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SEISMIC PERFORMANCE OF CONCRETE OIL PLATFORMS IN SHALLOW WATERS. THE LAKE MARACAIBO CASE.

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ABSTRACT

Lake Maracaibo basin is one of the most important oil fields in Venezuela. Most of the oil that comes from this area depends on production platforms whose structural configuration has many original features. The evolution of this configuration over the last seventy years was governed mainly by vertical load capacity, ease of construction and stiffness under moderate waves. These platforms are built in an open lake in water depths ranging from 10-35 m. Their structural system is composed of prestressed concrete piles arranged either vertically (one per pile head) or inclined (two or three per pile head). Based on recent reassessment of the seismicity of the Lake Maracaibo area a concern about the seismic reliability of the platforms arose. The object of the present paper is to present the results of the first stage of a project having as main objectives the identification of the critical components of the structural system and the study of their behavior under seismic forces well in excess of their yielding capacity. This objective was achieved by performing non-linear push-over static analysis of the basic structural subsystem that provides most of the lateral strength and stiffness, taking special attention in modeling the local response of the prestressed concrete piles, non-linear soil structure interaction in the piles (axially and laterally) and integrity of the connection between piles and pile heads. The results obtained show that the inclined piles provides most of the lateral strength and stiffness, and that the post-yielding behavior is governed by the axial stiffness and strength of the surrounding soil. The early axial yielding of the soil delays the exceedance of the flexural-axial strength of the piles, hence limiting the local ductility demand in them. This means that the platforms have a greater global ductility than what can be inferred from the local ductilities of the piles alone.

KEYWORDS

Concrete platforms, prestressed piles, non-linear analysis, moment-curvature relations, lateral loads, Lake Maracaibo.

INTRODUCTION

Lake Maracaibo basin is one of the most important oil fields in Venezuela. Most of the oil that comes from this area depends on production platforms whose structural configuration has many original features, among them the use of extremely long concrete piles and the very soft soils found at the lake bed. The evolution of this configuration along seventy years was governed mainly by three conditions: vertical load capacity, ease of construction and stiffness under moderate waves. Based on recent reassessment of the seismic hazard (Cascante and Gajardo, 1992) in Lake Maracaibo, a concern about the seismic reliability of these structures have arisen. As a consequence, new industry standards for their seismic design were published (PDVSA, 1994). Typically this kind of structures (similar in configuration to some docks and harbors) are considered non-ductile (PDVSA, 1994). Due to their unique nature, a research project is currently being carried at INTEVEP in order to study the behavior of the platforms and their components under seismic loads. In this paper the results of the first stage are presented, namely the study of the monotonic response under extreme lateral loads of the main component that provides lateral strength and stiffness to the complete structure.

DESCRIPTION OF THE STRUCTURAL SYSTEM OF THE PLATFORMS IN USE IN LAKE MARACAIBO

The structural system of the concrete platforms in Lake Maracaibo consists of an arrangement of reinforced or prestressed concrete piles with all their pile caps connected with a grid of horizontal bracing beams (steel or concrete). They are built in water depths ranging from 10 to 35 m. The equipment and production facilities are placed on a concrete slab resting on the pile caps and beams or are attached to a prefabricated (onshore) steel module which is connected to the pile caps. There are basically three different arrangements of piles with a common pile cap: i)an isolated vertical pile; ii)a couple of battered (inclined) piles symmetrical with respect to a vertical axis through the pile cap, the slope of the piles ranging from 1:5 to 1:8 (type I); and iii) a couple of piles, one vertical and one battered (type II). The bracing beams and the concrete slab or the floor grid of the steel module placed on top of the platform, all contribute to create a stiff horizontal diaphragm. In Fig. 1 the plan of a typical platform is shown and in Fig. 2 a detail of the type I couple.

The vertical loads are resisted by all the piles (with the exception of the battered pile in the type II couple). Under lateral loads, like the inertial loads induced by an earthquake, the couples of battered piles are the most important components of the lateral load resisting system. For an effective depth of 15 m, a type I couple is about 15 times more stiff than one vertical pile, for a depth of 30m, a type I couple would be about 300 times stiffer that one vertical pile (Mibelli and Díaz, 1994). Under the assumptions that the horizontal diaphragm is stiff enough in its own plane and that the lateral stiffness and strength of the vertical piles and of the couples out of their planes can be neglected, the response of a complete platform can be approximately predicted from the response of the couples in their own plane.

