Computed Friction in a Pile's Rock-Socket

Frottement à l'Emboiture en Roche d'un Pieu

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SYNOPSIS A special compression load test was performed for the exclusive evaluation of skin friction along the rock-socket of a pile whose tip rested on a flatjack.

The pile's socket was instrumented with both tell-tales and strain-gages. Whereas the tell-tale data was sufficient for result evaluations and interpretations, the meager strain-gage information made mandatory the use of a theoretical assumption for compression attenuation along the socket.

Nevertheless, the comparison between the theoretically computed skin friction and the displacement data was coherent and showed the process of friction build-up at the socket.

Shear stresses as high as 1700 kPa would have existed at the top of the socket without causing any apparent yielding of the concrete-rock bond.

INTRODUCTION

Project piles have been designed for both friction and tip resistances, with variable rock-socket lengths based upon assumed stress values for different degrees of rock quality.

A pile load test was designed to evaluate exclusively the friction resistance in the rock environment, aiming at a reduction of the adopted rock-socket lengths in the project.

Piles were of the bored type, made of reinforced concrete, with diameter of 1.1m or 1.3m; total lengths ranged from 25m to more than 50m; rock-sockets varied from 1.5m to more than 6m.

The underlain rock is, in general, a biotite-gneiss whose degree of weathering and fracturing change, within a few meters, from very high to almost none; this basement rock is capped by its residual soil, over which lies a somewhat thin, old colluvial deposit with granite boulders; this soil, in turn, is covered by an unconsolidated, young, marine sedimentary section with clays, silts and fine sands, about 15m thick.

The test pile was cased from elevation -2.80m up to nearly the socket top, at elevation -30.0m. The casing, a 0.038m thick steel cylinder, had been asphalt-coated both on the inside and outside where it was fully in contact with a bentonite slurry filling an oversized previous boring (see Fig. 1).

Axial compression was provided at the head of the pile by the action of two 6000 kN hydraulic jacks reacting against a metallic frame anchored in the basement rock by twelve 1200 kN capacity tiebacks (see Fig. 2).

To evaluate exclusively the friction resistance in its 1.1m diameter, 3.3m long rock-socket, the pile tip was made to rest on a water filled, thin walled metallic pad, previously placed at the bottom of the bored rock. Two flexible high-pressure hoses connected to the pad and

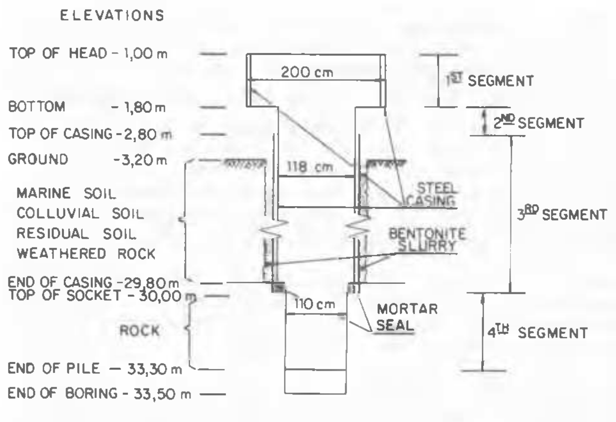

Fig. 1 Test pile general profile

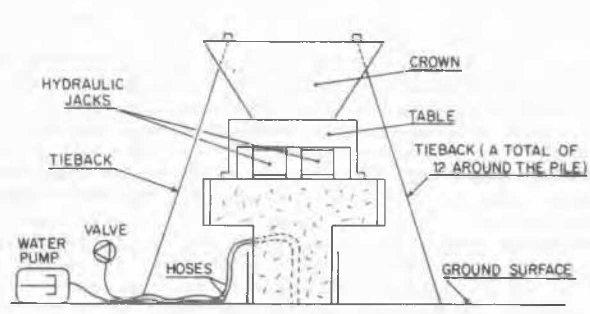

Fig. 2 Loading device

extending up to the ground surface, some 34m above, maintained the water under pressure during concreting and released that pressure during the load test, nullifyng therefore any tip resistance (see Fig. 2 and Fig. 3).

