Diaphragm wall panels subject to horizontal static loads

F. COLLESELLI *, A. MAZZUCATO **

SUMMARY: The results obtained from horizontal static load tests on two instrumented diaphragm wall panels are analyzed in this paper.
Experimental data and design forecast based on available solutions are examined. Flexural rigidity seems to have fundamental importance on the behaviour of the panels either in working or failure phases.

1. Foreword

Although the literature on the subject is quite extensive, estimates on the behaviour of structures undergoing horizontal loads still present some problems in connection with the choice of parameters to be used in determining the degree of stress and deformation.

In designing such structures, one normally resorts to models which, because of the type of soil involved, take into account the initial soil modulus in the general expression:

$$E_{si} = E_{si\,z=o} + k_i z \qquad (z = \text{depth})$$

where the results obtained by REESE *et al.* [1974 and 1975] and GARASSINO *et al.* [1975 and 1976] may be used for the gradient of modulus k_i and the value of $E_{si\,z=o}$.

When soil deformations become greater, the initial modulus E_{si} may be substituted by the secant modulus of soil reaction $E_s = E_s\ (z, y)$ varying with depth below ground surface and pile deflection.

Recent studies on the subject and the conclusions arrived at during the most recent international congresses [JAMIOLKOWSKI and GARASSINO, 1977] show that the method of calculation based on the relationship of load displacement gives satisfying results in everyday practice. According to the above Authors, the main uncertainties lie in the choice of initial tangent modulus E_{si}, and in the maximum resistance of soil P_u. It must be assumed that P_u depends not only on soil characteristics but, besides the width of the element, also notably on its stiffness.

Flexural rigidity therefore conditions the behaviour of such elements, both in the working stage and where soils approach their ultimate resistance.

Indeed, due to the presence of numerous factors whose influence is not yet completely clear, the study of diaphragm wall panels subjected to horizontal loads is extremely complex, as are also interaction problems.

2. Soil conditions

Static load tests were programmed and carried out on the site of the new Thermoelectric Power Plant at Sermide (Mantova) to determine the behaviour of the wall panels and obtain parameters for the design of foundations subjected to strong static horizontal loads (chimney, intake and discharge works).

Soils in which wall panels have been built are described by COLLESELLI and TRIPICIANO [1978], COLOMBO *et al.* [1979], MAZZUCATO [1980]. In the test area, the geotechnical survey gave the stratigraphy shown in Fig. 1.

The same figure also shows the two diaphragm walls, their planimetric position and respective dimensions (1.0 × 2.50 and 1.0 × × 4.5 m). The 40 m diaphragm walls reached a depth of −27.00 m, with a top at +13.00 m above sea level.

The diaphragm walls were built after the construction of a general sand fill with height varying from 2.5 to 4.5 m with respect to the original level at +9.5 ÷ +10.0 m a.s.l. Starting from the top of the general embankment, the stratigraphic situation may be summarized as:

Layer I: to a depth of about 4 m, made with a fill of fine sand characterized by a penetrometric static point with varying resistance between 5 and 8 MPa. Several density tests *in situ* and in the laboratory on undisturbed samples were carried out in this layer; dry density was between 15.3 and 15.5 KN/m^3.

* Associate Professor, Università di Padova.
** Researcher, Università di Padova.

