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MULTITECHNICAL APPROACH FOR DAMAGE ASSESSMENT AND REINFORCEMENT OF BUILDINGS LOCATED ON SUBSIDING AREAS: STUDY CASE OF A 7-STORY RC BUILDING IN MURCIA (SE SPAIN)

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Abstract

The work presented herein proposes a multitechnical methodology for damage assessment and reinforcement of buildings located on areas affected by land subsidence induced by water withdrawal. The proposed methodology is illustrated by a comprehensive damage assessment and subsequent reinforcement of a 7-story reinforced concrete building located in the city of Murcia (SE Spain). Construction took place in the 1980's and the building was severely damaged by differential settlements caused by land subsidence throughout time. The building suffered an important tilting, presenting maximum settlement and tilt values of 260 and 177 mm, respectively, which considerably reduced its habitability and security conditions. Damage began to manifest in 1995, coinciding with an intense drought that affected the Murcia area between 1991 and 1995. Average piezometric level decreases of at least 8 m were verified, reaching 10 m in the nearby areas of the building. These piezometric level decreases caused important consolidation settlements that seriously damaged the structure of the building. The proposed multitechnical damage assessment methodology was used to characterize the causes of damage, design reinforcement actions, and carry out subsequent monitoring to guarantee the structural stability of the building.

Keywords: Building damage; differential settlements; ground subsidence; building reinforcement; crack map; multitechnical approach.

1. Introduction.

Land subsidence caused by groundwater extraction is a human induced hazard that affects many cities and regions in the world (e.g. Hu et al. 2004; Phien-wej et al. 2006). Land subsidence can damage urban infrastructures, buildings and generally any structure in contact with the ground, due to its deformation. Damage is higher if ground deformation occurs differentially. Many cities around the world are located in subsiding areas, and therefore the buildings located within these areas are affected by ground movements that translate into loss of functionality, damage and in extreme cases, building failure (López Gayarre et al. 2010). Study and evaluation of such damage is especially relevant, with several published works regarding damage assessment schemes for buildings affected by subsidence (Cooper 2008; Del Soldato et al. 2017; Engineers 2000; Feng et al. 2008; Howard et al. 1993; Namazi and Mohamad 2013). These damage assessment schemes are based on are based on damage levels established by observation, and enable to define the degree of damage that a structure has suffered, according to a series of categories defined a priori. These categories relate the degree of damage with structural risk, without delving into more complex analyses on the techniques involving the study or mechanisms of damage.

Damage assessment schemes must be complemented by reinforcement or underpinning systems as well as concrete methodologies that collect data on the possible damage mechanisms, improving investigation and evaluation of such damage.

The overarching aim of this work is to propose a multitechnical approach for damage assessment and reinforcement of buildings located on subsiding areas. A study case is utilized to demonstrate the applicability of the method, consisting of a building with basement and 6 floors located in the urban area (Pintor Sobejano Street) of Murcia (SE Spain), affected by land subsidence. The building suffers severe problems due to differential settlements that endanger its habitability conditions.

The building is located on a subsiding area affected by groundwater withdrawal, known as the Vega of Murcia (Herrera et al. 2009; Jaramillo and Ballesteros 1997; Justo and Vázquez 2002; Rodríguez Ortiz and Mulas 2002; Tomás 2009; Tomás et al. 2005).

Subsidence involves the settlement of ground over a wide area, caused by natural or human-related factors (Corapcioglu 1984; Poland 1984). Subsidence can be classified according to the causing mechanisms (Scott 1978), being the subsidence caused by the extraction of fluids from the subsoil is one of the most important types of subsidence, and it affects important cities around the world (Galloway and Burbey 2011; Poland 1984). The extraction of water from an aquifer usually leads to a drop in the piezometric level, which can induce the consolidation of the affected layers, generating ground settlements. The case of Murcia was the first documented subsidence case in Spain related to a decrease in the piezometric level of the aquifer system called Vega Media and Baja del Segura. Land subsidence appeared during a prolonged drought period, which caused an overexploitation of the aquifer system.

Between 1975 and 1992, the piezometric level of this aquifer could be considered roughly constant (except for some variations under than 2-3 m). Between 1992 and 1995, a considerable decrease in the piezometric level was observed, with maximum values from 7.6 m to 10.8 m (Aragón et al. 2006). Data on subsidence settlements in Murcia during the piezometric crisis of the 1990s showed maximum values of up to 30 cm (Jaramillo and Ballesteros 1997). These settlements occurred mainly between 1992 and 1995, in the city of Murcia, and affected more than 150 buildings and other structures, entailing costs over 50 million Euros (Mulas et al. 2003; Rodríguez Ortiz and Mulas 2002).

2. Common damage in buildings located on subsiding areas due to groundwater withdrawal.

The damage suffered by buildings located on subsiding areas varies according to different factors related to the building itself and to the subsoil, such as load asymmetry, coexistence of different types of foundations, variation of the thickness of soft soil layers, to name a few.

The relative stiffness of concurrent elements (soil, foundation and superstructure) can also influence subsidence. Subsidence causes damage to buildings by the following mechanisms:

- Absolute settlements, which are always present and are precursors of damage (although this can be negligible);

- Differential settlements, which generate significant damage in the case of isolated and poorly rigid foundations and / or significant heterogeneities in the ground or the loads.
- Tilts, in relatively rigid foundations with also rigid structures. Tilts can be quite spectacular without affecting excessively the mechanics of the structure (but functionality is seriously affected).
- Structural failure of the foundation, in very specific cases only, such as end-bearing piles with a low failure safety factor, when a great negative skin friction is generated, or in foundations with non-isolated but relatively flexible, when deformations are significant.

Table 1 shows the types of foundations usually utilized, in decreasing order of damage probability.

	Type of foundation	Relative damage			
		Absolute settlements	Differential settlements	Tilts	Structural failure of the foundation
Less vulnerability	Isolated footings	xxxxx	xxxxx	xx	x
	Friction piles	xxxxx	xxxxx	xx	x
	Stripped footings	xxx	xxx	xxx	xxx
	Flexible slabs	xxx	xxx	xxxx	xxx
	Rigid slabs	xxx	x	xxxxx	xx
	End bearing piles	x	x	x	xxx

Table 1. Vulnerability to subsidence, for the most common types of foundations. Magnitude of damage: ***xxxxx*** *very important,* ***xxxx*** *important,* ***xxx*** *intermediate,* ***xx*** *reduced,* ***x*** *negligible.*

3. Description of the building.

The gross floor area of the building is 1861 m^2 (Figure 1). This area is fully occupied by parking in the basement, and is divided into two different zones, starting from the ground floor: a) the first zone is founded on isolated footings and has not been strongly affected, and b) the second zone presents six stories and is founded on reinforced concrete ribbed raft foundation, 30 cm height with 0.50 x 0.80 m (width x height) beam stiffeners, aligned with the porticoes parallel to the main façade.

Regarding the structural disposition, the two areas are divided by a transverse expansion joint that separates the building into two independent structures. The structure presents one-way spanning slabs of prestressed semi resistant joists and precast concrete hollow flooring bricks, supported by 30 cm height reinforced concrete beams, parallel to the main façade and over columns of the same material. The asymmetric shape of the building results in that the clear spam between columns can present heterogeneous values, ranging from 4 m to almost 7 m in the third portico. This fact, together with the difference in height between the two areas of the aforementioned building, causes asymmetry in the distribution of loads, where the most heavily-loaded columns are those surrounding the South façade.

