Geotechnical and Structural Behavior of Industrial Buildings Located at Valencia Lake Basin. Carabobo State. Republic of Venezuela.

E. A. Guanchez.
University of Carabobo, Venezuela.

SUMMARY:
This research work consists in a geotechnical and structural evaluation of different buildings located at the Industrial Park of Valencia, Venezuela, sited in the basin of the largest country's inland body of freshwater, the Lake of Valencia. The typical geotechnical profile contains sedimentary soils mixed with organic soils, and the site has presence of several geological faults. The results are focused to explain the erratic geotechnical and geological behavior in the area, and include different repairing methods of structural and non structural elements to reduce the existing stresses levels in the elements of different structures. Two foundation systems have been proposed as a solution in these cases, consisting on a sliding membrane for lightweight structures and rigid slabs for heavy structures. The validation process was performed by using a finite element model of the system, applying the ground movements in order to evaluate the stress-strain behavior in the slab foundations.

Keywords: Subsidence, raft foundations, geotechnical, geology.

1. SCOPE OF RESEARCH.

Develop an integrated geophysical, geotechnical and structural evaluation to different cases of industrial buildings located in Lake Valencia Basin. Venezuela.

2. METHODOLOGY.

The steps carried out to complete the present research are detailed as follows:

a) Perform a visual damage evaluation of structural and non-structural elements of the assessed buildings, with a complete description of damages and deformation characteristics. Measure structural systems deformations, pavement slabs settlements and non-structural elements deformations using topographic methods.
b) Determine and characterize the geotechnical profile by performing Standard Penetration Tests (SPT), including seismic refraction tests to estimate shear waves velocity (Vs) and elastic soils properties such as Poisson ratios (μ) and Young modulus (E).
c) Analyze results to explain the erratic geotechnical and geological behavior in the research area and develop different repairing methods of structural and non structural elements to reduce the existing stress levels in structures.
d) Propose a methodology for structural analysis, which considers the effects of ground movements, to be used for foundations design in areas that exhibit this behavior.
e) Validate the proposed solution by performing a finite element analysis, modeling the system and including the ground movements, in order to evaluate stresses behavior in the membranes and "rigid slab" foundation systems.
f) Propose constructive recommendations for foundation systems of industrial buildings located in Lake Valencia Basin.

3. GEOLOGICAL DESCRIPTION.

The Industrial Zone of Valencia is located next to the Lake Valencia, which is the biggest internal freshwater lake of the country. The typical geotechnical profile contains sedimentary soils mixed with organic soils, and important expansive clays levels in some cases. This condition has been related with the presence of different geological faults in the research area (Peeters 1971). The tectonic depression “Valencia Graben” consists in two block failures that slide about WSW to ENE and these are the fault of “Valencia” and the fault of “La Victoria”. The fault zone of “La Victoria” includes regional faulting affecting the southern part of the “Cordillera de la Costa”. There are other associated faults, such as faults “Santa Rosa”, “Tacata” and “Taiguaiguay”. The fault zone of “La Victoria” reaches a long narrow valley between “San Mateo” and “Las Tejerias”. There are two continuing failures across the lake, heading E-NW, located at north of the “Horno Island”. These faults are called this way because of its proximity to the island and the Failure of “Cabrera”, also the main peninsula of the lake. Schubert and Laredo (1979) state that the last movement along the fault zone of “El Horno” occurred during the Pleistocene, while the failure “Cabrera” still has important seismic activity. (See Fig 1)

