Miguel Hidalgo Vehicular Tunnel

Tunnel de Véhicules Miguel Hidalgo

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SYNOPSIS. The paper describes, from the point of view of soil-structure interaction, the construction of a vehicular tunnel 750-m-long built in the heart of Guadalajara, capital city of the State of Jalisco, in Mexico. The predominant soils at the site consist of pumice sand deposits. The main problem to deal with was the alignement of the tunnel axis very close to important historical buildings. The design called for a rectangular tunnel with walls formed by adjacent piles cast in place, which was found to be a successful solution, since practically nearby structures did not show any appreciable movement during the construction operations. Theoretical analyses were performed in the course of construction in order to compare the results with the movement recorded in situ. Continuous monitoring of the instrumentation installed to follow the behaviour of the surrounding buildings during construction, was considered a key factor for control purposes.

INTRODUCTION

The State government of Jalisco has built in the capital city of Guadalajara, a vehicular tunnel which forms part of the new traffic system crossing the downtown area, where buildings of significant historical importance are found along the route. This was the reason why a street-level solution was found to be not feasible.

The alignement of the tunnel axis along the stretch to be discussed, falls very close to three public buildings, the most important of them being the Metropolitan Cathedral whose facade is topped by two tall campaniles representing the landmark of the city (Fig. 1). This fact implied the necessity of an adequate tunnel design to avoid any possible damage to the building.

Fig. 1 Metropolitan Cathedral

The main problem faced by the designers was the passage of the tunnel near one side of the Cathedral North tower (Fig. 2) at a distance of 4.4 m and at a depth 2.5 m below foundation level. This tower is 66-m-tall and weighs about 6,300 ton; it is founded upon a masonry slab footing constructed at a depth of 4.3 m, which transmits a contact pressure to the ground of 490 kN / m^2.

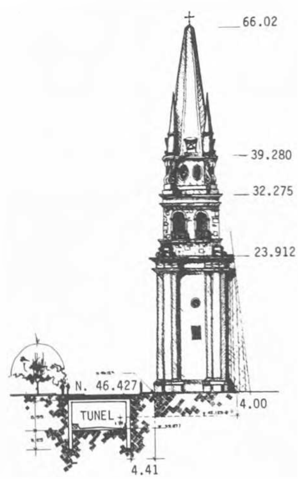

Fig. 2 Location of the Tunnel

Figure 2 shows the location of the tunnel with respect to the Cathedral. The inside dimensions of the tunnel are 10 m in width by 4.5 m in height. Its total length is 770 m of which 170 m correspond to the access ramps.

PRELIMINARY SOIL MECHANICS STUDIES

In accordance with the project design, the subsurface investigation covered 10 exploratory sites along the tunnel stretch under study (Soiltec, 1977), comprising standard penetration tests and undisturbed sampling with Shelby tubes. Seven of the borings reached depths ranging from 13 to 32 m.

The other three were open test pits excavated to a depth of 4 m with undisturbed block samples trimmed by hand from the walls at the different strata encountered.

Index properties and strength tests were performed on the undisturbed samples.

STRATIGRAPHY AND SOIL PROPERTIES

Along the tunnel axis the subsoil is composed of pumice and vitreous sand deposits of pyroclastic origin. This material shows a resilient behaviour and a low unit weight; it is easily altered by weathering and has a high absorption and friction.

From the subsurface exploration results a representative soil profile shown in Fig. 3 was obtained. The subsoil is formed mainly by clayey sand deposits (SC) with fine pumice gravel; the sand has a relative density varying from the loose to medium (6 to 28 blows in the standard penetration test) in the upper 5 m, and from medium to compact (15 to 50 blows) at greater depths; in some instances, clayey lenses were found interbedded in the sand deposits.