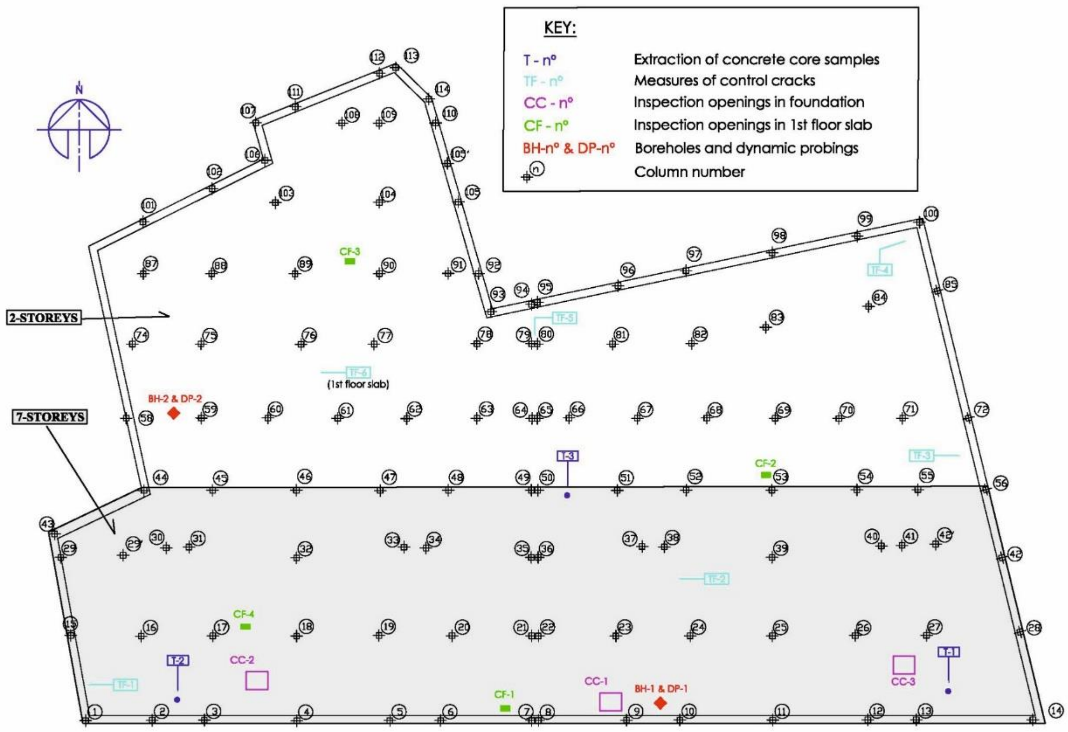

Figure 1. Basement plan of the building with the location of the tests developed. The shadowed area corresponds to the 7-storey zone of the building.

4. Forensic investigation.

Forensic investigation started in June 2008 and concluded in March 2013, when structure monitoring was carried out, comprising four groups of actions:

- Those related to the verification, inventory and classification of the damage observed during visits to the building.
- Works carried out to determine the geotechnical characteristics of the foundation ground of the building.
- Those related to structure, consisting of the measurement of deformations, installation of crack monitors, extraction and testing of concrete test specimens and the execution of openings in foundation and floor slabs.
- Those related to structure monitoring after underpinning, which will be discussed in detail in section 7.

Each action carried out is described next.

4.1. Building inspections. Observed damage.

The inspections revealed damage due to an important tilt of the building towards its main façade (South) simultaneously to the existence of various crack patterns of different elements (e.g., widespread cracking of the concrete slab and walls of the basement, isolated cracking of some vertical and horizontal elements, etc.).

The tilt of the main façade is noticeable to the naked eye because, at the height of the rooftop floor, 115 and 177 mm displacements have been registered in the SW and SE directions respectively, with respect to the base. These displacements are also evident when comparing the adjacent façades vertically (Figure 2a).

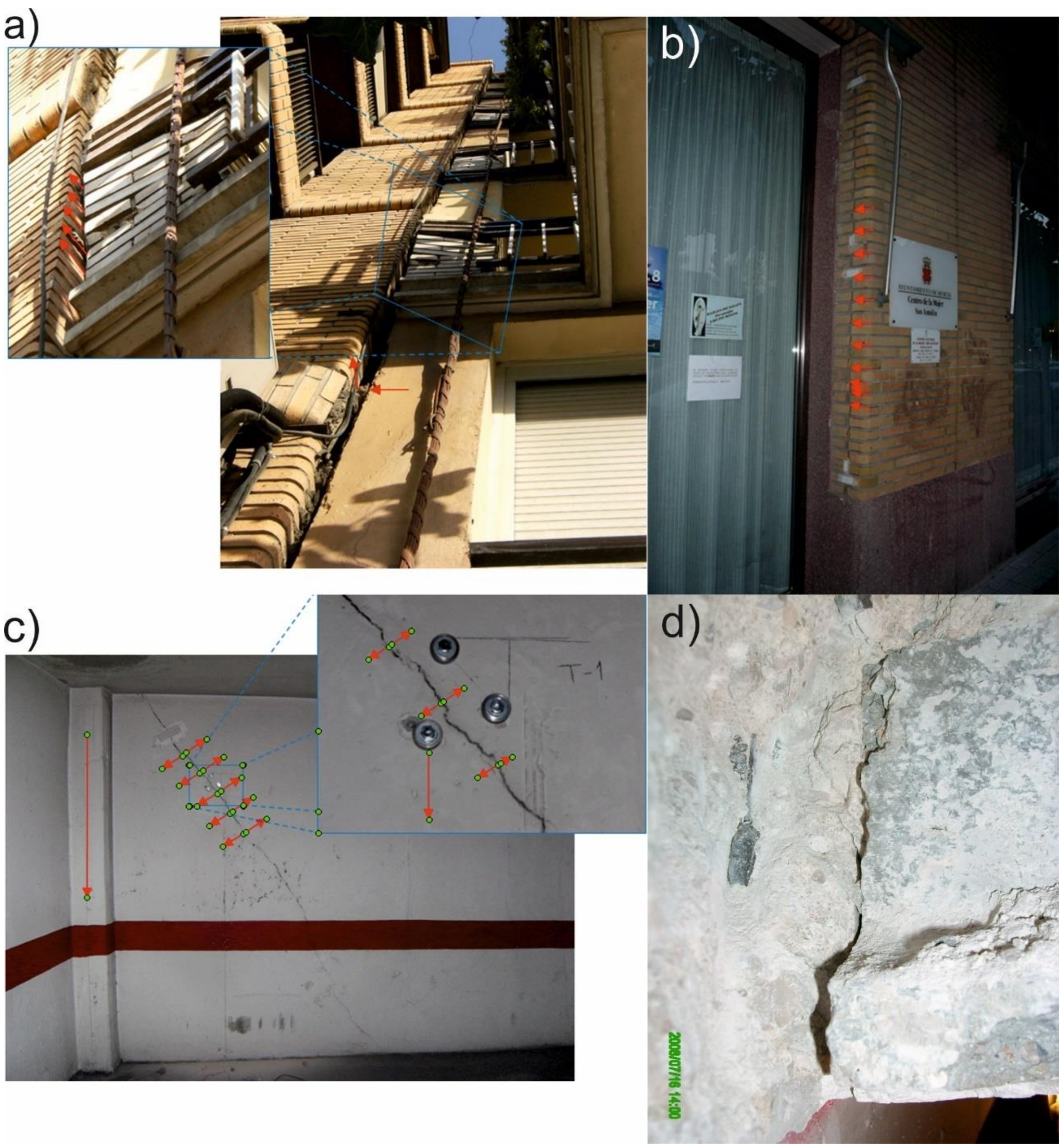

Figure 2. a) Details of the collapse of the study case building and of the mismatch with the adjacent building due to interaction between buildings. b) Vertical cracking of façade bricks due to excessive compression stress. c) View of the crack on the South end of the West wall and detail of the pins installed for crack control. d) Inspection opening on the first-floor slab (code R-2), where it was verified that the semi resistant joist is suspended and therefore does not fulfil its objective.