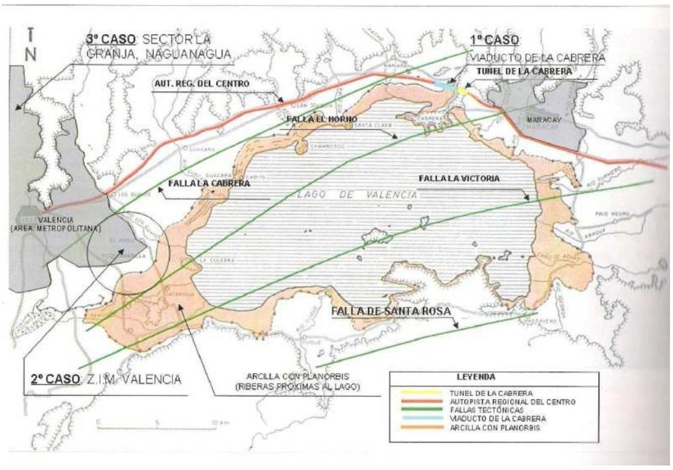


Figure 1. Geological Faults Locations. Lake Valencia Basin

4. REPORTED DAMAGES IN INDUSTRIAL BUILDINGS.

It was performed a visual damage evaluation of structural and non structural elements of the buildings (See Figs 2 and 3). The deformations measurements of structural systems, pavements slabs elevations and non structural elements, were taken using topographic methods.

Figure 2. Excessive Columns Deformations and Differential Settlements in Structures.

Figure 3. Pavements Elevations and Cracking in Pavements Slabs.

5. GEOTECHNICAL RESEARCH.

Different Standard Penetration Tests (SPT) have been performed in order to determine the typical geotechnical profile of the area under investigation. The geotechnical profile is composed by alternating layers of granular and cohesive soils. Silty Sands (SM) are primarily found as a top layer with thickness ranging between 1.50 meters and 2.0 meters. The lithological profile show a predominant presence of Silty Clays (CL) that can reach up to 5 meters in some cases. Deeper layers correspond to Fine Silty Sands (SM) and Silty Clays ranging from Low to Medium Plasticity (CL). (See Fig 4).

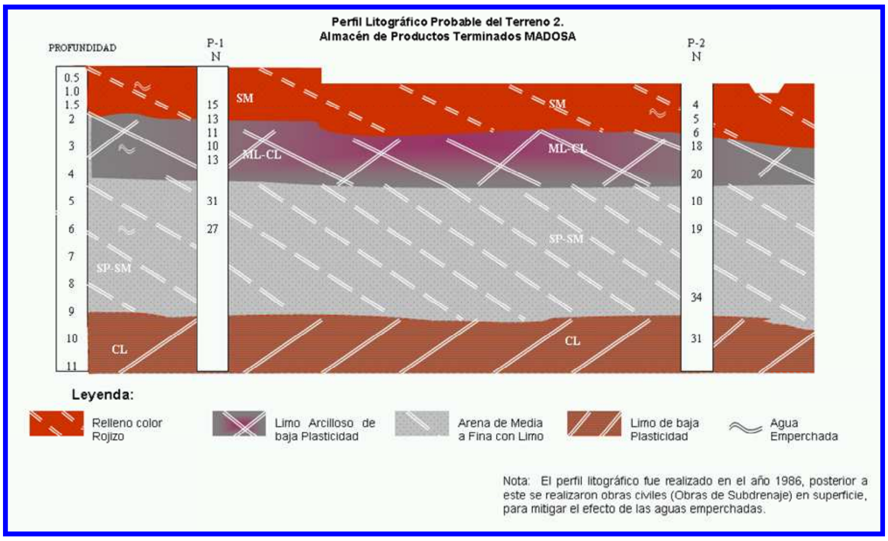


Figure 4. Typical Geotechnical Profile in the Research Area.

6. SEISMIC REFRACTION TESTS.

The seismic refraction identified an interface between two layers. The first one corresponds to a wave velocity between 430 m/s and 540 m/s, and underlying this one another layer was identified with a range of P-Wave velocity from 700 m/s to 1500 m/s. (See Fig 5). The conclusions obtained from the test are stated as follows:

a) “The refraction study shows an interface between two layers, characterized by a dome-shaped geometry with North-South preferential direction. The apex of it is being located under the platform of the industrial plants”.

b) “The elastic modulus ranges match with the values for the lithological types identified with the seismic, mainly composed of sediments with low compaction in the surface area and greater compaction in the deeper layers”.