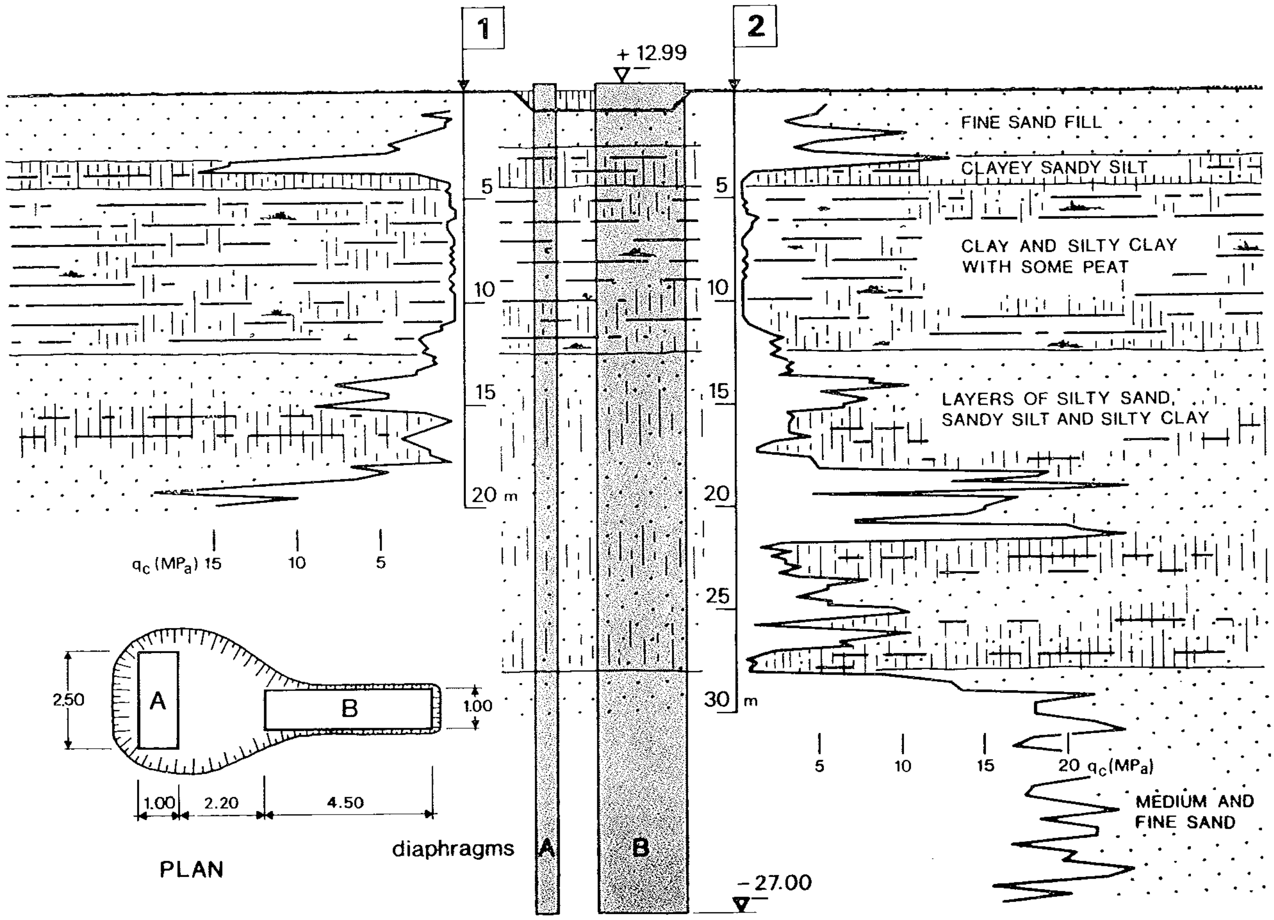

Fig. 1. - Soil profile and characteristics of diaphragm wall panels.
Fig. 1. - *Caratteristiche stratigrafiche della zona di prova.*

Extension triaxial tests were also performed on reconstructed samples to reproduce the tensional conditions found in front of the diaphragm walls. Fig. 2 shows the results of the extension tests on samples of dry density varying between 15.3 and 16.4 KN/m^3.

Layer II: to a thickness of 9-10 m, consisting of medium to stiff clay and silty clay, normally or slightly over-consolidated. Triaxial C.I.U. tests showed a ratio of undrained shear strength and initial tangent modulus of 150 and 200, and lateral deformation at 50% of the maximum deviator stress less than 1%.

Layer III: to a depth of about 16 m, formed of layers of sand ($q_c = 10$ MPa) and sandy silt ($q_c = 5 \div 8$ MPa) containing layers with a maximum thickness of 2 m silty clay of average consistency.

Layer IV: to a depth of about 25 m, consisting of medium and fine dense silty sand characterized by cone resistence values of 10 MPa.

The triaxial compression C.I.U. and C.K$_0$.U. tests carried out on the sandy soils of the third and fourth layers gave friction angles varying between 36° and 44°, with initial tangent modulus of 200-300 MPa [Colombo *et al.*, 1979].

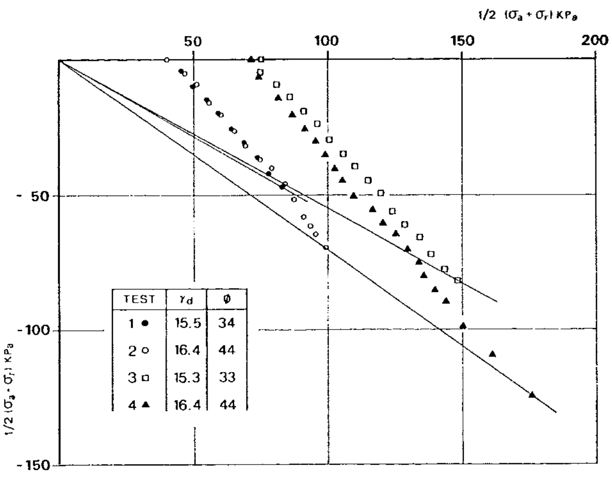

Fig. 2. - Triaxial (CIDLC) test.
Fig. 2. - *Risultati delle prove triassiali CIDLC.*