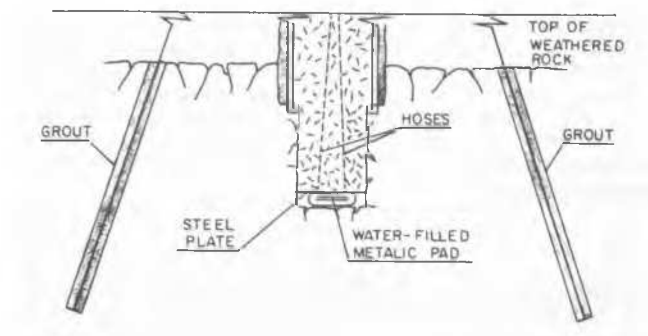

Fig. 3 Anchoring and bottom installation

Pile instrumentation was both external and internal. At the pile head, 4 dial gages and 4 target rods monitored total settlements;another 4 dial gages monitored the steel casing's vertical movements; a pair of dial gages indicated horizontal deslocations of the pile head. Also at the head, 5 dial gages monitored the movements of the tell-tales, whose tips were regularly spaced along the socket length. The reference beams were controlled by 4 target rods. Within the pile, 2 levels of strain measurement were located in the upper part of the socket utilizing strain-gages installed in the reb: .s There were 5 strain gage pairs (one active, one dummy) at each level; the upper level was at the socket top and the next 0.50m below it (see Fig. 4).

<u>Test Results and Comments</u>

Fig. 5 shows the pile head vertical displacements, those of the steel casing top 1.8m below, and those of the tell-tales (above 6440 kN as explained later by Fig. 7). Tell-tale nº 4 results are not plotted due to its malfunctioning; in unloading, tell-tale nº 5 results were not coherent and so not plotted.

The pile's total settlement was less than 10^{-2}m; remnant displacement was less than 10^{-3}m; this 90% recovery indicates an almost elastic response during the load test.

Steel casing displacements followed those of the pile's head up to aproximately 4960 kN; in unloading they yielded a higher remnant value, probably due to the casing's insertion in the mortar seal at the top of the socket (see Fig.1).

Fig. 6 shows the axial strains at the pile's cross-section 0.50m below the rock-socket elevation. These values correspond to the original strain-gage readings divided by 1.3, due to the half-bridge circuitry. Of the 5 strain-gages installed only 3 could be used.The upper installation, corresponding to the socket top did not function coherently. Strain-gage readings have been corrected for drift and for elastic hysteresis before plotting. The 4960-6440 kN load step corresponds to a sharp increase in strain-gage response.