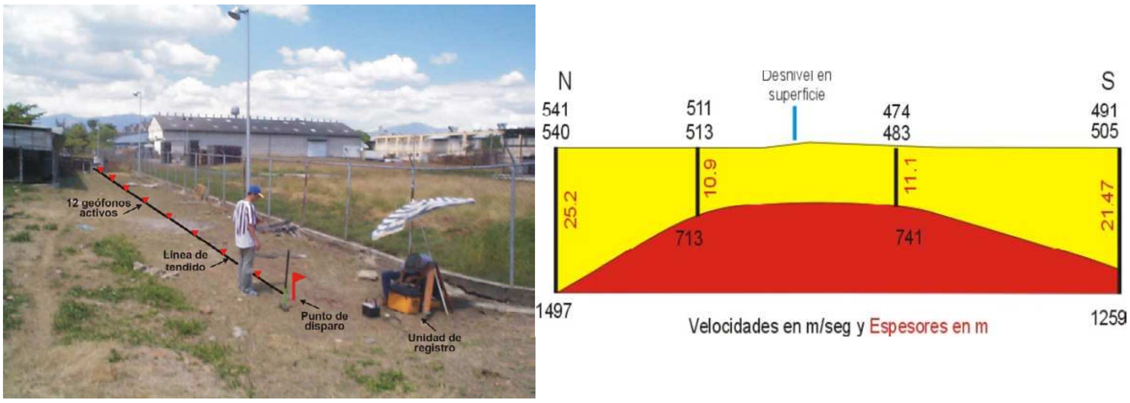


Figure 5. Seismic Refractions Lines used and Soil profile Reported.

7. METHODOLOGY OF EXECUTION OF STRUCTURAL ANALYSIS.

After the topographic survey procedure was possible to know the different movements that have affected the columns and the beams, also evaluating the magnitude of different elevations measured on pavements and floor slabs. By knowing the magnitude of movement of critical points in the structure in both horizontal-in plane directions and also in vertical directions, it was possible to model them the resistant system with the corresponding actions and determine the different stresses levels in the elements subjected to service loads. Therefore it was possible to evaluate their performance based on the presence of failure condition. The structure was analyzed in the same way using the design loads in accordance with current design codes. The structural analysis was performed using the software Structural Analysis Program (SAP 2000). (See Fig 6)

After obtaining the structural analysis results it was possible to provide a diagnosis that allowed to classify the structure based on the existing level of damages. This was used to set up a repairing method for the existing damages, propose different alternatives to dissipate stress concentrations in sectors concerned and reduce progressive increase of damages into the structure at a range of time.

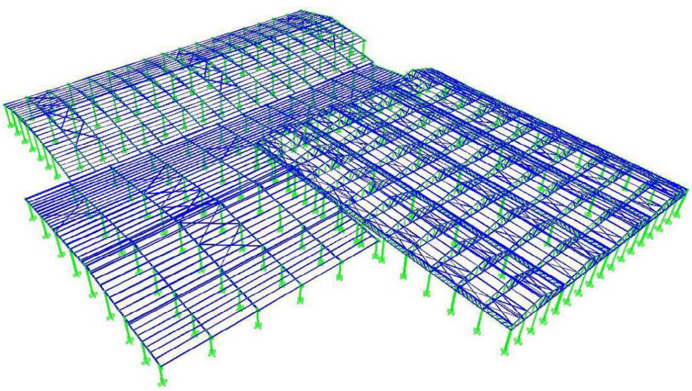


Figure 6. Case of Study. Typical structure in the research area.