3. Instrumentation used for monitoring stress and deformation

The design of the instrumentation to be installed aimed, by means of different instruments, at revealing levels of tension and deformation of the diaphragm-soil system, in order to study interaction phenomena by means of the classic equation:

$$EJ \frac{d^4y}{dz^4} + E_s y = 0$$

in which y = panel displacement, EJ = flexural rigidity, and E_s = modulus of soil reaction.

Strain-gauge cells with full wheatstone bridges, inclinometers and servo-accelerometers to measure rotation of the top were inserted into the diaphragm walls.

Horizontal movements were measured by transducers of displacement (LWDT).

The five sections, with strain-gauges placed at the edges of the diaphragms at intervals of 3-5 m, covered a distance of about 20 m from the horizontal load axis (Fig. 3).

This instrumentation, also used for the control of other constructions in the same building yard, always gave reliable and concordant data.

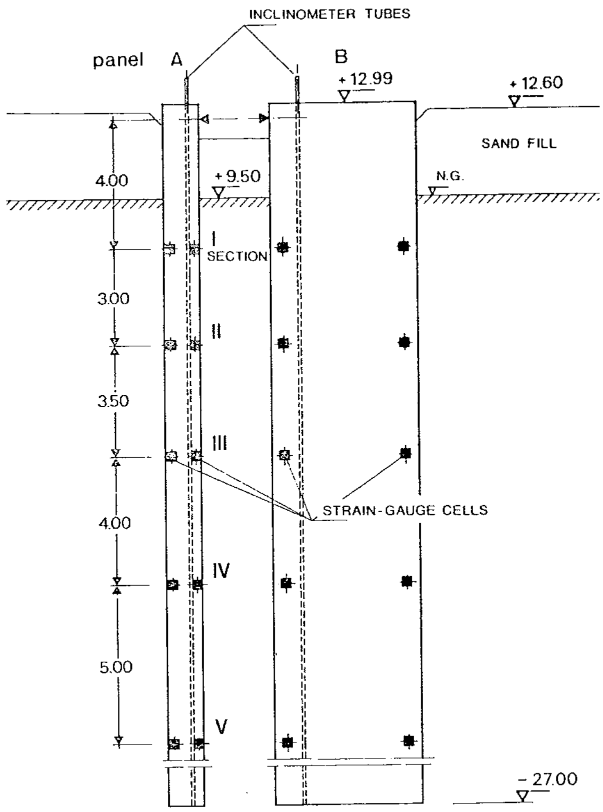

Fig. 3. - Location of strain-gauges and inclinometers.
Fig. 3. - *Strumentazione installata nei diaframmi.*

4. Load tests and analysis of results

The diaphragm design aimed at carrying out two structures of very different rigidity, so as to show better the relatively flexible and rigid elements; this was why the planimetric lay-out shown in Fig. 1 was chosen.

In evaluating stress state in the two elements, we referred simply to a gradient of reaction modulus E_s with a variation between 2×10^4 and 0.3×10^4 KN/m³; the more rigid element was dimensioned using a $k = 2 \times 10^4$ KN/m³ modulus, while the above-mentioned values were chosen for the more flexible element. Although it was recognized that a similar working method was somewhat empirical, it sufficiently well summarized the results reported in the literature [BAGUELIN-JEZEQUEL, 1971; BAGUELIN *et al.*, 1972; DYSLI, 1976; GARASSINO *et al.*, 1976 and MANOLIU, 1976].

On the other hand, at this stage, it was a question of grouping together into a single variable the process of diaphragm-soil interaction, starting from standard test results such as penetrometric and triaxial laboratory tests.

From these analyses it was observed that, with loads between 1.4 and 1.8 MN, element A showed elastic behaviour within the diaphragm wall-soil system, with a maximum top displacement of approximately 10 mm, while quite relevant movements were expected with higher loads.

In this range of tension, and for loads of about twice as much element B always showed elastic behaviour.

During tests, the horizontal static load was applied level with the ground surface with an increase of 0.2 MN up to a value of 4 MN.

