Underpinning of Buildings on the Santiago Gravel

Réprise en Sous-oeuvre en Gravier de Santiago

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SYNOPSIS During the last decade increasing needs for construction of multilevel basements on the fluvial gravel of Santiago, mainly destinated to parking space, have arised. Due to the complete use of the site plants, the underpinning of adyacent existing buildings has been the main factor for developing design and construction methods, particularly when the are old or historical or monumental structures. This work gathers the chilean experience on discontinuous earth-retaining and underpinning structures using reinforced concrete piers, unsupported or supported with steel, wood or reinforced concrete struts. Besides, the main design criteria are included for both static and seismic conditions and the theoretical values are compared with field measurements of the stresses acting in the struts. Based on the actual construction costs a brief economical analysis of this underpinning-supporting approach is presented.

INTRODUCTION

Excavations with vertical cuts going down to depths of 10 - 12m, necessary for the construction of multilevel basement of buildings located on the fluvial gravel of Santiago, have obligated to perform underpinning works of existing structures. The characteristics of the underpinned structures correspond in some cases to reinforced concrete buildings structurated with shear wall or seismic resistant frames. In other cases, they correspond to old adobe buildings or 2 - 3 story ancient masonry buildings, poorly structurated and frecuently founded on weak strip footings made of unreinforced concrete or boulders stuck with mortar.

In many cases, the high prices of land in commercial areas (US$ 500 to $ 1.200 per each square meter) compared to the cost of each linear meter of underpinned perimeter (US$ 650 to 750 per linear meter) has turned attractive the development of underground basements using 100% of the available site plant. In Table I an example confirming the above statement is presented

TABLE I

Simplified Comparison of Costs

40 x 50 m piece of land in commercial area	US$	850	per m^2
3 level subterranean parking with total area of		6.000	m^2
Average cost of underpinning 10 m deep	US$	700	per m
Average cost of excavation	US$	8	per m^3
Average cost of subterranean construction	US$	250	per m^2
Average cost of aerial construction	US$	180	per m^2

Case 1. Offices building with 3 level subterranean parking		Case 2. Offices building with aerial parking in a 3 level separated plant	
Land	US$ 1.700.000.-	Land for offices bldg. US$	1.700.000.-
Underpinning works	126.000.-	Land for parking bldg.	1.700.000.-
Excavation	160.000.-	Parking Construction	1.080.000.-
Parking Construction	1.500.000.-		
TOTAL	US$ 3.486.000.-	TOTAL US$	4.480.000.-

* In case 1, Foundations for offices and parking bldg. are common.

SOIL CHARACTERISTICS

The fluvial gravel of the city of Santiago (3.500.000 inhabitants) is a thick deposit of compact sandy gravel, with about 15 - 20% of boulders sizing up to 0.3 m in diameter. Almost everywhere the gravel is overlaid by a 1.5 - 3.0 m thick deposit of CL type clay of medium to high consistency. From its surface down to a depth of 5 - 7 m the gravel presents a typical amount of low plasticity silty fines, with a significative cohesion estimated in c = 14.7 kPa and an angle of internal friction of $\phi = 45^{o}$. In deeper deposits the presence of more plastic clayish fines surrounding the coarse particles is rather typical and a stronger cohesion of c = 34.3 kPa must be assigned to the gravel, keeping the same friction angle. The ground water table is normally deeper than 30 m. The gravel total unit weight in its natural state is 22 - 22,6 kN/m^3. The determination of these parameters as well as those related to deformability of the gravel, for static and cyclic stresses are exposed in detail by Ortigosa et al (1981).