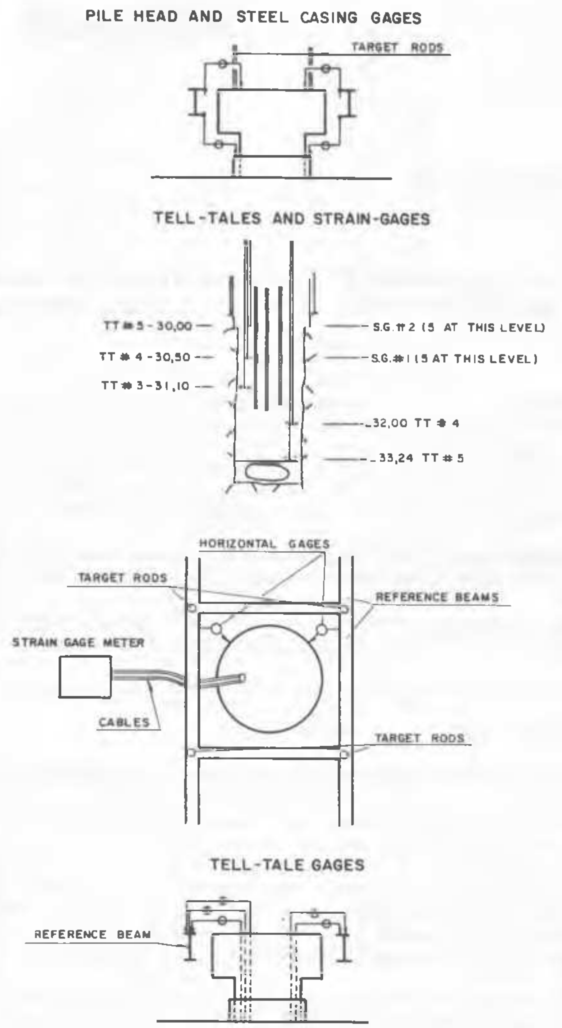

Fig. 4 Pile Instrumentation - Details

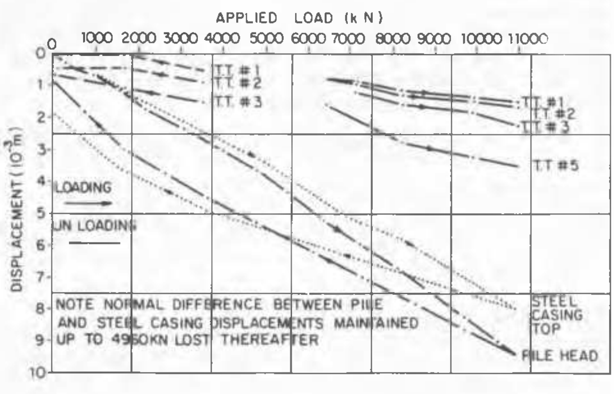

Fig. 5 Vertical Displacement(From Dial Gages)

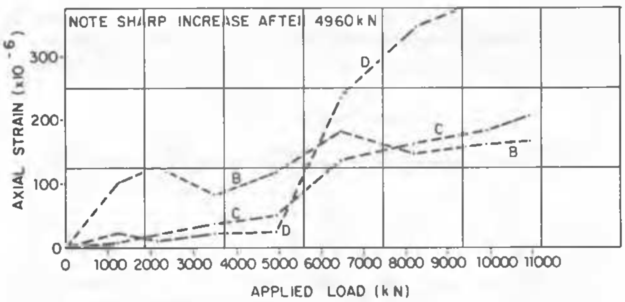

Fig. 6 Axial Strains (From Strain-Gages).

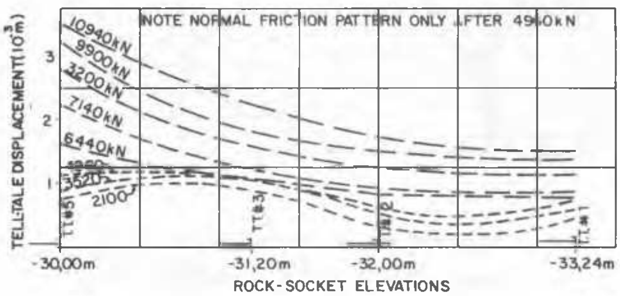

Fig. 7 Socket Displacements (From Tell-Tales).

Fig. 7 shows the tell-tales displacements in the rock-socket zone for each applied load. There is an anomalous behaviour of their displacement field, which only ceases after 4960 kN; therefore, the displacement plot of any use for interpretation purposes starts for 6440 kN (as in Fig. 5).