The results obtained in structural analysis were as follows:

a) The trusses should be released by placing sliding supports under their end joints. (See Fig 7)

b) The structural elements are subjected to stresses that happen to be greater than those estimated in the original design, causing cracks and important crashes. There were reported important stresses concentration on beam-column nodes.

c) The damage patterns observed were matched to horizontal shallow landslides on the supporting ground structure, as well as the erratic lifting pavement slabs and existing foundation systems. Altimetry survey evaluations in different industrial buildings in the assessed area have reported floor elevations in more than 1.0 meter high in some cases.

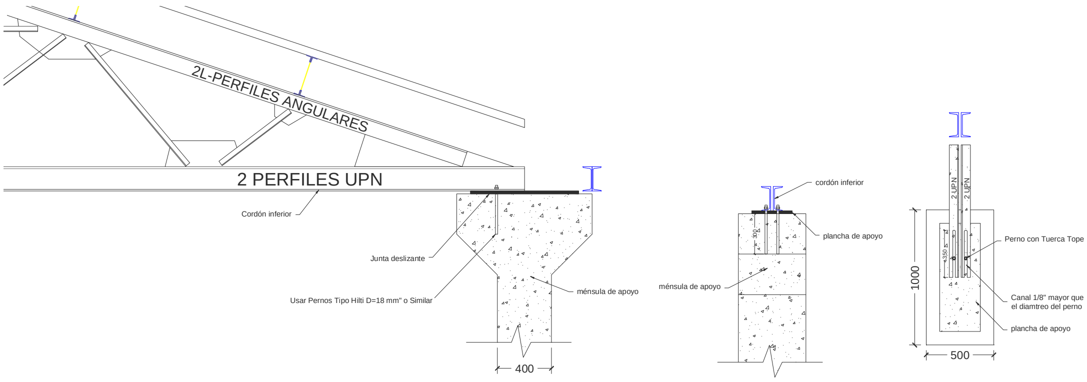


Figure 7. Sliding Support Details for Trusses.

8. DESIGN RECOMMENDATIONS FOR FOUNDATION SYSTEMS OF INDUSTRIAL BUILDINGS LOCATED AT LAKE VALENCIA BASIN. CARABOBO STATE, VENEZUELA.

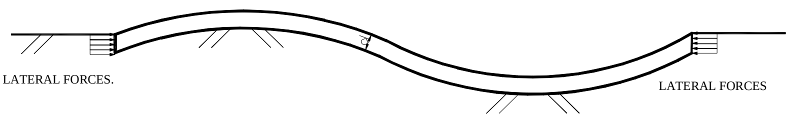
The bibliographical review showed some cases in mining field in countries like United Kingdom with similar characteristics than the case assessed in this study like those in Northumberland Coalfield and Coal Strike (1926). The solutions in those cases include the application of several structural improvement techniques to mitigate the effects of subsidence phenomena while the mining activities are performed, such as structural protection retrofitting.

The general principles for structural protection recommended in the Subsidence Engineers' Handbook, published by the National Coal Board of England (1975) are summarized as follows:

a) Structures should be completely rigid or completely flexible. Simply supported spans and flexible superstructures should be used whenever possible.
b) The shallow raft foundation is the best method of protection against tension or compression strain in the ground surface.
c) Large structures should be divided into dependent units. The width of the gaps between the units can be calculated from previous knowledge of the tensile ground strain derived from the predicted ground subsidence.

According to the same Handbook, raft foundations should be as shallow as possible, preferably on the surface, so that compressive strains can take place beneath them instead of transmitting direct compressive forces to their edges, and they should be constructed on a membrane so that they will slide as the ground movements occur beneath them. According to these recommendations, it is possible to avoid the transmission of lateral forces to the raft. (See Fig 8)

NOT RECOMMENDED BEHAVIOR.



RECOMMENDED BEHAVIOR

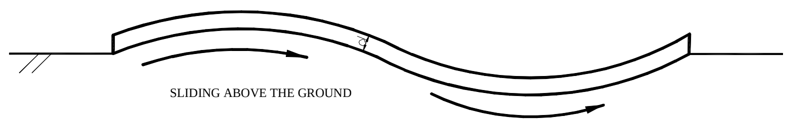


Figure 8. Sliding Rafts.