CONSTRUCTION METHODS

Given the soil characteristics the use of piles or diaphragm walls has proved to be unprofitable. In practice the best results have came out from a system based on reinforced concrete piers, whose behaviour has been quite satisfactory in all the cases (more than 15) as for its constructive feasibility as well as for its compara tively low costs. These piers, whose typical sections are shown in Fig. 1, are constructed in alternated sequence within hand excavated vertical pits without bracings, allowing between them an unsupported front of soil that takes advantage of the cohesion for holding by arching effect a stable vertical cut (Schneebeli, 1974). When a poor existing foundation has required reinforcement a continuous beam of reinforced concrete has been underpinned as illustrated in Fig. 2 . The use of struts to support the piers has turned out to be necessary in order to reduce lateral deformations of the underpinned front as well as to withstand the seismic inertia forces. In some cases where the structure to be supported is less important or little sensitive to soil displacements, timber struts (eucaliptus ϕ 0.25 m) have been used. Wood struts give the advantage of its low cost and easy instalation. For important or sensitive structures ϕ 25 m metalic pipes and reinforced concrete struts have been also used, although concrete has came out with some constructive difficulties that caused its elimination in further projects as a feasible solution, except in supporting corner piers. In this last case its construction and latter performance have proved to be satisfactory,

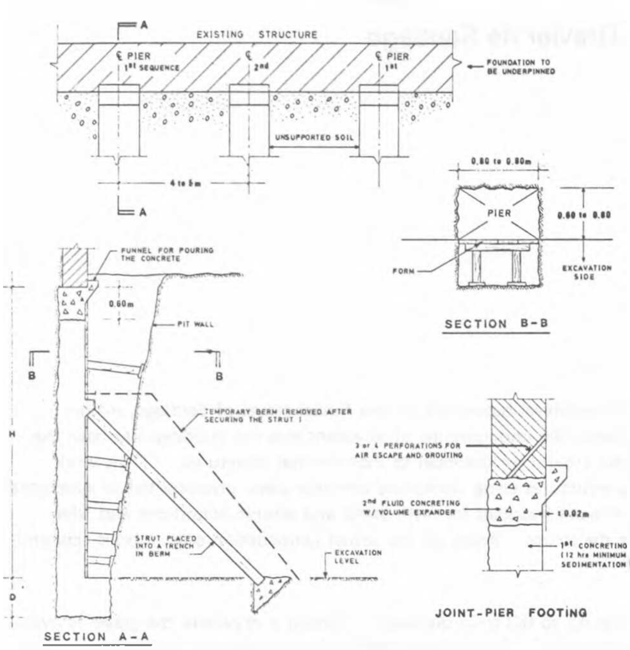

Fig. 1. Construction Details of Underpinning Pier.

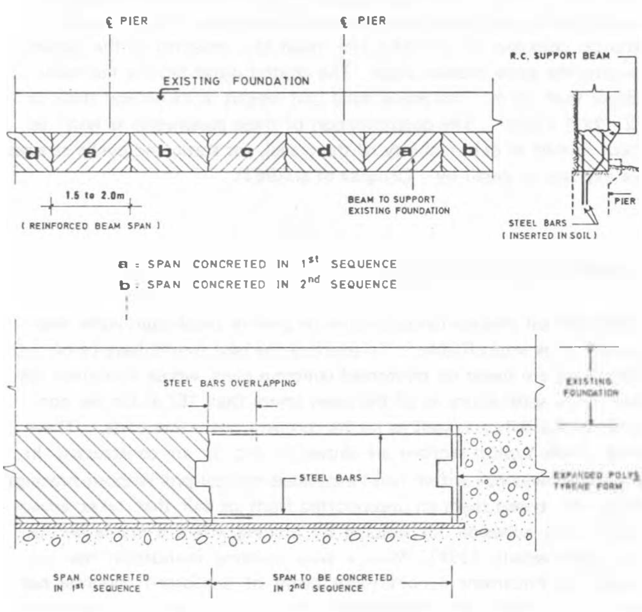

Fig. 2. Construction Details of Support Beam.

since they are arranged horizontally between two piers. Connections between piers and struts are illustrated in Fig. 3 . In same cases where the structure to be underpinned is laterally selfsupported (direction perpendicular to the excavated front) and when the resulting piers are shorter than 6 m, traditionally no struts have been contemplated, even when considering seismic loads. A system having 4.5 m high unsupported piers underpinning a 4 story reinforced concrete building behaved satisfactorily during a 0.18 g maximum acceleration quake registered on July 8, 1971. (University of Chile, 1972).