In what follows, this behavior is studied by means of non-linear push over static analysis of isolated couples. In this way the collapse mechanism and the available ductility of this couples can be assessed. The analysis were made under a static and monotonically increased lateral load. The pile-soil interaction, the local response of the pile and the effect of large displacements were all taken into consideration. Only a summary of the results for type I piles will be shown, since they are the most commonly used in the deeper waters of Lake Maracaibo (about 30 m). For type II couples refer to (Parra, et al., 1996).

SOIL-PILE INTERACTION

The data available for the geotechnical characterization of the soils in Lake Maracaibo are mainly pile driving records, some SPT of the lake bed, static load tests of piles (Rajani and Soto, 1986), and a thorough characterization of the soils in the east coast of the lake (Echezuría et al., 1990). Typically the top layer is a low plasticity soft clay (CL), normally consolidated (soft mud). Underneath it there are layers of stiff clay with low to high plasticity (CL-CH), highly overconsolidated, with some lenses of dense silts and sands (firm ground). In order to define typical thickness for these layers a statistical analysis of the pile driving records (4218 in total) was performed. The results are shown in Table 1.

Table 1. Typical soil layerings assumed

Identification	Soft mud		Firm ground		Axial capacity (shaft + tip)
	depth (m)	Su/σ'v	depth (m)	Su/σ'v	(t)
A	3,0	0,2	16,0	1,2	300
B	9,5	0,2	9,5	1,4	300
C	16,0	0,2	3,0	1,9	300
D	-	-	19,0	0,69	200
E	-	-	19,0	2,2	500

To assess the undrained shear strength (Su), the SHANSEP methodology was used (Ladd and Foot, 1974) together with a back analysis of some of the load tests of axially loaded vertical piles. The Su values were adjusted in order to obtain 200, 300 and 500 t (1t≈10 kN). The normalized shear strength (Su/σ'v) are associated with overconsolidation ratios (OCR) from 2 to 6. For the normally consolidated clays (soft mud), the Su values were taken from triaxial tests (Echezuría et al., 1990). The effect of the non-plastic materials were neglected since they are not always present.

The pile-soil interaction was modelled with the use of non-linear springs that simulate the axial load transfer along the shaft (t-z curves) and pile end (Q-z), together with the lateral resistance of the soils (p-y). The t-z, Q-z, p-y curves were taken from API-RP2A (1993), using a residual strength for the friction resistance along the shaft equal to 70% of the maximum, and the soil properties given in Table 1.

LOCAL RESPONSE OF THE PRESTRESSED CONCRETE PILES

For the type I couple, prestressed concrete piles are used. These piles have 36" (914 mm) external diameter, 26" (660 mm) internal diameter and 24 to 32 1/2"ϕ strands, depending on the length of the pile. In order to incorporate their local response for all load levels, the flexural and axial stiffness, ultimate flexural capacity and the available local ductility in the piles should be modeled. These variables can be studied with the aid of the moment-curvature-axial load relations (M-Φ-P).

Moment-curvature-axial load relations

The monotonic M-Φ-P relations were calculated under the following assumptions: i)plane sections remain plane after bending; ii)there is perfect bonding between the prestressing steel and the surrounding concrete; iii)for the concrete the stress-strain relation proposed by Mander et al. (1988) was used (Fig. 3), with a compressive cylinder strength of 450 kg/cm2 (45 MPa), without considering any effect of confinement since there are few stirrups not closely spaced; iv) for the prestressing steel the model given in Blakely and Park (1973) was used, adjusting its parameters to the average of the results of several tension tests of the strands used in the piles (ϕ 1/2", ASTM A-416) (Fig. 4).

The strands are tensioned to 70% of their ultimate nominal capacity and the concrete is assumed to have a strength of 250 kg/cm2 (25 MPa) at the time of transfer of stress. Before the flexural deformation is applied the losses due to the elastic deformation and creep of concrete ($\varepsilon t_{\infty}/\varepsilon t_0$ = 2,1), and steel relaxation are calculated, using the strain-stress diagrams for a very slowly applied strain. In this way the stress state in the pile after a long time is approximately assessed. Since the earthquake load is of very short duration, it is assumed that each concrete fiber and steel strand follow their strain-stress relation for quickly applied load from the state of equilibrium reached after all the losses have occurred.