Vertical cracks were verified in the façade bricks of some walls, which started at street level until reaching the second-floor slab (Figure 2b).

In the basement, the walls perpendicular to the façades present 45° sloping cracks increasing in height and opening towards the main façade (Figure 2c). Still in the basement, large sections presented zigzagging cracks at the base of the first-floor slab, in the joist-beam embedment area (Figure 2d), as well as some fine rectilinear fissures (transverse lines) between the porticoes. The slab of the basement (parking) also presented generalized cracking, where longitudinal cracks can be highlighted, practically parallel to the main façade, running from one wall to the opposite, coinciding with the equidistant space between the different porticoes. Figure 3 shows a detailed map of the cracks observed in the basement. There are other less important cracks for which description is omitted due to the negligible contribution herein.

Construction of the building began in September, 1983 and lasted until April, 1986. The first visible damage appeared in 1995, coinciding with the drought that occurred in the Vega of Murcia between 1991 and 1995 .

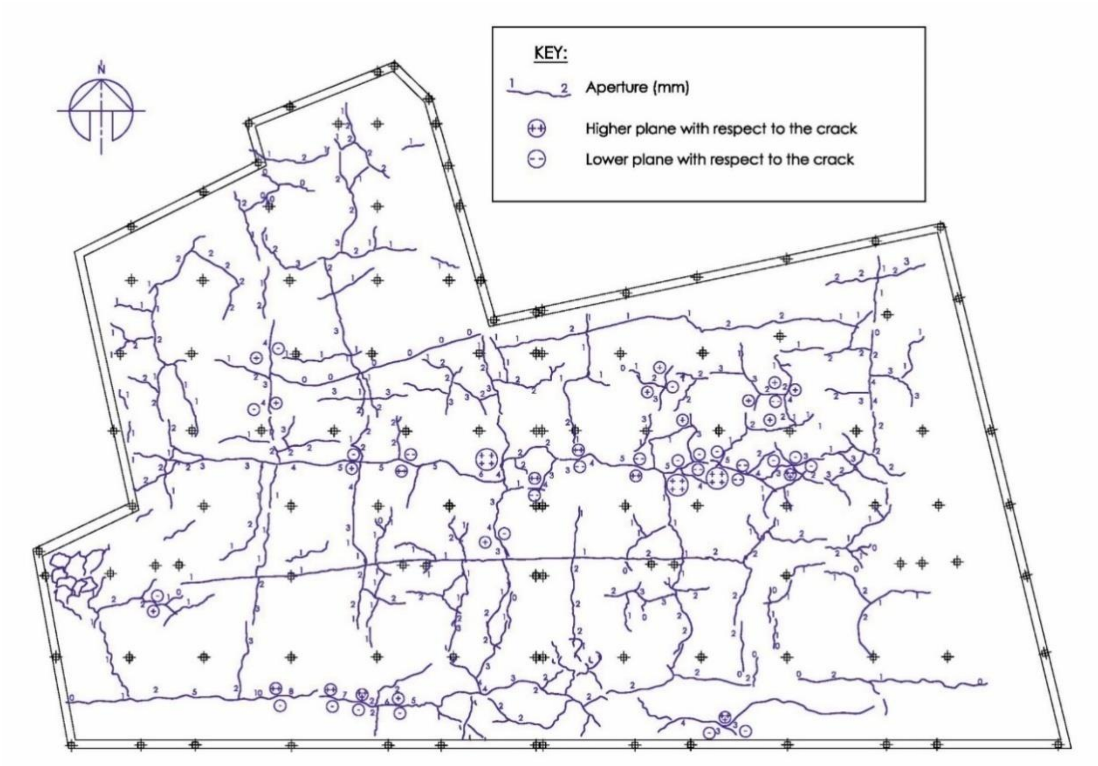

Figure 3. Map of the cracks detected on the basement floor.

4.2. Geotechnical conditions.

Two boreholes were made by a rotating drill system, with continuous core extraction (location shown in Figure 1) to determine the geotechnical conditions of the site. Investigation continued with two dynamic super heavy probes (DPSH). The most relevant properties of each geotechnical unit are shown in Table 2.

During the execution of boreholes (July 3, 2008), the water table was located at 10.5 m below the street level. Nearby municipal wells provided information on the recent variations of the piezometric level of the zone: on average, that the position of the water table in 1983 was approximately 3 m below the street level, and in 1995, was 13 m below the street level.

Figure 4 shows the geotechnical profile of the subsoil.

Unit	Depth (m)	USCS	Pass 0.08 mm (%)	N_{30}	q_u (kPa)	LL / PI	γ_d (kN/m^3)	w (%)	c´ (KPa)	ϕ´(º)	e_0	c_c
I. Silty clay	0.0 - 3.65/5.40	CL or ML	79-90	5-16	--	39.4/18.4	--	21.7	--	--	0.80	0.24
II. Sandysilt clay	3.65/5.40 – 15.00/13.10	CL or ML or SM	85-98	0-5	38-140	37.6/19.2	16.5	28.1	15	25	0.73	0.18
III. Sandy gravel	15.00/13.10 - End	GW	2.3	35-R*	--	--	--	--	0	41	--	--

*Table 2. Geotechnical properties of the subsoil. With, USCS: Soil classification according to USCS. Pass 0.08 mm: pass through 0.08 mm sieve. N_{30}: Result of Standard Penetration Test. q_u: uniaxial compressive strength. LL: Liquid limit. PI: Plasticity Index. γ_d: dry density. w = water content. c´: effective cohesion. ϕ´: effective internal angle of friction. e_0: initial void ratio. c_c: compression index. *N_{20}: Result of Dynamic Probing Super Heavy.*

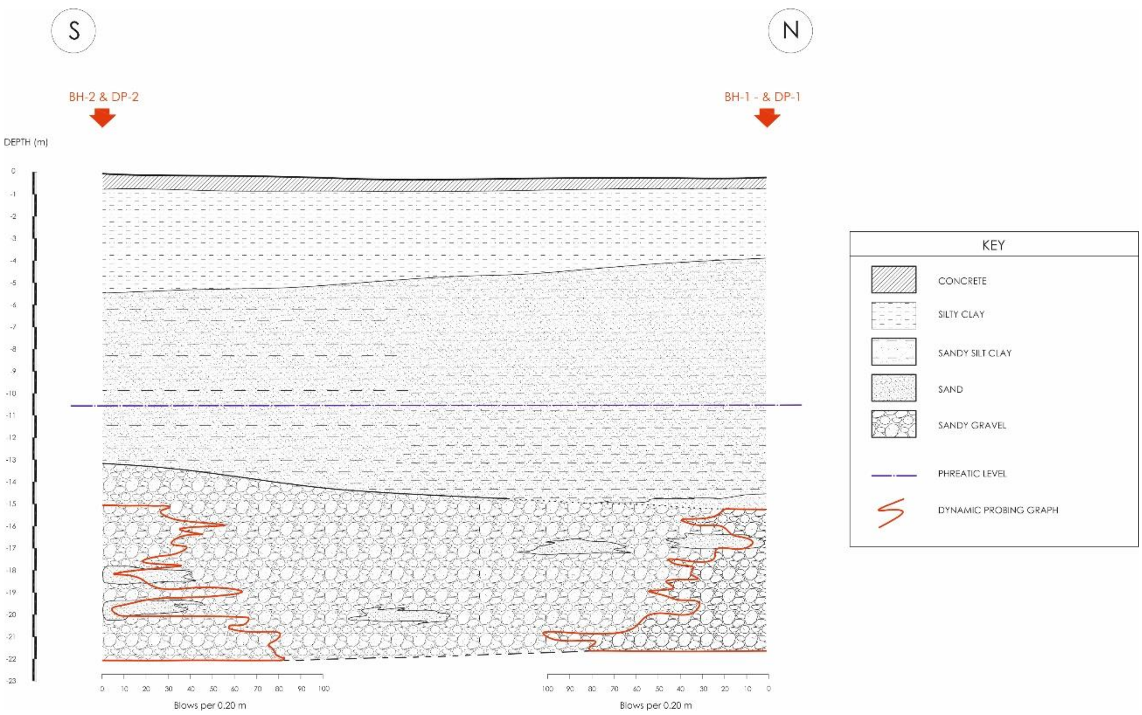

Figure 4. Geotechnical profile of the subsoil under the study case building. The silty clay level presents a slight southward dip. The location of boreholes is shown in Figure 1.