For light structures, raft foundation should be constructed on a membrane so that they will slide as ground movements occur beneath them. It is then only necessary to provide enough steel reinforcement in the rafts to resist tensile and compressive stresses set up by friction in the membrane. In these cases it is not usually practicable to build the raft any smaller than the plan area of the building. This condition induces a reduction in the bearing pressure to the minimum value, allowing the raft to slide freely on the ground. In the case of heavy structures it is desirable adopt the highest possible bearing pressure, so that the plan dimensions of the raft are the smallest possible. By this means the lengths of raft acting as a cantilever at the hogging stage or as a beam at the sagging stage are also a minimum. (Mauntner, 1948) has analyzed these support conditions as follows:

8.1 Bearing Pressure at Hogging Stage (See Fig 9).

8.1.1. When "l" is greater than b/4.

$$q_{máx} = \frac{4\,q\,b}{3(b-2l)} \tag{8.1}$$

8.1.2. When "l" is less than b/4.

$$q_{máx} = q\left\lfloor 1 + \frac{3\,b\,l}{(b-l)^2} \right\rfloor \tag{8.2}$$

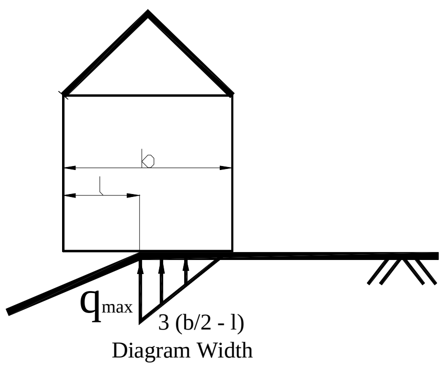


Figure 9. Bearing Pressure at Hogging Stage.

8.2 Bearing Pressure at Sagging Stage (See Fig 10).

8.2.1. For free support.

$$q_{máx} = \frac{qb}{(b-l)} \tag{8.3}$$

According to (Maunter, 1948), yielding will take place if "l" is greater than:

$$q\left(1 - \frac{4\,q}{4q_f}\right) \tag{8.4}$$

Where:

b = length of the structure in the vertical plane under consideration.
q = Uniformly assumed design pressure in disturbed ground.
l = Unsupported length for cantilevering of free support.
q_f = Ultimate bearing capacity.

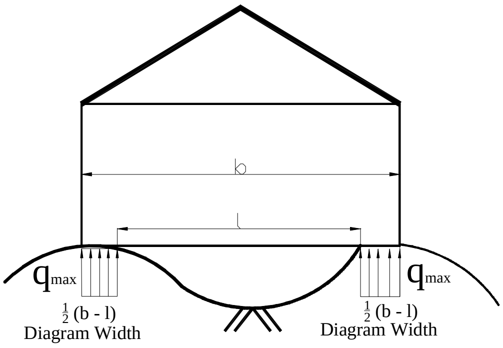


Figure 10. Bearing Pressure at Sagging Stage.

As soon as the value of q_{max} approaches the ultimate bearing capacity of the ground, yielding of the ground will occur, causing the structure to tilt in the cantilever case and to settle more or less uniformly in the free support case. In both cases the effect is an increment in the supporting area given to the underside of the foundation, hence reducing the length of the cantilever –or the span of the beam– and also reducing the stresses in the foundation structure or superstructure. The design bearing pressure should be kept as close as possible to the ultimate bearing capacity q_{max}. It is necessary to analyze different positions of subsidence wave and calculate the worst position of support for the structure. In order to mitigate the effects of subsidence waves and the different movements of the ground, some recommendations and design criteria are listed related to the construction of foundation systems of slightly loaded structures located in Lake Valencia Basin. All of these recommendations are based in the "Subsidence Engineers' Handbook" (1975) and have been adapted according to observed behavior of different structures located in the assessed area:

1. Place a 150 mm to 200 mm layer of compacted granular base on the ground surface. Place a layer of plastics geotextiles over the granular sub-base layer to act as a surface for sliding.
2. Provide steel reinforcement to resist frictional forces acting on the underside of the slab as it slides over the sub-base.
3. The frictional force may be in transverse or longitudinal direction, and may be taken as the product of half of the weight of structures and the coefficient of friction between slab and granular material.
4. The permissible tensile stress in steel reinforcement may be taken as 0.5 fy (fy = Minimum Yielding Stress). The permissible compressive stress on concrete may be taken as 0.6 f'c (f'c = Maximum Compressive Strength).

5. If single layer reinforcement, place it in the centre of the slab to allow both for hogging and sagging of the ground surface, designing the thickness of the raft and the rebar percentage to allow deformations in the raft under vertical movements.
6. In order to improve the punching shear resistance of the membrane may be necessary to increase the thickness of the slab only in the columns supports. This should be done in the upper side of the raft.
7. Although the design is made according to membrane stresses, the flexural stresses must be analyzed in every column support.
8. A provision should be made at the ends of the foundation trenches for longitudinal movements. This avoids the concentration of lateral forces in the raft ends.

The large settlements commonly observed in the evaluated area force appreciable deflections in the rafts. This results in the need for applying some appropriate precautions in the superstructure design procedure, in order to provide suitably strengthened raft. Foundation trenches should be used in order to avoid the concentration of lateral forces at the ends of the rafts. Piled foundations should not be used under any circumstances in the Lake Valencia Basin for industrial buildings, since horizontal forces will either shear through the piles or else cause failure in tension of the tie beams or raft connecting the heads of the piles.

9. VALIDATION OF PROPOSED METHODOLOGY (FINITE ELEMENT ANALYSIS).

Different finite element models were developed, loaded and analyzed in order to validate all the recommendations viewed, by using CSI SAFE Software v12.3.1. The analyzed models are described below:

9.1. Raft Foundation for light structures in weak soil with subsidence.

The primary step consisted in a thin raft foundation model slightly loaded with no subsidence effect. The model with subsidence effect was developed including the soil effect in some locations; therefore the raft could behave as a simply supported element. In this condition the deformations and stresses in the foundation system due to gravitational and horizontal loads are determined. (See Fig 11)

The model properties and loading criteria are summarized below:

- Concrete Compressive Strength (f`c) = 250 k/cm^2
- Steel Minimum Yielding Stress (fy) = 4200 k/cm^2
- Thickness (cm) = 15.
- Dead Load = 300 k/m^2.
- Live Load = 100 k/m^2.
- Horizontal Loads = The horizontal loads were estimated such as could be higher than the friction resistance to simulate the horizontal raft movement.
- Modulus of Subgrade Reaction = 0.5 k/cm^2 (Weak Soil).

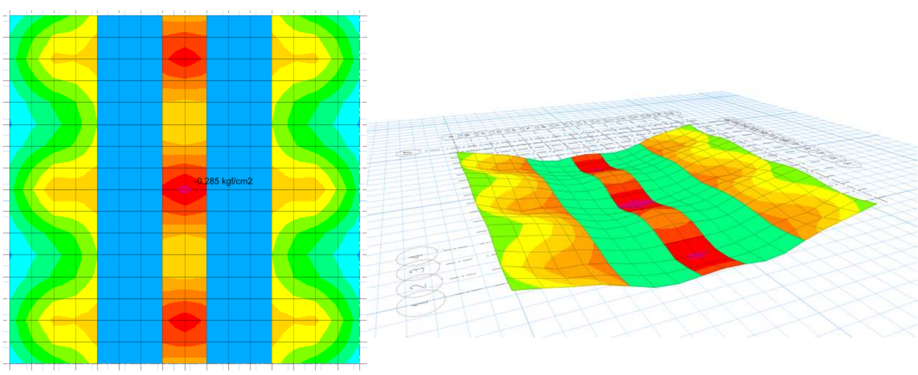


Figure 11. Raft Model with subsidence effect (Maximum Bearing Pressure). Membrane Raft.