Fig. 4 shows the load displacement curves obtained on the two diaphragms. In the first cycle, panel A showed elastic behaviour up to 1,2 MN; at 1.6 MN it underwent a variation characterized by an increase of five times the displacement observed in the first cycle, with the shorter side of the diaphragm, and by possible cracking of the concrete. Fig. 5 shows the strain-gauge readings at about 4 m from the point of applied load. It may be seen that deformations in compression and tension were more or less equal to the average values of 100 με; beyond this point the readings, taken

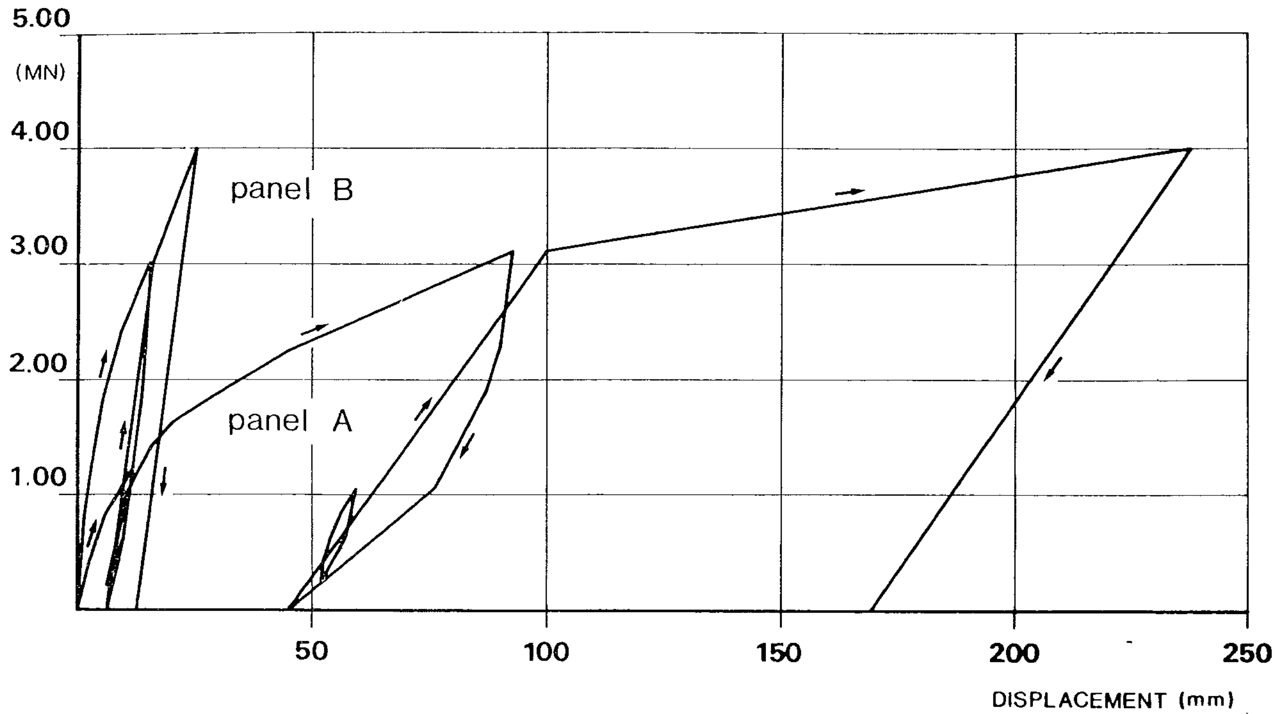

Fig. 4. - Load deflection curves.
Fig. 4. - *Relazioni sforzi orizzontali - spostamenti dei diaframmi.*

at tension points, underwent sharp variations which may be interpreted as the initial cracking stage in the concrete. Tension strength in the concrete was evaluated as equivalent to about 3 ÷ 4 MPa; this calculation was made by referring to the lowest values of the concrete modulus which, by means of laboratory tests, was shown to vary between 50.000 and 30.000 N/mm^2 with deformation between zero and 950 $\mu\varepsilon$.

Although top displacement reached maximum levels of 0.24 m, the structures did not collapse with the increase in applied force, as seen by strain in the tension and compressed areas: 180 $\mu\varepsilon$ and 1240 $\mu\varepsilon$ (Fig. 5). Very probably, the contemporary cracking of the soil and the increased displacement and cracking of the concrete were the moment of transition from one type of behaviour to another when the structural element transferred horizontal load to the surrounding soil. It may be assumed that, initially, the element behaves almost rigidly, influencing the soil at great depth. At the moment of cracking, horizontal load is redistributed with depth, influencing the upper layers in relation to the greater deformation of the soil and also, probably, to the lesser capacity of the structure to transfer horizontal loads in depth.