The concreting of piers head (top) directly underneath the existing footing have been made using a simple slow pressure grouting instead of the traditional mortar packing, with hignly satisfactory results Fig. 1.

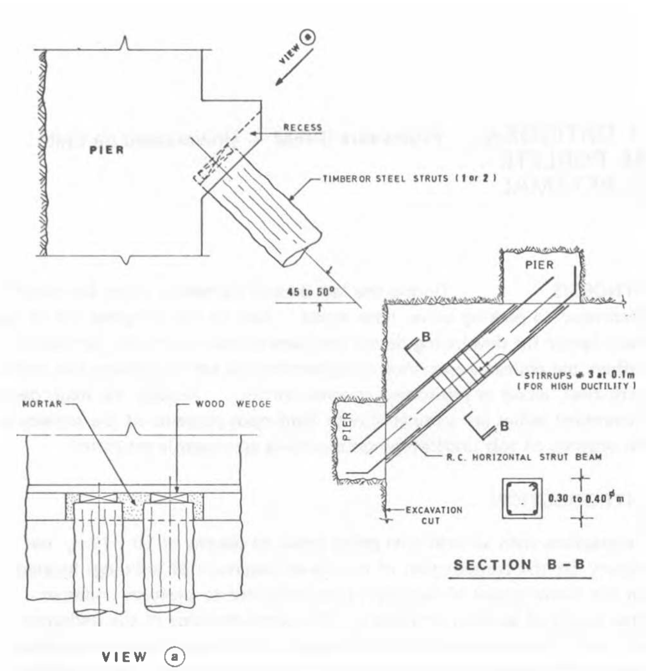

Fig. 3. Details of Strut - Pier Conections and R.C. Corner Struts.

DESIGN PROCEDURE

Piers and struts have been dimensioned by means of the design model and stresses shown in Fig. 4 . Static thrusts have been evaluated in active limiting conditions on account of the expected lateral stress relaxation of the soil during the construction process. This assumption is based on the results of triaxial compression tests on undisturbed samples of the gravel, which gave very small failure strains (ϵ_f = 0.5-1.0%) at confinning pressures of the same order to those that were present within the soil mass next to the excavation front (Ortigosa et al, 1981). The seismic thrusts have been considered to be somewhat equivalent to a horizontal inertia force F_s equals to 15% of the weight of a rigid free body wedge defined by its height H an a plane sloping 70 degrees with respect to the horizontal. This criteria is based on actual landslide observations after quakes affecting almost vertical cuts in the gravel, which define planes sloping 70 - 75 degree with respect to the horizontal (University of Chile, 1972).

Seismic force T_s due to inertia forces from the underpinned structure are considered only when its foundations fall entirely within the assumed 70 degree wedge or when the over wedge part of the structure is not adequately tied to the other part beyond the wedge. When the structure to be underpinned is adequately backtied in direction normal to the excavation front and when the height H is less than 6 m a simplified procedure has been developed as indicated in Fig. 4b . In the figure, θ_s represents the maximum soil shear distortion by seismic effects, K_z the cyclic subgrade reaction of soil behind the pier and $K\theta$ the cyclic angular stiffness at the pier bottom.

The design value for θ_s considering an accelerogram record in San -

tiago during the July 8, 1971 earthquake, scaled to 0.30 g, results of the order of 2.5×10^{-4} radians (Medina, 1976). Pier foundation depth D and force T_p on struts are obtained by the equilibrium of force and moments. As noted, friction at the pier bottom ($R = V\,tg\phi$) is not included in the analysis, thus giving a conservative estimation of