4.3. Works in the structure.

4.3.1. Inspection openings.

For an adequate evaluation of the dimensions, constructive procedures, damages and / or deficiencies of a selection of hidden elements of the structure, a series of inspection openings were made for the establishment of real dimensions and reinforcements. These data were then compared with those indicated in the original drawings and subsequently utilized in the calculation of the proposed reinforcement solution. In total, three inspection openings were made in the foundation, with locations shown in Figure 1, demonstrating that the dimensions and reinforcements of the project were fulfilled. An example of the procedure carried out is shown in Figure 5.

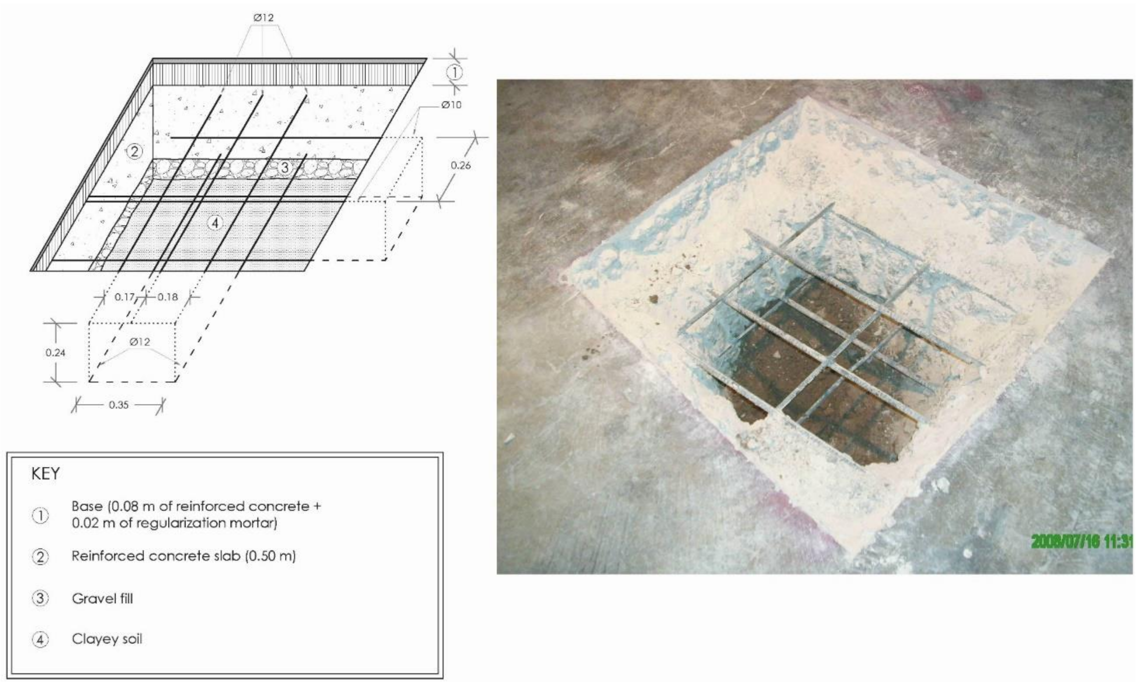

Figure 5. Inspection opening performed in the slab foundation (code CC-3). See location of inspection openings in Figure 1.

On the other hand, the cracks observed in the links of the joist of the first-floor slab with their respective main beams (Figure 2d) as well as some open cracks between the floor tiles (near and parallel to the main façade) indicate the existence of tension stresses as origin of its displacement. Four inspection openings were made in the first-floor slab (see location in Figure 1) and it was confirmed that the first-floor slab was built with prestressed joist. Regarding the joist-beam embedment in the points investigated, unequal embedment measurements were obtained (shown in Table 3). A detail of the inspection opening between pillars 89 and 90 is depicted in Figure 6.

Support of the precast semi resistant joist of the 1st floor slab				
Inspection opening	CF-1	CF-2	CF-3	CF-4
Embedment length (cm)	5.8	-0.3	-1.3	2.5

Table 3. Embedment length of the precast semi resistant joist and respective main beams, in the 1st floor slab. Positive values indicate embedment or contact with support on the main beam, while negative values indicate the minimum separation between the ends of the joist and its support on the beam.

Figure 6. Inspection opening on the first-floor concrete slab (code CF-3 in Figure 1). A detachment of 1.3 cm between the semi resistant joist and the main beam was registered.

4.3.2. Determination of displacements.

Displacements were measured to determine the magnitude of the movements experienced by the building. These data will be utilized to determine the type of movement (tilt and settlement, differential or in block) that the structure has experienced. To this end, the actions described next were carried out.

4.3.2.1. Topographic levelling of the basement floor.

Precision levelling of the column bases and perimeter walls was carried out to establish the topography of the upper side of the foundation slab. These data were compared with planimetric data provided by the drawings of the project, enabling determination of the topography of the basement floor and deduction of the local descents of the analysed points. A representation of the analysis is shown in Figure 7a.

4.3.2.2. Calculation of tilts.

Tilt measurements were analysed in various elements of the structure to determine the type of turning movement and possible structural repercussions. The overall tilt of the columns in the basement floor was checked, and the results are represented in Figure 7b.

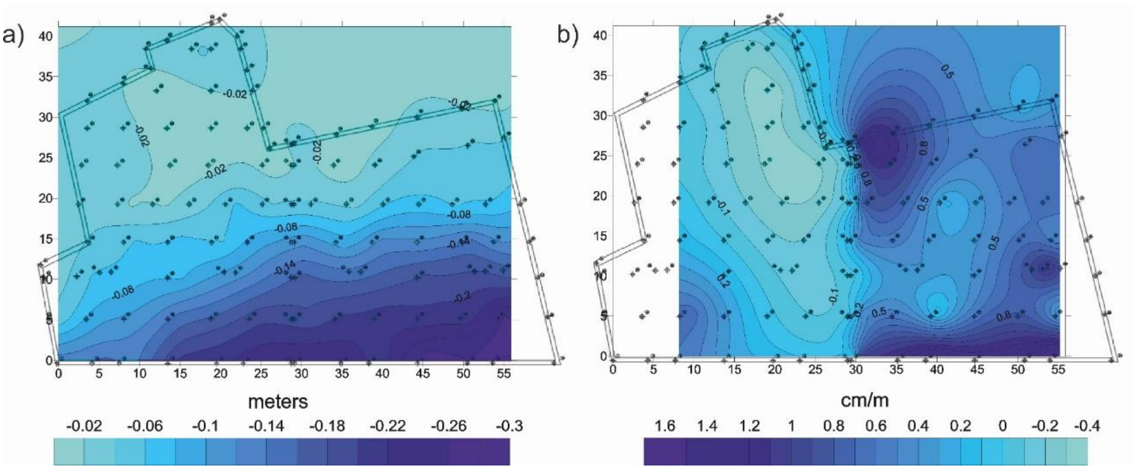

Figure 7. a) Topography of the upper side of the foundation slab. b) Tilt of columns in the basement floor.