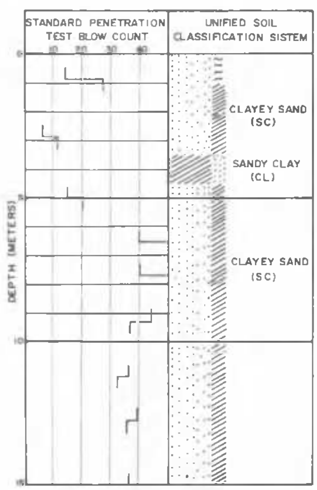

Fig. 3 Soil Profile at Boring No. 5

Index tests showed the following results:

Natural water content, W	40 - 60%
Void ratio, e	0.91 - 1.87
Specific gravity, S_s	2.31 - 2.40
Bulk unit weight, γ	900 - 1,500 kg/m^3

Strength parameters were found to be as follows:

	Angle of friction ϕ	Cohesion kN / m^2
Unconfined compression	——	58 - 100
Quick triaxial test	20°- 36°	29 - 140
Standard penetration *	27°- 40°	——

* According to Bowles (1968)

The ground water table was found at some places at different depths, being 9 m the average value. The upper boundary of the sound rock underlying the site varied from 10 m in depth in the low areas to 30 m in the higher zones.

THEORETICAL ANALYSIS

The design of the tunnel combined field test results with the intended use of the project in such a way that the the resulting construction method would not disturb the soil surrounding historical buildings that could be damaged otherwise.

After analyzing several options, it was decided to construct a tunnel with a rectangular cross-section, the walls being formed by adjacent piles cast in place, in order to achieve a rigid frame together with the roof and floor slabs acting as a structural unit.

To design the structural elements of the tunnel box, the following factors were taken into account:

a) Depth of wall embedding to avoid a bottom failure of the excavation
b) Bearing capacity of piles
c) Earth pressure against the tunnel walls

Heaving of the excavation bottom due to the loads transmitted by the neighboring buildings was arrested by fixing a depth of embankment of 3.6 m, having attained a reasonable safety factor.

The bearing capacity analysis of the piles was performed for two cases: 1) considering an embedment distance equal to the full length of the piles (9.15 m); and 2) with the pile acting as the tunnel wall with a depth of embedment of 3.60 m. For the round piles 0.80 m in diameter finally selected, the bearing capacity for the first case using two different criteria was found to be as follows:

	Q_p	Q_s	Q_t
Terzaghi & Peck (1967): Local failure	45	56	101
General failure	122	56	178
Transportation Research Board (1975) (Vesić's criterion)	135	70	205

where:

Q_t = total bearing capacity, ton
Q_p = load carried by point bearing, ton
Q_s = load carried by friction along the pile shaft, ton

The earth pressure acting against the walls was calculated considering the at rest condition of the soil, using the Rankine theory, as well as the surcharge induced by the adjacent buildings. To compute the earth pressure against the most critical span of the wall due to the surcharge produced by the North tower of the Cathedral, pressure distribution was determined with Boussinesq's theory. The resulting maximum thrust against the wall was equal to 59 ton per linear meter, of which 20 ton were due to the earth pressure and 39 ton to the surcharge load.

As a result of the previous analyses, a structural frame was selected for the tunnel box, having a rigidity at its upper part (formed by the roof slab and walls) high enough to resist the earth pressure without deforming during the excavation process, until the floor slab could be completed. However, in order to increase safety against failure, it was decided to place horizontal struts at mid-height of the excavation at critical areas (Fig. 4) and leave them until the structural section would set as a rigid frame.

TUNNEL DESIGN AND CONSTRUCTION PROCEDURE
The final design involved a rectangular tunnel built with reinforced concrete (Fig. 5). The walls would be formed by piles 0.80 m in diameter and 9.15-m-long; the roof slab was 0.50-m-thick and the floor slab had an average thickness of 0.45 m.

The construction procedure was set as follows: construction of cast-in-place adjacent piles without disturbing the ground to form the tunnel walls; pour the roof slab on top of the piles and of the ground surface; start the underground excavation with tunnelling methods; and finally pour the floor slab and appurtenant structures (Fig. 6).

INSTRUMENTATION FOR MONITORING BEHAVIOUR OF SURROUNDING BUILDINGS
The following instruments were installed:
1. Seventy six surface reference points for control of settlement, of which 31 were placed at the Cathedral facade and the rest along other buildings.
2. To monitor development of cracks, 486 sheet metal reference gages were placed, 346 of which correspond to the Cathedral.
3. To measure deviations from a vertical line, 34 check points were installed at the Cathedral and 15 in the rest of the buildings.