The monotonic M-Φ relation under constant axial load (P) was calculated following the procedure described in Joen and Park (1990). The ultimate curvature (Φu) is reached when a sudden drop in flexural strength is observed, due either to the rupturing of one strand or the excessive crushing on the compression side of the cross section. The corresponding local ductility values are defined as $D\Phi=\Phi u/\Phi y$, with Φy equal to the yielding curvature in a elastic-perfectly-plastic M-Φ curve with the same Φu and with Mu equal to 95% of the maximum flexural resistance. The results are shown in Figs. 5 and 6 for a pile with 28 strands (together with the results for equivalent materials, see following paragraphs). The calculated $D\Phi$ values are between 2 and 3 for the compression load levels of interest. As a comparison, Joen and Park (1990) report local ductilities of the order of 9 to 12 for prestressed octagonal piles laterally confined with spirals. With the maximum moments for each axial load the Pu-Mu diagrams of Fig. 7 were obtained. The diagrams without losses and without prestress are also included, so that the effect of these actions can be assessed.

Approximate modeling of the local behavior of the piles

The program available to perform the non-linear analysis can handle non-linear plastic materials through the definition of the strain-stress relation for it, as long as there is only one material in the cross section of the beam-column elements (homogeneous cross section). In order to overcome this limitation, an equivalent material was defined with a non-linear strain-stress relation such that the corresponding M-Φ-P curves (calculated with the original cross section) approximate those obtained with the actual concrete and steel properties. Obviously, it is not possible to obtain a good approximation for all axial load levels, hence two different material were defined: i)one for the pile in compression between 100 and 300 t, and ii) one for the pile in tension between 180 and 100 t.

Three different material models in each case were used (Figs. 8): i)elastic-plastic material with a (isotropic) hardening and a softening branch, which allows to model the sudden drop in flexural resistance beyond the ultimate curvature; ii)bilinear elastic-plastic material with a (kinematic) hardening branch, which model piles of similar strength but much larger local ductility; and ii)elastic-perfectly plastic material, which also would imply very large local ductility but with no increase in moment under constant axial load. The nature of the hardening rule (isotropic or kinematic) is not important due to the monotonically applied loads.

In Fig. 5 the M-Φ-P curves for 90, 180 and 350 t in compression are shown for both the real pile and the one with the equivalent material with a softening branch. The agreement is reasonable for 90 and 180 t, but predicts a lower flexural strength for 350 t. In Fig. 6 the same curves are shown for 100 y 180 t in tension. In Fig. 9, the axial strain-axial load curves for tension and compression are given. In this case none of the models predicts the gradual decrease in tension stiffness after the pretension is overcome. This gradual decrease is due to the creation of discrete tension cracks and it is very difficult to model.

The same curves can be constructed for the other materials. The agreement is approximately equal, except for curvatures close to the ultimate. It was assumed that the response obtained with these equivalent materials would model with enough approximation the monotonic response of the prestressed concrete piles in the range of axial loads of interest.

RESULTS OF NON-LINEAR STATIC PUSH-OVER ANALYSIS

The non-linear push over static analysis of the coupled battered piles allows the identification of the collapse mechanism, the parameters that control it, and the assessment of the global ductility of the substructure. The analysis were carried under the following hypotheses: i) a 2D model was used, assuming that the pile cap is braced by the couples placed in planes normal to the one being analyzed through the horizontal diaphragm. Due to the axisymmetry of the cross section of the pile, it makes no difference to restrain the out-of-plane buckling of the compressed pile between its supports; ii) the local response of the pile and the soil-pile interaction are modeled as previously explained, with the t-z, Q-z and p-y non-linear springs reacting along the undeformed direction of the axis of the pile and normal to it; iii)the effect of geometric non-linearity due to large displacements is considered by the analysis program; iv)the strength and integrity of the connection between the piles was checked. The piles are connected by steel pipes grouted to the piles and joined by a welded plate (Fig. 2). A concrete cap with a cage reinforcement is poured around this assembly. The shear strength of the cap, through friction action, is large enough to resist the axial reactions of the piles (Parra, et al. 1995); and v) the pile cap was assumed to be unrestrained.