The maximum bearing pressure calculated using this model was 0.285 kg/cm^2, in the central support. There is an average deformation in the supports of 5.50 millimeters and 3.00 millimeters in the rest of the raft. The subsidence effect increases the pressure on the ground in the contact areas where the soil was modeled. However, this increment does not represent a yielding condition in the ground due the extension of the raft and the behavior of the membrane. Due to the increasing deformations on the raft foundation, the flexural stresses levels reported by the analysis kept low in the supports and the spans. An additional uniformly distributed load was incorporated in the same model for a second analysis, applied over the raft to simulate the superficial service loads. In this case the bearing pressure was increased proportionally according the new value. However, the flexural stress levels are kept low. The tension stresses in the spans were lower than 2 √f´c = 31.62 k/cm^2. This indicates that the main stresses are primarily axial, which allows the membrane to deform together with the ground support. According to *Mauntner* (1948), for this support condition the bearing pressure could be estimated as $q_{máx} = \frac{qb}{(b-l)}$ (Simply supported spans). Based on this approach the estimated pressure is 1.60 kg/cm^2, which is very different to the 0.32 kg/cm^2 value reported in the FEM analysis. This situation is explained considering the behavior in the structural element, which corresponds to a membrane deforming with no restrictions along the ground, allowing the tension forces to be carried through the element by the reinforcement steel supplied in the raft. By enabling the deformations at the ends of the rafts it is possible to reduce the lateral forces due to passive earth pressures.

9.2. Raft Foundation for heavy structures in weak soil with subsidence.

The primary model was developed as a thick raft foundation heavily loaded with no subsidence effect. The model with subsidence effect was assessed by using the same criteria than the light structure case in relation to supporting soil effect and evaluation parameters. (See Fig 12)

The model properties and loading criteria are summarized below:

- Raft Area (m^2) = 14x14. (The Smallest possible to adopt the highest bearing pressure)
- Concrete Compressive Strength (f`c) = 250 k/cm^2
- Steel Minimum Yielding Stress (fy) = 4200 k/cm^2
- Thickness (cm) = 60.
- Dead Load = 1500 k/m^2.
- Live Load = 500 k/m^2.
- Modulus of Subgrade Reaction = 0.5 k/cm^2 (Weak Soil).

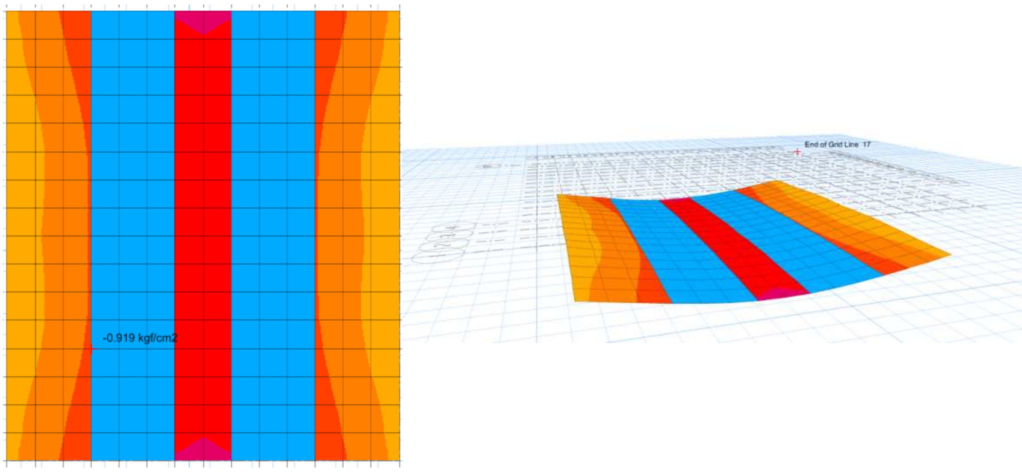


Figure 12. Raft Model with subsidence effect (Maximum Bearing Pressure). Rigid Raft.