ADDITIONAL TESTS DURING CONSTRUCTION
In order to compare theoretical results with actual behaviour, the following operations were performed:
1. Pile load tests
2. Standard penetration tests and soil sampling
3. Measuring of strut pressures

Load tests were done on selected fully embedded piles before pouring the roof slab, in order to avoid future interference with construction operations. The piles were 0.80 m in diameter and 9.15 m-long. The loading frame is shown in Fig. 7. The central pile was tested under compression while the lateral ones were subjected to extraction in order to have data related to lateral skin friction.

The loads were applied in increments up to a maximum of 120 ton. Unloading by stages followed until the applied load was released. The following results were obtained:

COMPRESSION

Pile No.	Maximum load ton	Total deflection mm	Permanent deformation mm
1	75	3.6	2.3
2	90	5.1	3.8
3	100	4.6	2.8
4	120	6.3	3.9

EXTRACTION

Pile No.	Maximum load ton	Total deflection mm	Permanent deformation
1	37.5	0.8	0
2	45	0.6	0.1
3	50	2.3	0.9
4	60	4.2	2.0

Standard penetration tests and soil sampling were made near those piles tested under load, with the aim of determining soil properties, which turned out to be similar to those obtained during the preliminary subsoil exploitation.

Pressure measurements were taken at the horizontal struts placed at mid-height of the tunnel excavation, to define the necessary load needed to release the anchoring devices and dismantle the struts once the floor slab was completed. The struts were originally positioned under a load of 10 ton. Excess pressure would be that developed by the ground while lowering to excavation to its final depth. Pressures needed to remove the strut anchors varied from 14 to 22 ton at the struts where measurements were taken; these values indicate that the excavation process induced an appreciable increase in the strut loads.