The piles have a slope of 6:1 to the vertical, with a total length of 50,7 m. In the first step of the analysis, a vertical force of 140 t (1400 kN) is placed on the pile cap to model the vertical loads resisted by the couple. In the second step a controlled horizontal displacement is applied to the pile cap. In the following paragraphs some of the results obtained are discussed, along with the implications for seismic design.

Effect of pile-soil interaction. Sensitivity to local behavior of the piles.

Without considering the soil and with the ends of the piles completely fixed, the lateral force (FL) vs. lateral displacement (ΔL) relations at the cap are as shown in Fig. 10. If the non-linear local response of the piles is considered, through the non-linear equivalent materials previously defined, the response is non-ductile. After the maximum load is reached, a small increment in ΔL causes a sudden strength reduction, with snapback instability (instability under control of displacement) (Bazant and Cedolin, 1991). The same behavior is observed for the three different materials models, which implies that the lack of global ductility of the couple is not sensitive to the local curvature ductility of the piles.

If the pile-soil interaction is considered (for the type A soil layering), the FL-ΔL curves changes as shown in Fig. 11. The behavior is controlled by the strength of the soil under axial load. The first peak corresponds to the development of the friction resistance along the shaft of the compression pile. After it, the residual shaft resistance is achieved, causing a reduction in lateral strength. The load then increases due to the development of the end bearing capacity. The total capacity is first reached in the compression pile. The ultimate lateral resistance can be approximately predicted by consideration of the limit static equilibrium of the cap. The response is insensitive to large changes in the p-y springs.

The maximum displacement in each FL-ΔL curve corresponds to the displacement beyond which no convergence is achieved. The reason for the lack of convergence is the localization of damage (curvature) in a softening plastic hinge due to the effect of the increasing axial load (Bazant and Cedolin, 1991). The finite element used in the analysis has a linear variation of rotation and a constant curvature. Each element has a length of 0,5m, hence the minimum plastic hinge length implied by the discretization is 0,5m. Analysis with 50 and 200 elements gave almost the same results. The maximum lateral displacements are 1,8; 2,2 and 2,3m for the hardening-softenig, perfectly plastic and plastic-hardening materials. The maximum difference (~23%) is not as large as expected, given the much larger differences in local ductilities in the piles associated with the different material models.

In Fig. 12 the curvature distribution along the piles is shown for ΔL=1,7m (the numbers in the X axis correspond to the element's ID numbers: 1 is the lower left element, 200 the lower right, 100 and 101 are connected to the pile cap). There are two plastic hinges, one in the tension pile at the mud-firm soil contact and the other in the compression pile, 6 m above the water-mud contact. For the model with softening material a small increment in ΔL causes a very large curvature increase in the compression hinge. This hinge is closer to its ultimate curvature, about 0,006 rad/m under 350 t in compression (Fig. 5), than the tension pile ($\Phi u \approx 0{,}03$ rad/m). In the other models, although larger curvatures are predicted, there is convergence for larger ΔL values, and even snapback for the plastic-hardening case.

It is assumed that the maximum displacements associated with lack of convergence in the models with the softening material are a conservative estimate of the ultimate displacements. Formal stability analysis to determine the snapback point under the assumed plastic hinge length are possible, following the methods described in Bazant and Cedolin (1991).

Effect of different soil layerings and of axial capacity of the soil

The FL-ΔL curves for different soil layerings (A,B,C) with equal axial capacity (300t), and for a weaker and stronger soils (200t and 500t axial capacity) are shown in Fig. 13. The models with different layering show almost the same response. Although the details of the force transfer between the pile and the soil are different, the global response of the couple is controlled by the total axial capacity due to the soil resistance. For the weaker soil the maximum FL value is lower, but the system has a greater ductility. For the stronger soil, there is hardly any ductility available in the system, because the compressed pile reaches its ultimate curvature just before the maximum axial capacity of the soil is developed. The FL-ΔL curve is similar to the one obtained by assuming that there is no soil and pile ends completely fixed. Hence, a greater strength in the soil (i.e. an axial capacity greater than 500t) does not imply greater lateral resistance.