Figures 7a and 7b show that significant deformations are concentrated on the South end of the building, with settlement values slightly above 260 mm.

4.3.2.2. Angular distortions.

The structural system of the building presents porticoes parallel to the main façade (even the foundation slab is ribbed in that direction with greater height). Angular distortions were determined as the difference between the vertical displacements corresponding to contiguous columns in the direction of the frame and the separating distance. Seventy-one columns (considered as simple porticoes) were evaluated and grouped into four categories (shown in Table 4). Figure 8 depicts the plant representation of the obtained values.

Group	Angular distortion	Nº of columns/ % of total	Risk of cracking partitions	Risk of structural damage
1	<1/500	32 / 45%	Low	Negligible
2	1/500-1/300	11 / 16%	Moderate	Low
3	1/300-1/150	18 / 25%	High	Moderate
4	>1/150	10 / 14%	Very high	High

Table 4. Results of the angular distortion study and classification according to the Burland et al. (1977) criterion.

The results presented in Table 4 are congruent with the crack map of the building and it is noteworthy to mention that areas with high absolute deformation (right lower quadrant of Figure 7a) present zones with small angular distortions, indicating that the turning movement of the building has been, mostly, as a rigid solid.

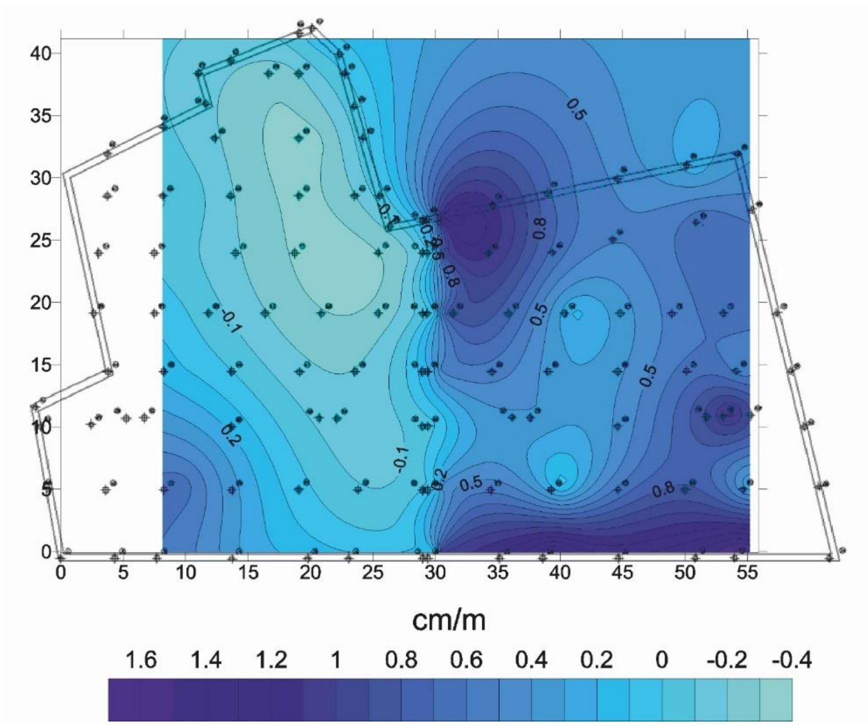

Figure 8. Angular distortions of the basement floor columns.

4.4. Cracks control

Six pins were placed at selected representative cracks to analyse their movement; pins were attached to each side of the crack and arranged in a triangular configuration. Tracking of the distance between points with known relative coordinates enables the determination of magnitude, direction, sense and temporal variation of movements. Figure 2c shows a detail of the type of control utilized. In addition, the specific locations of the crack control systems are shown in Figure 1. Table 5 presents control data prior to underpinning.

X Axis															
Reading date	Displacement (0.1mm)					Velocity (0.1mm/day)					Acceleration (0.1 mm/day^2)				
	T-1	T-2	T-3	T-4	T-5	T-1	T-2	T-3	T-4	T-5	T-1	T-2	T-3	T-4	T-5
7-aug-08	---	---	---	---	0.00	---	---	---	---	0.000	---	---	---	---	0.000
25-aug-08	---	---	---	---	0.10	---	---	---	---	0.006	---	---	---	---	0.003
18-sep-08	---	---	---	---	1.10	---	---	---	---	0.042	---	---	---	---	0.015
2-oct-08	---	---	---	---	3.40	---	---	---	---	0.160	---	---	---	---	0.084
16-oct-08	---	---	---	---	2.40	---	---	---	---	-0.070	---	---	---	---	-0.164
28-oct-08	---	---	---	---	2.30	---	---	---	---	-0.009	---	---	---	---	0.050
10-nov-08	---	---	---	---	7.90	---	---	---	---	0.432	---	---	---	---	0.339
21-nov-08	---	---	---	---	9.00	---	---	---	---	0.103	---	---	---	---	-0.299

Y Axis															
Reading date	Displacement (0.1mm)					Velocity (0.1mm/day)					Acceleration (0.1 mm/day^2)				
	T-1	T-2	T-3	T-4	T-5	T-1	T-2	T-3	T-4	T-5	T-1	T-2	T-3	T-4	T-5
7-aug-08	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
25-aug-08	-0.40	-0.40	-0.30	0.30	-1.00	-0.023	-0.020	-0.016	0.014	-0.053	-0.013	-0.011	-0.009	0.008	-0.030
18-sep-08	-0.80	-0.90	-0.70	0.70	-2.20	-0.014	-0.022	-0.016	0.017	-0.050	0.004	-0.001	0.000	0.001	0.001
2-oct-08	-0.80	-0.80	-0.90	1.10	-1.60	-0.005	0.008	-0.013	0.029	0.039	0.006	0.022	0.003	0.009	0.064
16-oct-08	-1.00	-0.50	-1.00	1.00	-0.90	-0.015	0.018	-0.009	-0.005	0.050	-0.007	0.007	0.003	-0.024	0.008
28-oct-08	-0.80	0.00	-0.90	1.30	0.00	0.019	0.046	0.006	0.029	0.079	0.029	0.023	0.013	0.029	0.024
10-nov-08	-1.10	-0.30	-1.10	2.40	-1.00	-0.020	-0.027	-0.018	0.079	-0.079	-0.030	-0.056	-0.018	0.039	-0.122
21-nov-08	-1.40	0.10	-1.60	2.80	-2.20	-0.028	0.036	-0.039	0.036	-0.109	-0.007	0.057	-0.020	-0.039	-0.028

Z Axis															
Reading date	Displacement (0.1mm)					Velocity (0.1mm/day)					Acceleration (0.1 mm/day^2)				
	T-1	T-2	T-3	T-4	T-5	T-1	T-2	T-3	T-4	T-5	T-1	T-2	T-3	T-4	T-5
7-aug-08	0.00	0.00	0.00	0.00	---	0.000	0.000	0.000	0.000	---	0.000	0.000	0.000	0.000	---
25-aug-08	0.00	0.20	0.10	-0.50	---	-0.002	0.013	0.007	-0.025	---	-0.001	0.007	0.004	-0.014	---
18-sep-08	0.20	-0.10	0.20	-0.80	---	0.011	-0.014	0.005	-0.013	---	0.005	-0.011	-0.001	0.005	---
2-oct-08	0.10	-0.10	0.20	-1.10	---	-0.006	-0.003	-0.007	-0.022	---	-0.012	0.008	-0.008	-0.007	---
16-oct-08	-0.10	0.10	0.00	-1.10	---	-0.019	0.015	-0.014	-0.005	---	-0.009	0.013	-0.005	0.012	---
28-oct-08	-0.20	-0.40	0.30	-0.90	---	-0.003	-0.040	0.025	0.017	---	0.014	-0.046	0.033	0.018	---
10-nov-08	-0.30	0.10	0.30	-1.80	---	-0.012	0.038	0.001	-0.063	---	-0.007	0.059	-0.019	-0.061	---
21-nov-08	-0.40	-0.20	-0.20	-2.10	---	-0.005	-0.027	-0.045	-0.028	---	0.007	-0.058	-0.041	0.031	---

Table 5. Results of cracks control. Vertical Z axis corresponds to upward positive direction. Only readings from controls 1 to 5 are shown, as control 6 was utilized exclusively in the monitoring phase.