Panel B did not show the same behaviour. The diaphragm-wall system, above all with applied loads, always reacted elastically, with maximum displacement at 25 mm with an applied load of 4 MN. In this case, the tests did not reach sufficient tensional levels to trigger cracking phenomena.

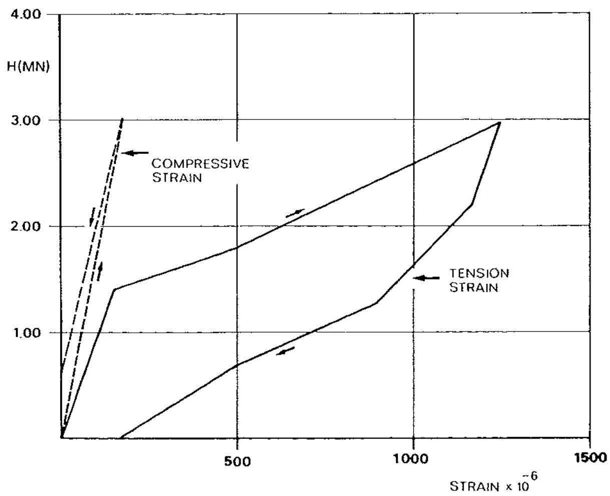

Fig. 5. - Strain-gauge reading at second section (-7.00 m) from G. L.
Fig. 5. - *Letture estensimetriche al secondo livello strumentato del diaframma A in funzione dello sforzo orizzontale applicato.*

Fig. 6 shows measurements of diaphragm displacement taken by the inclinometer placed along them. These measurements confirmed the hypothesis on the behaviour of diaphragm walls during tests: in particular, it was observed that with high loads only the first 6-7 m of panel A were influenced, while for most of the length of panel B the soil was involved.

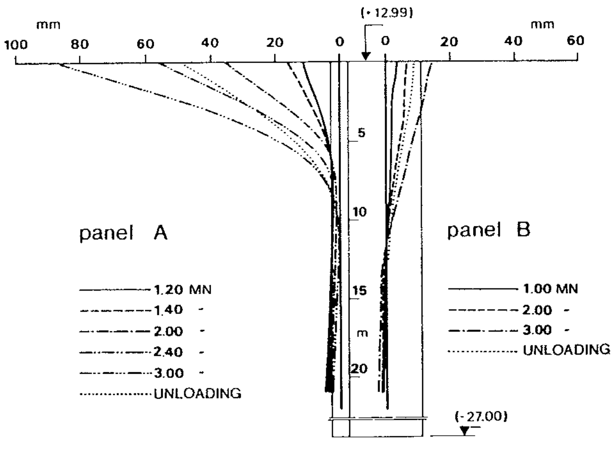

Fig. 6. - Inclinometer measurements.
Fig. 6. - *Misure inclinometriche sui due diaframmi al variare dello sforzo applicato.*

5. Interpretation of inclinometric data and strain-gauge measurements

Measurements obtained from inclinometers and strain gauges were used for examining the flexible behaviour of the more flexible element A when subjected to loads at the top and the reaction of the soil. A sufficiently stable polynomial form was given to the inclinometric data, so that pressure and displacement parameters could be determined by derivation [GARASSINO *et al.*, 1975]. By means of strain-gauge measurements, other parameters were reached, starting from the rotation of the sections and considering the concrete first in its elastic phase and then, with increasing force, in its cracking stage [MACCHI-SIVIERO, 1974]. Data from the strain-gauges, placed at -8.60 and -5.40 m in the two sections, clearly showed the nonlinear link between moments and curvatures with great variations in rigidity between 1.2 ÷ ÷ 1.4 MN; these values also corresponded to the end of the elastic phase of the diaphragm-soil system and the beginning of behaviour characterized by growing residuary deformations.