Effect of restraints in the pile cap

In all the results presented it has been assumed that the pile cap has no restraints to its rotation and vertical translation and that there is continuity in the pile-cap connection. This is equivalent to assume that the stiffness in a vertical plane of the bracing beams and horizontal diaphragm is negligible. The effect of lack of continuity in the cap connection (i.e. hinged connection) was investigated, and it was found that the response, as described by the FL-ΔL curves is basically the same (Mibelli and Díaz, 1994). With regards to the stiffness of the bracing beams, two extreme case were analyzed: one with the rotation and the vertical translation restrained, modelling the effect of very stiff bracing beams connected in their far ends to vertical piles axially rigid, and the other with only the vertical translation restrained. In Fig. 14 the resulting FL-ΔL curves are shown. It can be concluded that if the lateral strength is controlled by the soil capacity, restraining of the pile head causes a small increase in lateral strength and a decrease in ductility due to the formation of a plastic hinge in the pile-cap connection. If only the vertical translation is restrained, the available ductility is the same as with an unrestrained cap. There are many possibilities if the finite flexural strength of the bracing beams is considered, but it is believed that none of them would imply an important change in the response and available ductility of the couple.

Summary of ductility factors under monotonic loading

In Table 2 the ductility factors under monotonic loading (D_Δ) for the type I couples are shown. They were obtained from an equivalent elastic-perfectly plastic FL-ΔL relation with the same ΔLu. The contribution of the first peak was ignored and the secant stiffness was taken at the minimum immediately after it. The yielding displacement (ΔLy) and the maximum lateral strength (FLu) are then calculated so that the total area under the real and equivalent FL-ΔL curves are equal.

If a smaller ΔLu value is chosen, the non-linear response would be confined only to the soil surrounding the piles. The effect of rapidly applied cyclic loading could then be inferred from the cyclic response of the soil. Following API-RP2A (1993) there are two opposite effects: a strength reduction due to degradation and a strength increase due to the high load rate. A research project is currently in progress with the objective of taking into account this effects.

Table 2. Ductility factors of coupled battered piles under monotonic lateral loads

Model (Layering/material)	Axial capacity (t)	ΔLy (m)	ΔLu (m)	FLu (t)	D_Δ
A/e-p-hard-soft	300	0,176	1,70	75,6	9,7
A/e-perfect-plastic	300	0,175	2,26	75,2	12,9
A/e-plastic-hard.	300	0,173	2,34	74,2	13,6
B/e-p-hard-soft	300	0,180	1,69	76,1	9,4
C/e-p-hard-soft	300	0,203	1,69	75,8	8,4
D/e-p-hard-soft	200	0,146	2,29	44,2	15,6
E/e-p-hard-soft	500	--	--	136	1

CONCLUSIONS

The couples of battered piles provide most of the lateral strength and stiffness to the platforms in Lake Maracaibo. The piles typically used are locally non-ductile. If the axial capacity of the piles associated with the strength of the soil is lower (at least by 50%) than the axial load associated with the collapse of a couple of battered piles under lateral load, without soil and with its ends completely fixed, then the system would be ductile. The reason is that the early axial yielding of the soil delays the exceedance of the flexural-axial strength of the piles, hence limiting the local ductility demand in them. This means that the platforms would have a greater global ductility than what can be inferred from the local ductilities of the piles alone and that their seismic reliability would be better than expected, although at the cost of large permanent displacements after an extreme event. More analysis under rapidly applied cyclic loads are necessary in order to verify this conclusion.

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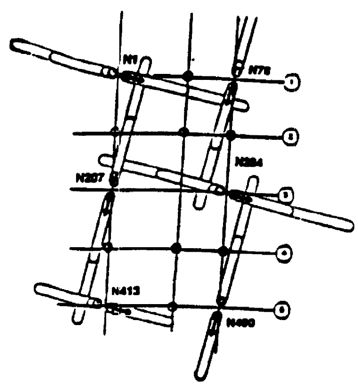

Fig. 1. Typical plan

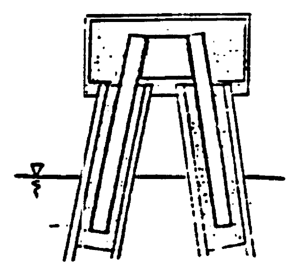

Fig. 2. Type I couple.