Comparison of obtained data enabled determination of the correlation across the different quantified movements. Pearson's correlation coefficient is a statistical parameter that analyses two vectors and expresses their relationship by assigning a value between 1 and -1 (1 = direct relationship, 0 =

unrelated, -1 = inverse relationship). Herein all correlation coefficients were found between 0.91 and 0.99, which indicates a high degree of correlation between the movements of all cracks, suggesting that the cause was the same.

4.5. Concrete sampling

The extraction points for concrete sampling were selected considering statistical and representativeness criteria. At each selected location of the foundation slab, three cylindrical concrete core samples were extracted (Figure 1) for subsequent uniaxial compressive strength tests. These tests are necessary for the design and calculation of the proposed reinforcement, providing information about the quality and state of the existing concrete. All obtained results were higher than 25 MPa (minimum uniaxial compressive strength required in the original project).

5. Finite Element Method analysis.

A comprehensive structural and geotechnical analysis was developed, using the Finite Element Method (FEM), to determine the causes of the observed damage. Analysis considered the actual loads of the building and the previously described load asymmetry. A transversal section of the structure was modelled, perpendicular to the main façade, considering two different computation phases: a) an initial phase with the phreatic level located at -3 m below ground surface (ground water level when construction of the building was finished); and b) a final phase with the phreatic level located at -13 m below ground surface (maximum registered depth). Finally, the settlements associated with the consolidation process caused by water level decrease between both phases was calculated. Figure 9 shows the model adopted for the calculation of settlements.

The obtained results provided a maximum settlement of 230.6 mm, located at the South zone of the building. This value is similar to those obtained via levelling at the upper side of the foundation slab of the building (Figure 7a), confirming the relative rotation of the building on the same direction and magnitude as observed in situ. It can be concluded that the decrease of groundwater level registered across the valley was the main cause of the observed damage. Modelling of the first phase (i.e.,1983 groundwater level) did not explain the magnitude of the observed damage, although it revealed the beginning of the rotation of the building. Rotation was probably due to the concentration of stress on the area under the façade of the building, which increased the settlement of the area as the groundwater table decreased to its minimum value registered in 1995 (modelled in the second phase).

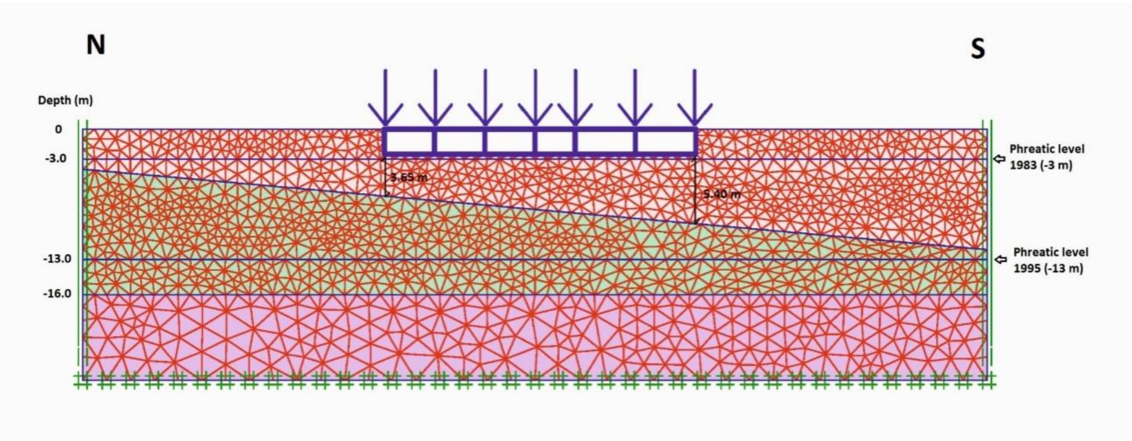

Figure 9. Finite Element Model of the studied building. The geotechnical properties of the different layers are described in detail in section 4.2.

The asymmetry of the loads on the slab foundation and the different thickness of the compressible soil layer (i.e., silty clays), combined with groundwater level decrease, have caused the building to rotate in the direction of the main façade (South) with similar tilts in the N and S façades (approximately 1.0%) and with horizontality losses on the building floors. As the structure is founded by a reinforce concrete slab, the effect of differential settlements on the structure has been considerably limited. The rotation axis of the building is almost parallel to the façades, being slightly rotated in the left-hand direction. This has caused the building to lean against the East neighbour building. Consequently, horizontal stress has been transmitted to the neighbour building, causing breakage of panels and tiles due to the induced compression. Damage has progressed due to the insufficient rigidity of the foundation against a rotation process of this magnitude.

6. Description of the reinforcement solution adopted.

The reinforcement system encompassed actions on the structure, with steel angle ("L" laminated profiles) reinforcements installed between the main beams and the semi resistant joists. Actions on the foundation were also carried out, underpinning the foundation with micropiles to end its movements. Before underpinning the foundation, steel angles were placed in the porticoes of the building (Figure 10) to reinforce the connections between the main beams and the semi resistant joists.

Figure 10. Detail of the steel angle reinforcements between the semi resistant joists and main beams.

Due to the characteristics of the ground within its first meters and the existing foundation, any type of superficial underpinning of the foundation was discarded. Deep underpinning with micropiles was executed, which transmitted the loads of the existing shallow foundation to the deeper gravel level (Level III).

The underpinning works finished in June, 2010. The micropiles were connected to the existing foundation by anchor plates. The adopted solution consisted of two to five micropiles per column, depending of the load. The micropiles had a drill diameter of 200 mm and were reinforced by a 88.9/73.9 mm steel pipe (outer/inner diameter). In the upper sections of the steel pipe, four corrugated steel bars (16 mm diameter) were welded to improve adhesion between the micropiles and the existing foundation (Figure 11). Finally, the embedded micropiles and the slab recess were completely filled with concrete repair mortar.

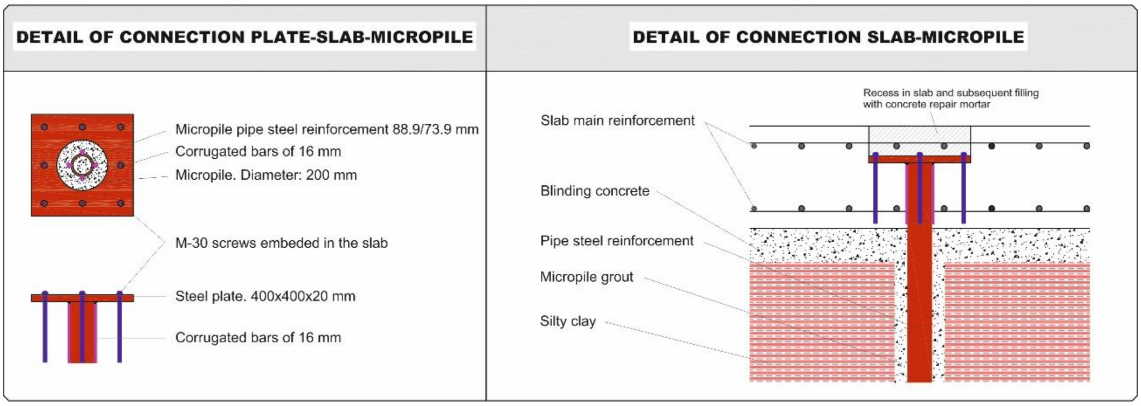

Figure 11. Detail of the connection between the existing foundation and the embedded micropiles.