Data Interpretation

To obtain the amount of load that reached the socket top, a necessary first step is the computation of the elastic moduli of the test pile (E), which for this purpose may be divided into four segments (see Fig. 1):

1st: at the enlarged head (diameter of 2.00m) and length of 0.80m, where its reinforced concrete is cased and unconfined (E_1).

$$E_1 = 3.47 \times 10^7 \text{ kPa}$$

2nd: at the portion below (diameter of 1.18m) and length of 1.00m, where the reinforced concrete is uncased and unconfined (E_2).

$$E_2 = 2.60 \times 10^7 \text{ kPa}$$

3rd: at the whole length embedded in the soils and weathered rock (diameter of 1.2054m) with a length of 27.20m, where it is cased and confined (E_3).

$$E_3 = 4.05 \times 10^7 \text{ kPa}$$

4th: along the rock-socket (diameter of 1.10m) with a length of 3.27m, where it is uncased and confined (E_4).

$$E_4 = 3.18 \times 10^7 \text{ kPa}$$

Displacement at the top of the 3rd segment could then be theoretically computed, by propagating down the pile (across the 1st and 2nd segments) the test data at the pile head. Displacements at the base of the 3rd segment correspond to the tell-tale nº 5 data. A check for the theoretical migration of displacements from the pile's head to the level of the cased segment, is provided by the test data from the steel casing at the same elevation (see Table I).

TABLE I

Applied Load at the Pile Head (kN)	DISPLACEMENTS		
	Measured at the top of the steel-casing (10^{-3}m)	Computed for concrete core at same elevation (10^{-3}m)	Difference core-casing (10^{-3}m)
250	0.29	0.28	-0.01
500	0.45	0.45	0
750	0.58	0.55	-0.03
1000	0.69	0.67	-0.02
1250	0.82	0.81	-0.01
2100	1.51	1.56	0.05
3520	2.42	2.46	0.04
4960	3.42	3.56	0.14
6440	4.67	4.98	0.31
7140	5.26	5.61	0.35
8200	5.78	6.48	0.70
9900	7.16	7.99	0.83
10940	8.01	8.98	0.97

Differences are significant above 4960 kN where they indicate a slip of the reinforced concrete core relative to the steel casing. This check, plus the strain-gage and tell-tale data, point to a true socket mobilization under friction only above 4960 kN of applied load.

Once the displacements at the top and at the base of the 3rd pile segment are known it is possible to compute the amount of load that reached the rock-socket top (see Table II).

TABLE II

Applied Load at the Pile Head (kN)	At the top of the socket		
	Axial Strain (ε_o) (10^{-6})	Compressive Stress (σ_o) (kPa)	Incident Load (Q) (kN)
6440	158	5048	4770
7140	160	5077	4820
8200	174	5524	5250
9900	222	7056	6700
10940	256	8144	7740

Within the socket both compression and shear develop; the tell-tale movements are in part caused by compressive displacements and in part by shear drags. To differentiate between these two effects it is assumed that compressive stress will diminish along the socket according to Boussinesq's formulation, applying it at each tell-tale elevation:

$$\sigma_z = \frac{1.5Q}{\pi Z^2} \text{(with } Z_1\text{=1.10m; } Z_2\text{=2.00m; } Z_3\text{=3.324m)} \qquad (1)$$

From these values, axial strains have been computed as

$$\varepsilon_z = \frac{\sigma_z}{E_4} \qquad (2)$$

The differential displacements, between adjacent tell-tales, due exclusively to compression (Δu_c), have been evaluated according to the formula

$$\Delta u_c = \varepsilon_{av} \cdot \Delta Z \qquad (3)$$

where ε_{av} is the average strain between adjacent tell-tales; ΔZ is distance between them (ΔZ values were 1.10m; 0.99m and 1.324m).

Comparing these Δu_c values to the total displacement difference between tell-tales from the test data (Δu), it is possible to infer the differential displacement due solely to shear drag (Δu_s) simply by

$$\Delta u_s = \Delta u - \Delta u_c \qquad (4)$$

These are presented in the following table, together with the computed average skin frictions (τ_{av}) per tell-tale interval, given by

$$\tau_{av} = \frac{\Delta \sigma_z}{\Delta Z} \cdot \frac{R}{2} \qquad (5)$$

(R = 0.55m, the radius of the socket).