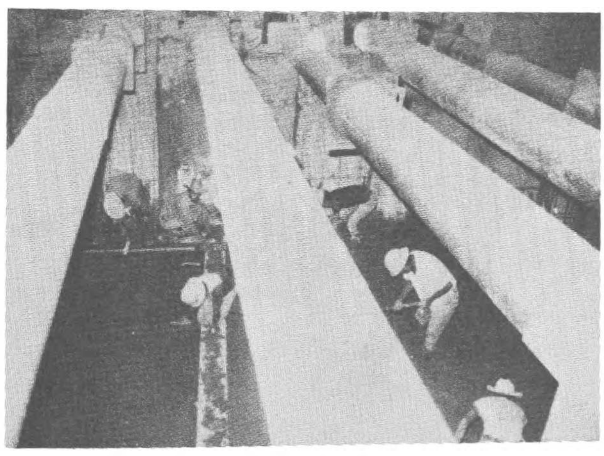

Fig. 4 Steel Struts

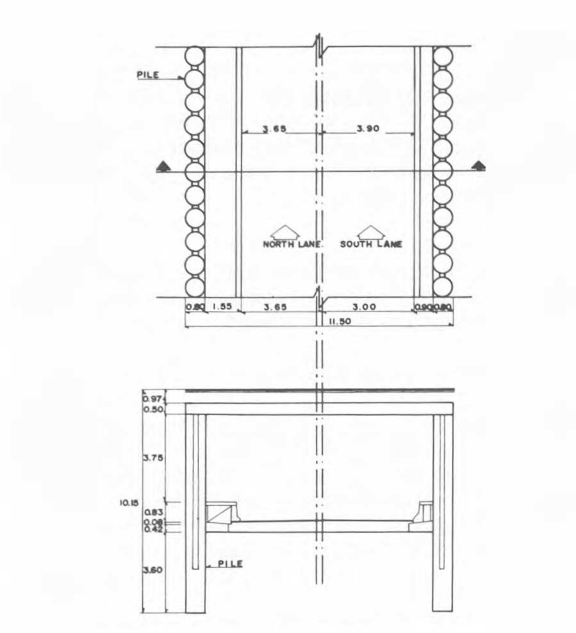

Fig. 5 Plan and cross section

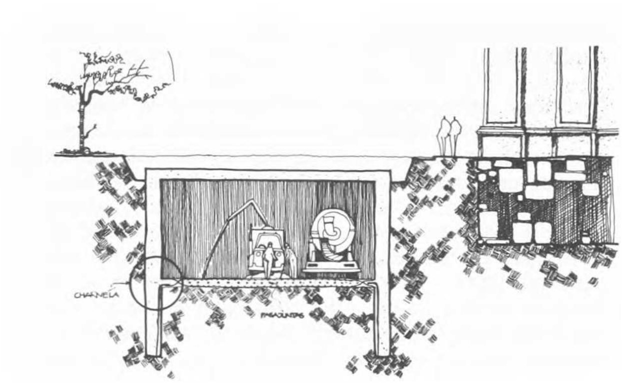

Fig. 6 Construction Process of the Floor Slab

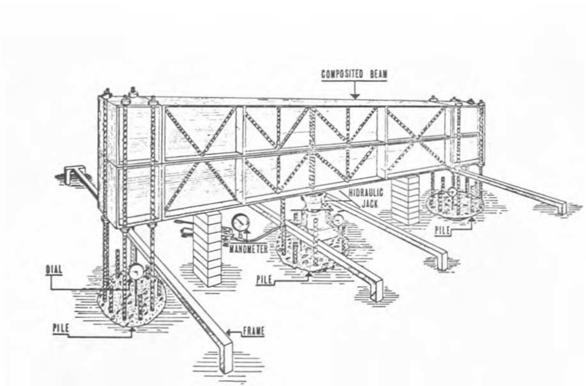

Fig. 7 Pile Load Testing Set-Up

BEHAVIOUR OF HISTORICAL BUILDINGS DURING CONSTRUCTION

Instrumentation Installed to monitor the behaviour of the historical sites provided the following information: At the Metropolitan Cathedral, settlements in the order of 1 mm were recorded at three leveling points, and of about 2 mm at 10 other points which were the closest to the tunnel axis; no movements were registered at the rest of the points. In what refers to crack gages and pendulum devices, no variation was detected.

Fig. 8 shows a plot of settlement versus time corresponding to a reference point placed adjacent to the construction site. Of the two 1 mm-steps that can be observed, the first one occurred when excavation of the stretch just in front of the North tower reached mid-height; the second coincided with the removal of the struts until the floor slab started to work as a structural unit.

A similar trend was observed at the other buildings being monitored, where settlements were very small and no tilting or crack development were detected.

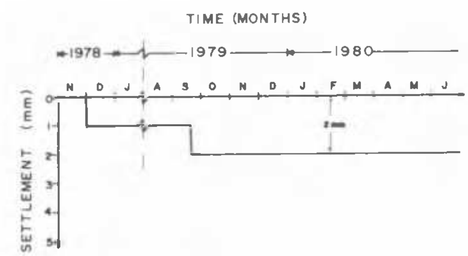

Fig. 8 Time - Settlement Curve

COMPARISON OF THEORETICAL RESULTS AND ACTUAL BEHAVIOUR

1. Regarding bearing capacity, although failure was never reached during the load test, a permanent settlement of 5 mm was derived by extrapolating the results. This value would correspond to an ultimate load applied of 170 ton. From the bearing capacity analyses an ultimate load varying from 101 to 205 ton was obtained, which is in good agreement with the real value.
2. Field test results showed a lateral friction of 60 ton that corresponds to the stage where permanent deformations are achieved during extraction of the piles. In the theoretical analyses, values ranging from 56 to 70 ton were obtained which coincide reasonable well with the actual results.
3. In spite of the safety factors adopted for the structural design of the tunnel, the rigid frame experienced deformations due to the lateral thrust acting on the walls.

CONCLUSIONS

Six months after completion of the tunnel (Fig. 9), the following conclusions could be drawn with respect to the observed behaviour of the historical buildings and to the measurements recorded during construction:

Fig. 9 Metropolitan Cathedral and Tunnel

1. The structural design of the tunnel box and the construction procedure were successful from the point of view that there were practically no movements observed at the surrounding buildings during the construction stage.
2. The use of cast-in-place piles was an adequate solution for constructing the walls with practically no deformations experienced on the adjacent soil.
3. The continuous recording of the instruments installed to monitor the behaviour of the historical buildings allowed the contractor to work on the tunnel with absolute confidence since any malfunction could be immediately detected, if one would have arisen, and the necessary remedial action be taken opportunely.
4. The results obtained from theoretical analyses were calibrated during the course of work with measurements taken in situ during construction; in both instances the agreement was very good.

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