One hundred and fifty-five micropiles were drilled, with variable lengths (16.5 or 21.0 m), according to the supported load.

7. Validation of the reinforcement solution through settlement monitoring.

- Once the underpinning of the building was finished, a structural monitoring plan was formulated to verify the suitability of the adopted remedial measures. The plan contemplated three measures:

- Monitoring of the relative settlement between the columns located within and outside the reinforced area.
- Monitoring of the crack control pins at different sectors of the building, to characterize the movements using a calliper (Figure 2c).
- Monitoring of the groundwater level under the building.

The levelling carried out between 2008 and 2013 indicated that the elevation increments of the base of the columns of the basement floor were null or extremely small. No defined displacement pattern was observed, with an average value of 2 mm (Figure 12). This order of magnitude is approximately the expected absolute error for the measurement instruments. Therefore, the measured displacements are not an indication of any instability of the building.

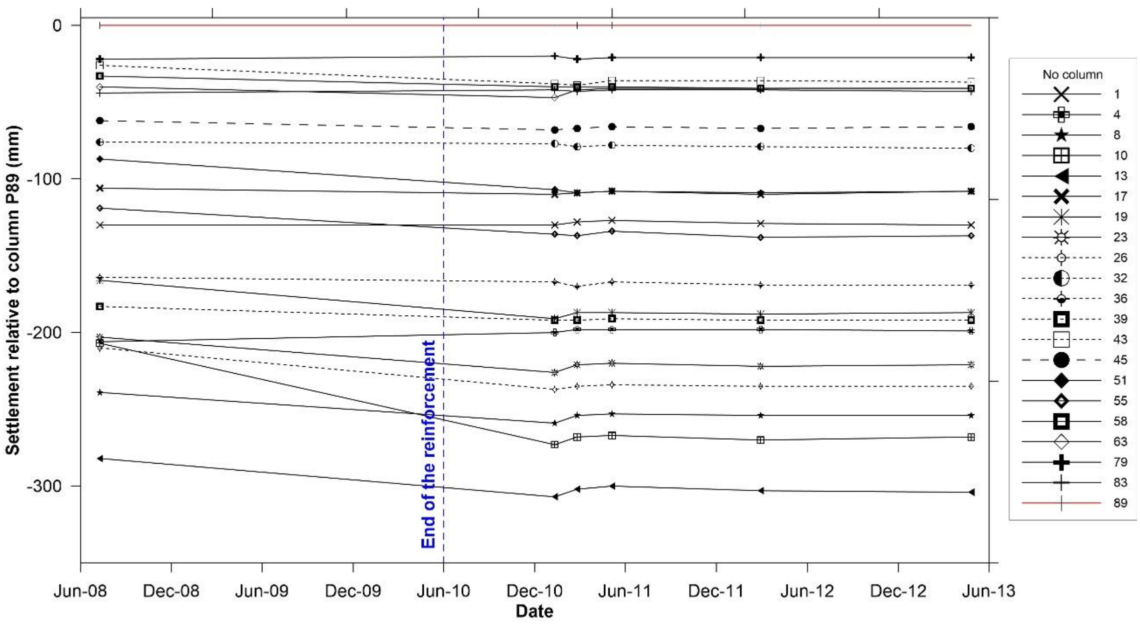

Figure 12. Evolution of the relative settlement of the base of the columns. The first data corresponds to July, 2008, when the building was settling. Note that measurements are relative to column 89. The location of columns is shown in Figure 1.

The crack monitors indicated the existence of oscillatory displacements that closed the fissures in summer and opened them in winter (Figure 13). These displacements, associated with seasonal temperature variations, present a main horizontal component and millimetric amplitude.

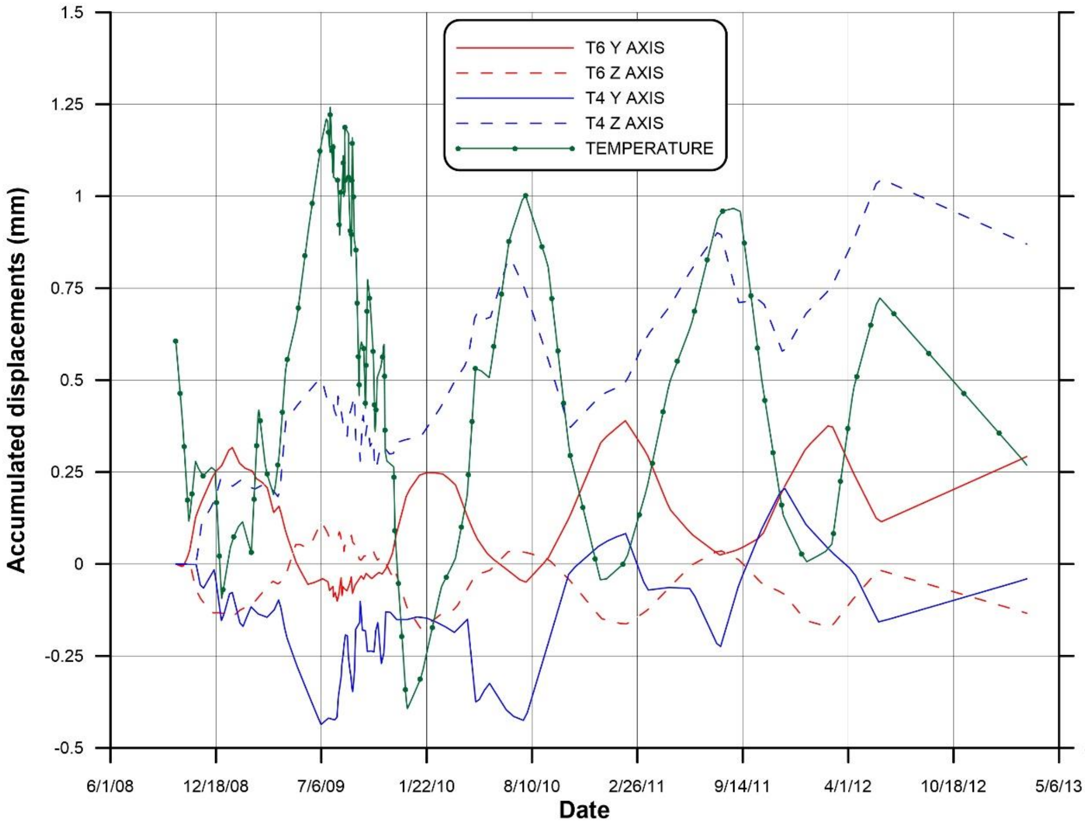

Figure 13. Accumulated displacements at crack monitor nº 4 (T4) and 6 (T6). Displacements along the X-axis were zero. The temperature time-series has been also represented on the plot.

Some of the crack monitors exhibited small movements with slight increasing activity, as a result of their function as joints and due to elastic hysteresis phenomena and fatigue of the materials. However, these movements did not correspond to foundation movements that could cause structural damage. These aspects are shown in Figure 13 where there is a clear relationship between temperature (MeteoMurcia (2017)) and the displacement time series.