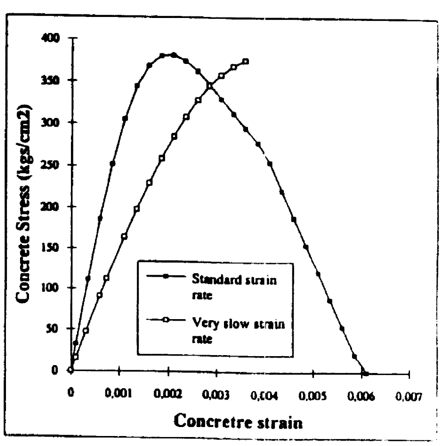

Fig. 3. Strain-stress relation for concrete

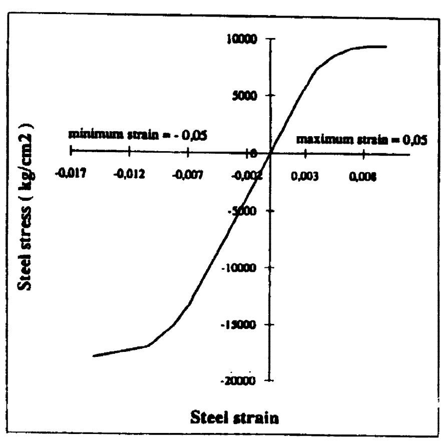

Fig. 4. Strain-stress relation for prestressing steel

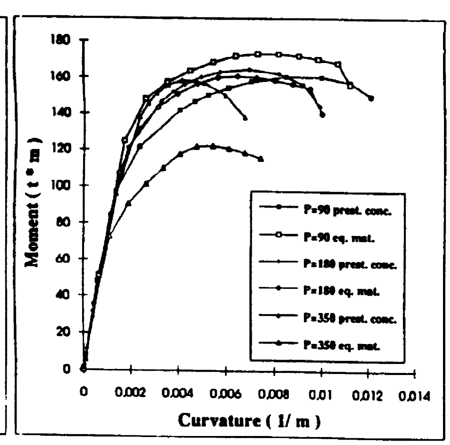

Fig.5. M-Φ-P under compression loads.

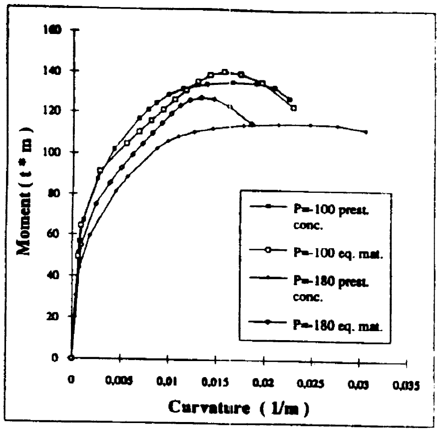

Fig. 6. M-Φ-P under tension loads.

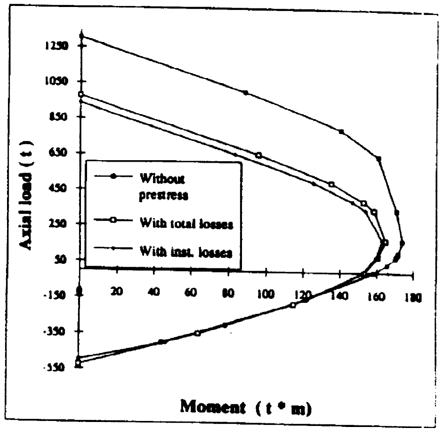

Fig. 7. Mu-Pu interaction diagram.

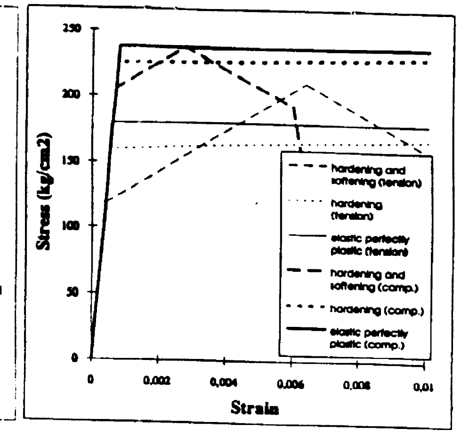

Fig. 8. Equivalent materials.