The maximum bearing pressure reported by the analysis results was 1.00 kg/cm^2 in the central support and 0.80 kg/cm^2 in the lateral supports. There is an average deformation along the whole foundation of 2 cm. The subsidence effect increases the pressure on the ground in the contact areas where the soil was modeled, and the deformations are almost constant along the raft, reaching the ground yielding condition. The soils located in the evaluated area reach no more than 1.0 k/cm^2 of bearing capacity. According to *Mauntner* (1948), for this support condition the bearing pressure could be estimated based on Eq. (8.3), resulting in 0.80 k/cm^2. This value is very similar to the one reported in the FEM, ranging from 0.80 k/cm^2 to 1.00 k/cm^2.

This situation is explained by comparing the model's behavior with a rigid slab with uniform deformation above the ground support and bearing pressures almost constant along the whole foundation. These stresses are very close to yielding ground condition; therefore the "simply supported condition" and "cantilevering condition" could be kept for reduced time periods, avoiding the cracking of the foundation system. As in the membrane model, enabling the deformations at the ends of the rafts is possible to reduce lateral forces due to passive earth pressures.

10. GENERAL CONCLUSIONS.

a) The geotechnical profiles indicate the presence of low compacity soils with important sediments thickness. This information was determined by standard penetration tests (SPT) and seismic refraction tests.
b) There is an important evidence of subsidence phenomena due to the variation of ground water levels in the soils of the Lake Valencia Basin.
c) The observed damages are caused by horizontal and vertical displacements in hogging and sagging stages.
d) The geophysical tests results reported an interface between two layers, characterized by a dome-shaped geometry. The upper layer is formed by very low density soils with P-Wave velocity values ranging from 430 m/s to 540 m/s, and the underlying layer showed high density with a range of P-Wave velocities from 700 m/s to 1500 m/s.
e) It was possible to validate the proposed methodology for design of foundation systems based on the Mauntner (1948) equations and the finite element models for both light and heavy structure cases in presence of subsidence phenomena.
f) The proposed methodology consisted in an improvement of the foundation system behavior through rafts, designed to absorb differential movement resulting for both consolidation of fill and ground movements due to the presence of geological faults.
g) The proposed design methodology for foundation systems in the studied area is an alternative that should be applied in order to mitigate the effect generated over these structures by the existing geological and geotechnical behavior.

REFERENCES

Guevara, M. y Martínez, M (2005) Caracterización y Diagnostico de los Problemas en la estructura física de una empresa, domiciliada en la Zona Industrial Sur de Valencia. Trabajo Especial de Grado para optar al Título de Ingeniero Civil, Departamento de Ingeniería Civil, Universidad de Carabobo, Valencia, Venezuela.

Longwell, Ch. y Flint, R (1974) Geologia Fisica. Editorial Limusa, Mexico.

Manrique, A. y Ramos, D (2005) Estudio de las posibles causas de daños estructurales en una planta industrial, ubicada en la Zona Industrial Municipal de Valencia. Trabajo Especial de Grado para optar al Título de Ingeniero Civil, Departamento de Ingeniería Civil, Universidad de Carabobo, Valencia, Venezuela.

Mauntner, K.W. (1948). Structures in areas subjected to mining subsidence. *2nd International Conference on Soil Mechanics Rotterdam*. **Proceedings Vol. 1**.

Ramirez, O. (2006) Fundaciones en las Riberas del Lago de Valencia. *Ingenieria Forense y Estudios de Sitio*. Editorial Banesco, Caracas, Venezuela. **Vol I**, 177-196.

Tomlinson M.J (2001) Foundation Design and Construction, Pearson Education, Ltd.