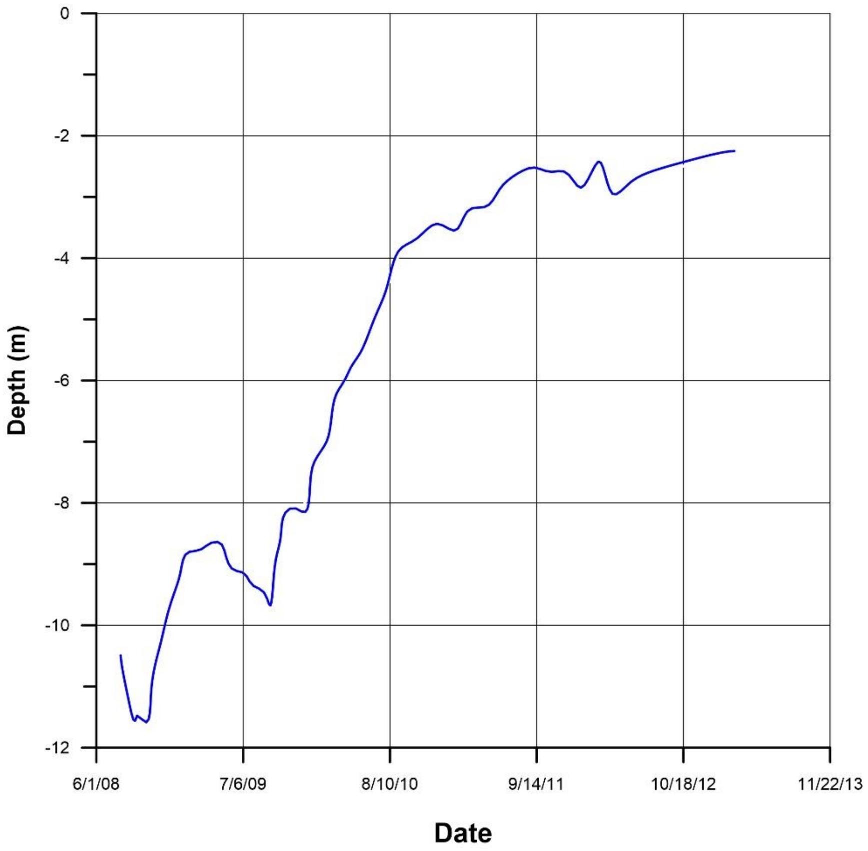

Figure 14. Time series of the groundwater level in borehole 1 (location shown in Figure 1).

Regarding the control of groundwater levels, analysis of the time series indicated the existence of an asymptotic trend at -2.0 m depth, in accordance with pluviometry. Higher-than-average precipitation caused fast groundwater level rise, over 8 m in 4.5 years (Figure 14). In September 2008 and 2009, two minimum precipitation records were reported (Figure 14), coinciding with the respective annual dry seasons. Seasonal variations are not accidental and must be considered at the time of designing the underpinning of the foundation, as occurred herein.

Table 5 summarizes the techniques utilized herein as well as the respective application objectives.

Investigation technique		Objectives				
		Resistant and deformational soil characterization	Groundwater level monitoring	Verification of the current state of the building	Monitoring of the building displacements	Assessment of the situation and proposals for action
Foundation study	In situ geotechnical tests	✓				✓
	Laboratory geotechnical tests	✓				✓
	Piezometer installation		✓		✓	✓
	Crack maps			✓	✓	✓
	Slab foundation levelling			✓	✓	✓
	Tilt measurements			✓	✓	✓
	Angular distortion measurement			✓	✓	✓
	Foundation inspection openings			✓		✓
Soil-structure interaction study	FEM modelling			✓	✓	✓
Structural study	Compilation of existing information			✓		✓
	Visual inspection			✓		✓
	Structure inspection openings			✓		✓
	2D crack monitors				✓	✓
	Crack maps			✓	✓	✓
	Concrete tests			✓		✓
	Study of the current state of the structure			✓		✓

Table 6. Data sources and investigation techniques utilized herein for forensic analysis.

8. Multitechnique approach for the study and reinforcement of buildings affected by land subsidence.

Considering the experience acquired in this case study, a general methodology is proposed for the study and reinforcement of buildings located on subsiding areas due to groundwater extraction. The methodology consists of five steps and is illustrated in Figure 15. The first step (1st stage) consists of the compilation of existing information and first inspections. These data will lead to the investigation step (2nd stage), during which the subsoil, the foundation and the structure will be characterized through the installation of devices and in situ monitoring systems (to obtain the magnitude and direction of the displacements affecting the building). These displacements will be monitored throughout time during the third step (3rd stage) to determine trends and progression of the displacements, which will be analysed during the fourth stage (4rd stage). Analysis of all the available information will enable a decision on the convenience of reinforcing the foundation and the structure (5th stage: study of alternatives and proposals of actions). If the movements remain active after implementation of actions, monitoring will continue until the displacements stop, guaranteeing the safety of the structure and verifying the appropriateness of the proposed reinforcement.

Figure 15. Flowchart of the proposed methodology.

9. Conclusions

Buildings located on subsiding areas are vulnerable to damage that can reduce habitability and, in some cases, jeopardize safety conditions. It is therefore necessary to formulate a general multitechnique methodology to address these specific buildings, enabling decision-making on the underpinning and/or reinforcement of these damaged structures.

The case study presented herein demonstrated the applicability of the proposed methodology, considering a building located in Murcia (SE Spain). The urban area of Murcia was affected by land subsidence induced by high groundwater variations caused by a long drought period and the overexploitation of the aquifer, being the first case study reported in Spain. The settlements caused by subsidence damaged more than 150 buildings (Justo and Vázquez 2002; Mulas et al. 2003; Rodríguez Ortiz and Mulas 2002), among which is the case study building.

The damage observed was mainly due to subsidence, and was in agreement with the 10 m groundwater level drop recorded in the area between 1992 and 1995. The tilting of the building was also confirmed by the FEM model of the consolidation process. Tilting was due to the unequal thickness of the most compressible layer (Level I, silty clay), which was thicker under the main façade, and also due to the asymmetry of the loads (higher in the façade zone). This situation caused slight tilting of the structure towards the South, which was further increased by land subsidence.

Any heterogeneity affecting the building (e.g., asymmetry of loads, geometric asymmetry, different types and levels of foundation, etc.) can further intensify the effects of subsidence, causing serious damage to the building. Furthermore, soil heterogeneities due to changes on the thickness or properties of deformable layers under the foundation can also intensify damage.

Therefore, these aspects must be carefully studied and considered during the design and project phases of constructions.

The maximum recorded tilt for the structure was approximately 1.0%. This value is higher than the limit value established by the Spanish Technical Code (1/500 ≅ 0.2%) (CTE 2006). The tilt of the basement columns reached maximum values of 1.67%, also exceeding the limit determined by Spanish standards (CTE 2006), 1/250 (≈ 0.4%).

Regarding angular distortion, the values calculated on the date of underpinning indicated that most of the building did not present high risk of structural damage. The movement of the building followed rigid body rotation.

The observed damage and the recorded evolution during the monitoring period resulted in a dangerous situation, forcing the adoption of actions to stop the differential settlements of the foundation. The building foundation was underpinned with micropiles, restoring its stability and bearing capacity.

The cracks caused by insufficient embedment of semi resistant joists into the main beams of the first slab floor required the reinforcement of the link to avoid potential partial collapses. Steel angles were coupled to the main beam with bolts to support the semi resistant joists. Once the underpinning works were finished, aesthetics and function of the building were restored.

Finally, it must be noted that, in general, the underpinning solutions to be considered in buildings affected by land subsidence must ensure the transmission of loads into deep resistant layers to avoid the consolidation of more superficial layers due to groundwater level changes. However, the downward movement of soils due to consolidation processes can cause negative skin friction, inducing a drag load on the pile that must be considered in calculations.

The multitechnique approach proposed herein along with the reinforcement of damaged buildings has provided successful results, as demonstrated by the post-reinforcement monitoring plan.

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