The rigidity noted on varying the forces applied was used for inclinometric data processing by means of derivations. When comparing

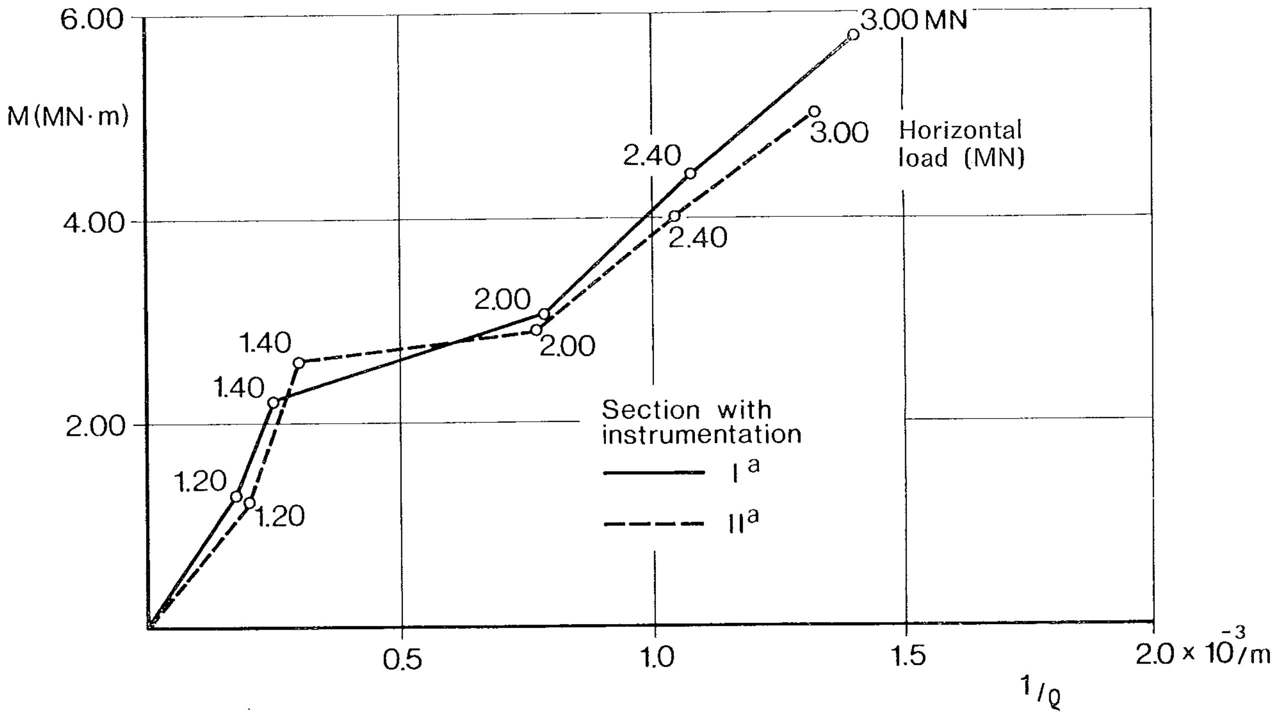

Fig. 7. - Bending moment in sections I and II, with curvatures (Panel A).
Fig. 7. - *Andamento dei momenti flettenti nelle sezioni I e II con le curvature (Pannello A).*

the results obtained, it was observed that the point of maximum bending deduced from the inclinometric data (Fig. 8) was 30÷35% greater than that determined by the strain gauges. Calculating the pressure level by means of the P-y curve method [REESE, 1980] — that is considering the soil as stratified — it gave bending moments 25 ÷ 30% higher than those reached by elaborating inclinometric data, which consider the soil to be homogeneous. Rotation of the tops calculated with $\Delta\varphi = \Sigma \frac{\varepsilon \Delta z}{D}$, using strain-gauge measurements and data obtained from servo-accelometers placed on the tops of the diaphragms, confirmed the reliability of the deformation measurements. Table I shows the rotation measured at the top of diaphragm A, those calculated from strain-gauge measurements, those obtained by elaborating inclinometric data, and those calculated using the P-y curve method.

It may generally be said that the pressure points arrived at by means of these three methods fall well within an acceptably variable field, considering the complexity of the interaction phenomena and the numerous factors involved.

TAB. 1

Load MN	Rotation at top $\Phi \cdot 10^{-3}$ rad Strain-Gauges	Inclino-meters	Servo-accelerometer.	P-Y curves
1.4	2.8	2.4	3.14	3.3
2.0	7.8	8.2	7.15	9.4
2.4	13.3	16.4	17.07	18.0

Elaboration of the inclinometric data showed that, to a depth of 3 ÷ 4 m from ground level, soil reaction modulus E_s increases with depth with a k gradient, function of the displacement (Fig. 9). This figure clearly shows the different reactions of the diaphragm to changes in horizontal load, as already observed, also in relation to the structural behaviour of the element. With maximum displacement between 10 and 15 mm, the gradient of soil reaction modulus E_s varies between 4×10^4 and 1.8×10^4 KN/m³, undergoing a reduction of 55%, while the maximum movement of 80 mm, the gradient modulus to 0.5×10^4 KN/m³, with a total reduction of 88%. The corresponding horizontal forces varied from 1.2 and 1.4, and 1.4 and 3.0 MN, with increase of 17% and 150% respectively.