TABLE III

		Incident Load on Top of Rock-Socket (kN)				
		4770	4820	5250	6700	7740
T.T. # 5	σ_z	5018	5077	5524	7058	8144
	ε_z	158	160	174	222	256
	Δu	0.51	0.99	1.20	1.44	1.26
	Δu_c	0.12	0.12	0.13	0.17	0.19
	Δu_s	0.39	0.87	1.07	1.27	1.07
	τ_{av}	784	794	863	1103	1272
T.T. # 3	σ_z	1883	1903	2073	2645	3056
	ε_z	59	59	65	83	96
	Δu	0.29	0.30	0.33	0.33	0.56
	Δu_c	0.04	0.04	0.04	0.05	0.06
	Δu	0.25	0.26	0.29	0.28	0.50
	τ_{av}	402	405	442	564	651
T.T. # 2	σ_z	569	576	627	800	924
	ε_z	18	18	19	25	29
	Δu	0.04	0.04	0.11	0.11	0.19
	Δu_c	0.02	0.02	0.02	0.02	0.03
	Δu_s	0.02	0.02	0.09	0.09	0.16
	τ_{av}	81	82	84	119	131
T.T. # 1	σ_z	206	208	227	290	335
	ε_z	6	7	7	9	11

Stress in kPa, Displacements in 10^{-3} m

Fig. 8a is a graphical presentation of the skin-friction profiles along the socket, as interval values and as a continuous distribution (drawn through mid-points of the intervals) for each applied load at the rock-socket top.

Fig. 8b shows the correlation between the skin-friction values and the cumulative shear drags, both referred to the tell-tales' elevations. It is a plot of shear displacement functions at selected points along the concrete-rock bond. It can be observed that the tendency of the three curves is to flatten (maximum shear strain) at much higher shear values than those reached by the test. No slip, not even partial or localized, appears to have occurred.

Therefore it can be concluded that the socket's mobilization during the load test was within the elastic regimen of the concrete-rock bond.

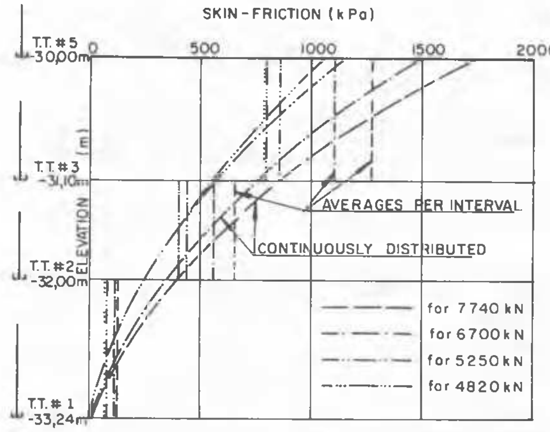

Fig. 8.a.

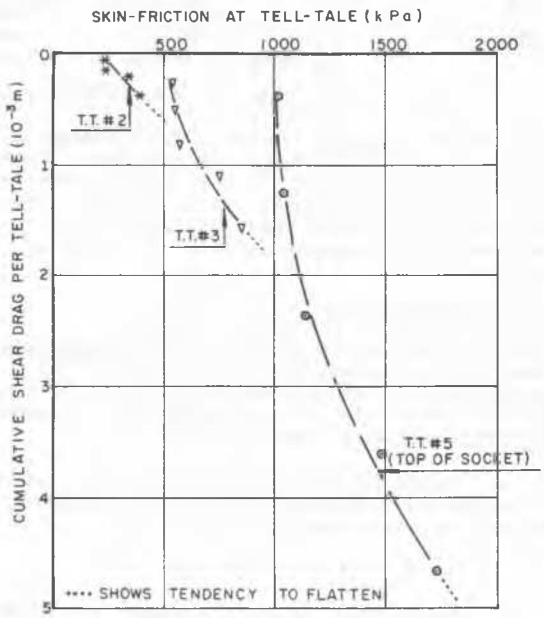

Fig. 8.b.

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