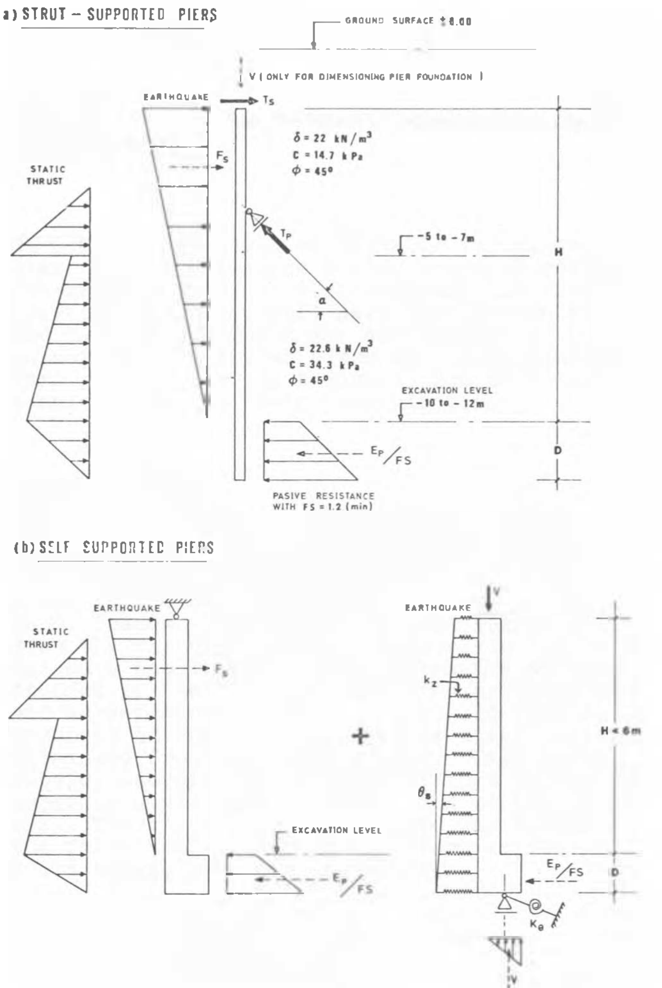

Fig. 4. Design Loads for Underpinning Piers.

D. Referring to T_p, to the resulting values obtained from equilibrium another component must be added, which takes rise from the seismic differential lateral displacement between the ends of struts. Finite elements analysis of this effect has yielded values of maximum differential horizontal displacements of the order of 0.002 m. Considering a lenght of 9 m and an inclination $\alpha = 45^{\circ}$ for an ordinary strut, a unit axial strain of 1.5×10^{-2}% is obtained. For wood struts with axial stress-strain mudulus of $E = 18 \times 10^{6}$ kPa, this effect may increase the axial stresses in about 3000 kPa.

For steel struts with $E = 206 \times 10^{6}$ kPa such increment may go up to 30.000 kPa.

Field measurements of the axial force T_p are presented in Table II (one strut per pier) for static steady state loads in three buildings. A small earthquake ocurred when the "Edificio Fundación" measurements were in progress, but they did not give mensurable variations of such forces. All determinations were conducted with vibrating wire extensometers, taking special care for readings at equal enviromental temperature, with struts under shadow and temperature conditions similar to the moment when the initial zero readings were taken. These zero readings were made inmediately before the withdrawal of berms and continued periodically until a stabilization of a maximum T_p value was obtained, approximately after a period of 20 - 30 days. Results of Table II indicate that the present design procedure and the soil parameters used provide very acceptable estimations of T_p, al least for steady state static loads.

TABLE II

Results of Struts Axial Loads Measurements

Case	α	H(m)	T_p measured (kN)*	T_p calculated (kN)	Struts material
Caracol de los Pajaros	50°	11.50	118 ± 20	127 ± 34	wood
Hospital Español	45°	10.0	113	137	steel
Edificio Fundación	45°	10.0	255 ± 34	226	steel

* It corresponds to an average of three different strut measurements.

CONCLUSION

A structurally discontinuous underpinning system has been presented which takes advantage of the good soil mechanics conditions of the Santiago gravel. This system has been in all cases quite feasible to construct with traditional means and its cost reduced to the point of turning it atractive for developping 3 to 4 levels basements occupping 100% of very valuable land. This system may also be used for simple earth retaining of vertical cuts eventually complemented with a shallow diaphragm wall for holding loose superficial soils. The design procedures which have been used appear to be essentially correct in the identification of work mechanisms as well as in the estimation of main loads.

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