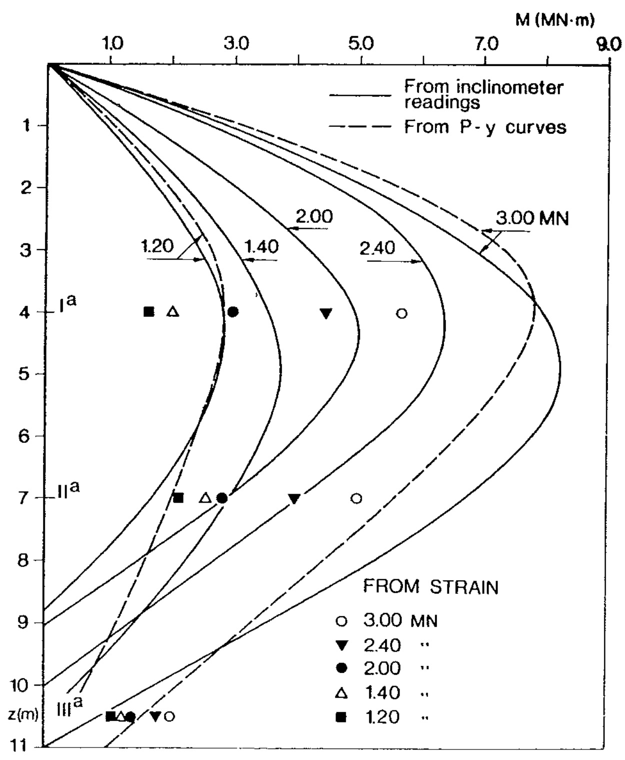

Fig. 8. - Bending moment computed by P-y curves, from inclinometers and measured with strain-gauges (Panel A).

Fig. 8. - Confronto dei regimi flessionali ottenuti con il metodo delle curve p-y, dalle misure estensimetriche e dalle misure inclinometriche (Pannello A).

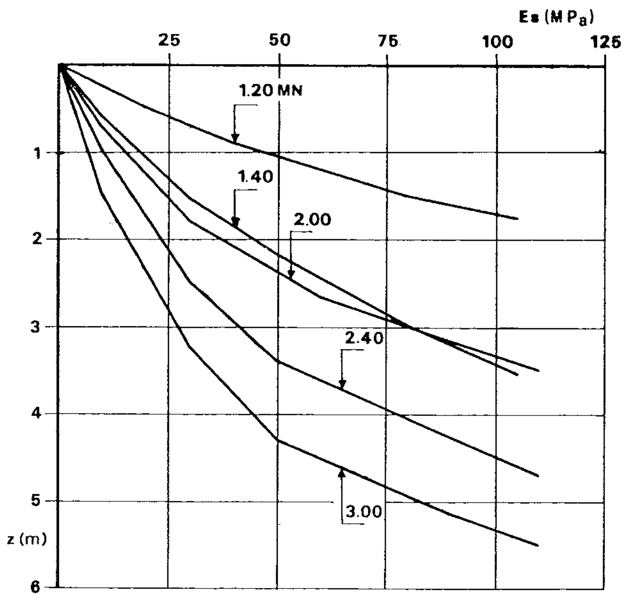

Fig. 9. - Soil modulus variation with depth as function of applied load (Panel A).

Fig. 9. - Variazioni del modulo di reazione con la profondità e con i carichi applicati (Pannello A).

With movements of a centimetre or more and loads corresponding to 1.4 MN, the behaviour of the diaphragm approached that observed in more flexible structures, as reported in the literature. A comparison was therefore made of the reaction modulus gradient within this field of movement. For k varying between 0.5×10^4 KN/m^3 and 1.8×10^4 KN/m^3, with maximum movements of 80 mm and 10 mm respectively, the interpolant function of the values arrived at gave the expression $k = 7.7 \times 10^4 y^{-0.628}$ (Fig. 10).

The initial tangent gradient modulus E_i, once the reaction curves P-y of the soil were normalized with hyperbolas [REESE, 1974], turned out to be equal to 3×10^4 KN/m^3.

The values reached are in agreement with the results proposed by REESE *et al.* [1974] for calculation of pressures limited to medium-density sands, such as those examined here, characterized by variable static penetrometric resistance of between 5 and 8 MPa.