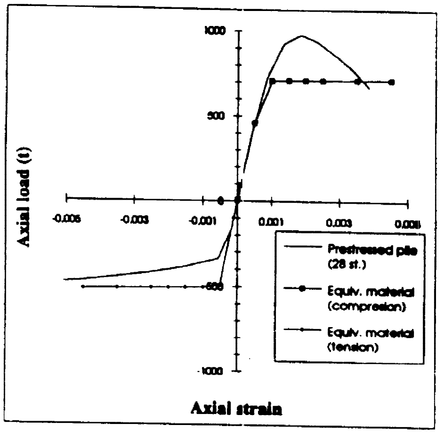

Fig. 9. Axial strain-axial load relation.

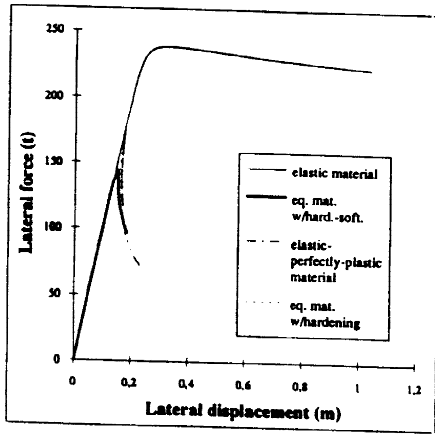

Fig. 10. FL-ΔL curves without soil.

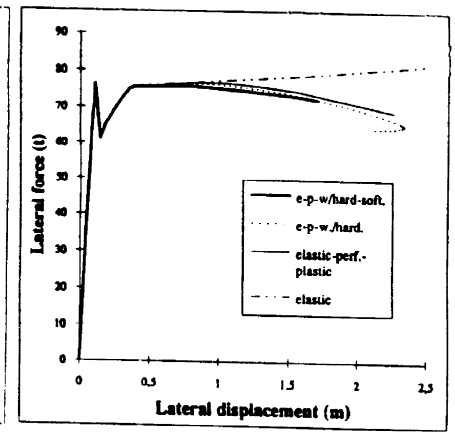

Fig. 11. FL-ΔL curves with pile-soil interaction.

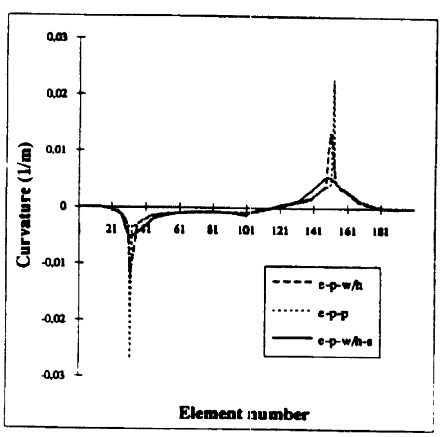

Fig. 12. Curvature distribution.

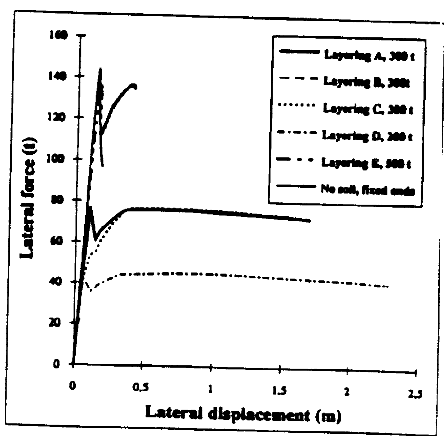

Fig. 13. FL-ΔL curves for different soils.

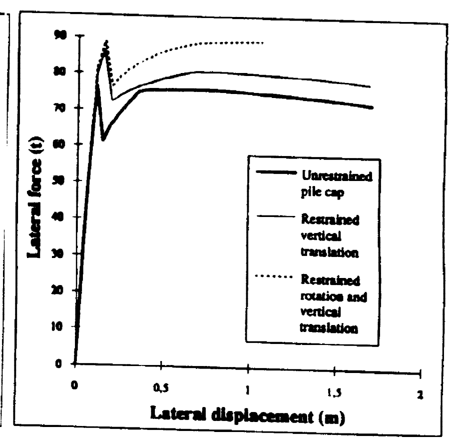

Fig. 14. FL-ΔL curves for different pile cap restraints.