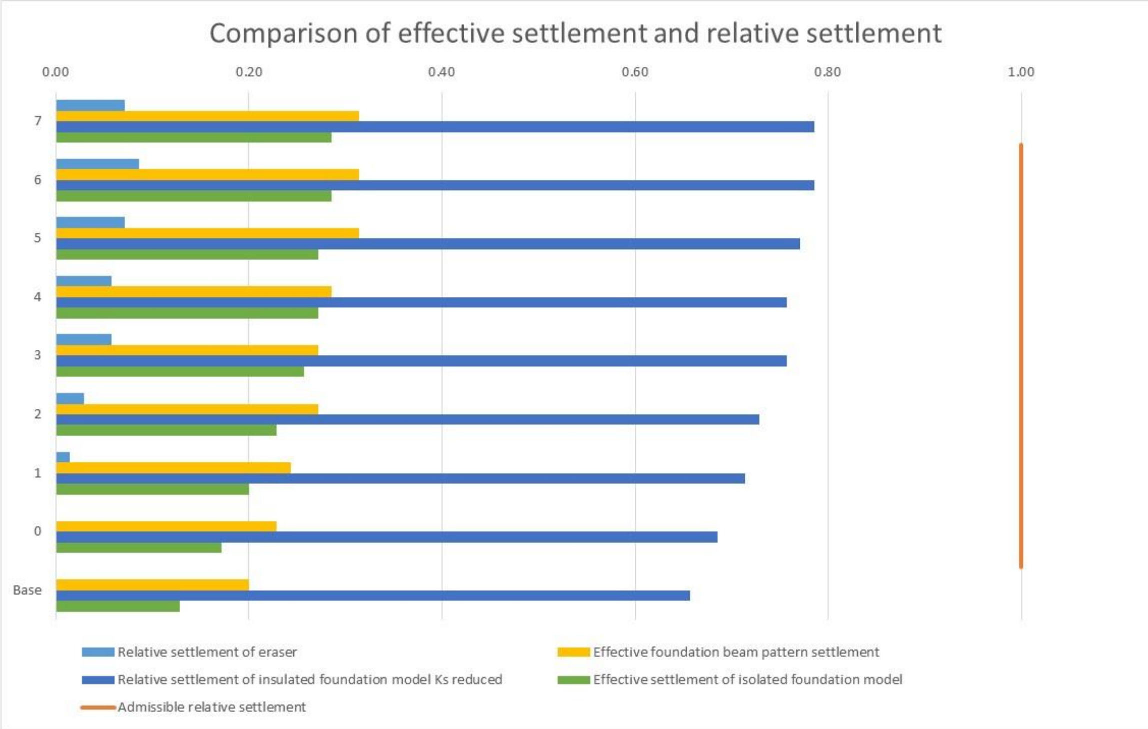


Figure 3. 25. Comparison of relative patterns

Cap.4 Conclusions and recommendations of the Chapter

From what is presented, the following conclusions are drawn from the push-over analyses, the moment charts, and the graphs of the relative settlement:

- The study of the settlement scans is very important because they can lead to both architectural and structural damage to buildings of the type reinforced concrete frames;
- The variation in the mechanical properties of the soil on the site leads to a differentiated settlement. It is noted from figures 3.6, 3.9, and 3.13 that in the case of land with different mechanical properties the relative tasses will be larger. In this situation, the relative settlement is close to the limit imposed by the design code;
- It is noted that a different state of effort appears in the structure compared to the model on which the sizing of the elements was made, namely the model with the structure embedded at the base;
- Even if the relative settlement is less than or equal to the limit imposed by the code (1) it is observed from figures 3.10, 3.14, and 3.21 that plastic joints appear in the structure, as a result of degradation.
- Figure 3.25 shows that the maximum relative settlement occurs for structures with isolated foundations, and the minimum relative settlement occurs in the case of structures with the eraser. It is specified that depending on the rigidity of the foundation the cuts will be different on a case-by-case basis.
- The reduction of the settlement can be achieved either by improving the foundation ground or by increasing the stiffness of the foundation.
- The rigidity of the structure has a great effect on the dedicated settlement. The stiffer the structure, the smaller the differentiated settlement.

In the case of structures in reinforced concrete frames, it is recommended to study differentiated settlements, using the interaction-land structure because they induce a distribution of the state of effort much different from a model in which no account has been taken of the interaction between the ground and the structure.

References

1. Beeby, A. W. (2001). The forces in back props during construction of flat slab structures.
2. Burland, J. B., and Wroth, C. P. (1975). Settlement of buildings and associated damage.
3. Constantin Ionescu, Teoria sistemelor de rezemare pe medii deformabile
4. CR-1-1-3-2012 – Cod de proiectare evaluarea acțiunii zăpezii asupra construțiilor
5. CR-1-1-4-2012 – Cod de proiectare evaluarea acțiunii vântului asupra construțiilor
6. Gerrit Smit, The behaviour of modern flexible framed structures undergoing differential settlement, August 2010
7. Institution of Structural Engineers (1978, 1989 and 2000). *Structure-soil interaction.A state of the art report.* Revised and extended in 1989, 2nd edition August 2000,The Institution of Structural Engineers, London.
8. Institution of Structural Engineers (1994). Subsidence *of low-rise buildings*. The Institution of Structural Engineers, London
9. Kumar, S. (2002). Live loads in office buildings: point-in-time load intensity.
10. Lungu Irina, A. Stanciu, *Fundații*, Editura Tehnică, București, 2006
11. NP 123-2013 Normativ privind proiectarea fundațiilor de suprafața
12. P100-1 2013 – Cod de proiectare seismică
13. Phoon, K. and Kulhawy, F. H. (1999). Characterization of geotechnical variability.
14. Rădulescu Nicoleta, Interacțiunea teren structură, note de curs, 2015
15. Skempton, A. W. and MacDonald, D. H, The allowable settlement of buildings.
16. SR EN 1990-2004 –Bazele proiectării structurilor
17. SR EN 1991-1-1-2004 – Acțiuni asupra construcțiilor
18. SR EN 1992-1-1-2004 – Proiectarea structurilor de beton
19. SR EN 1997-1-2004 – Proiectarea geotehnică
20. SR EN 1998-1-2004 – Proiectarea structurilor pentru rezistența la cutremur
21. Terzaghi, K. (1935). The actual factor of safety in foundations
22. Terzaghi. K. (1955). Influence of geologic factors on the engineering properties of sediments.
23. http://issuu.com/revistaconstructiilor/docs/rc_nr_99_decembrie_2013
24. www.civilengineeringdiscoveries.wordpress.com