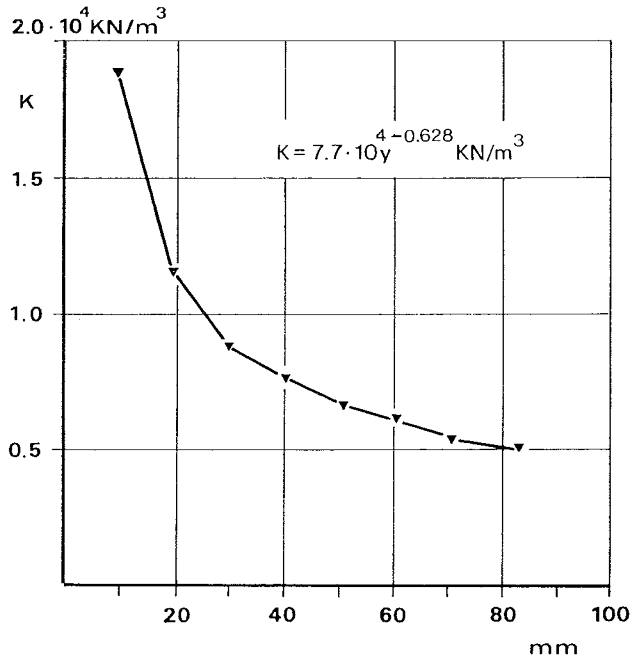

Fig. 10. - Gradient of soil modulus as function of top displacement.

Fig. 10. - *Gradiente del modulo di reazione dello strato superficiale col variare dello spostamento della sommità del diaframma.*

6. **Final remarks**

The behaviour of buried structures undergoing horizontal forces seems to be clearly influenced by their rigidity and flexural characteristics which, influencing the kinetics with which the soil-structure system approaches the plastic stage, condition the tensional level formed in them.

Normal calculation methods seem to give results which fall within the range of acceptable approximation with limited deformation and maximum shifting of the top within the order of the centimetre. The reaction of the subsoil may generally be represented by the initial tangent modulus E_{si}, the choice of which in a relatively extensive field may be made on the bases of data obtained from the literature [JAMIOLKOWSKI and GARASSINO, 1979].

When deformations and displacements of the top become notable, the reaction of the soil changes. In fact, these marked deformations are to be associated with the relative rigidity of the structure and the level of stress it has reached, and involve the formation of wedge-shaped kinetics in the layers, which vary with both type of soil and structure.

Within the field of great deformations, soil reaction may on the whole be represented by tangent modulus E_s, which varies with displacements and decreases with them. The values which, case by case, may be obtained for E_s by elaborating test data cannot be separated from the bending characteristics of the structure, since the ultimate resistance of the soil does not depend only on soil characteristics, but also on the flexural and constraint features of the structures.

These criteria must still be identified, so as to be able to separate the two components which are very important for the correct design of such structures. A proposal in this direction could be that of referring to initial tangent moduli [REESE *et al.*, 1974] and to the initial rigidity of the element, while tensional levels remain at acceptable values in the non-cracking stage. Later, with increase of horizontal load on the initial moduli, the tangent modulus suggested by TERZAGHI [1955] should be used, and the rigidity corresponding to the tensional level reached should be considered. However, while small displacement behaviour of flexible and relatively rigid structures may be represented by moduli E_i, particular attention should be taken in choosing moduli E_s when wishing to identify stress levels in the field of large deformations. For large constructions, it seems indispensable, in the working stage, to resort to full-scale tests, with suitable instrumentation capable of showing the actual behaviour of the structures.

SOMMARIO

Strutture interrate soggette ad azioni orizzontali statiche

Nel presente articolo vengono analizzati i risultati ottenuti da prove di carico eseguite su due diaframmi in calcestruzzo costruiti in opera, completamente immersi nel terreno, strumentati e soggetti ad azioni orizzontali. Le prove di carico sono state eseguite nell'area della Nuova Centrale Termoelettrica di Sermide (MN) per valutare il comportamento di pannelli di diaframma e ricavare parametri per il dimensionamento di fondazioni sollecitate da forti azioni orizzontali (camino, opera di presa e di restituzione etc.). I due diaframmi disposti trasversalmente (fig. 1) hanno dimensioni di $1.0 \times 2,50$ e $1.0 \times 4,5$ m rispettivamente, una lunghezza di 40 m e immersi in un terreno costituito da alternanze di strati argillosi, limosi e sabbiosi, terreni ampiamente descritti e caratterizzati da COLLESELLI e TRIPICIANO [1978] e COLOMBO *et al.* [1979].

I diaframmi sono stati strumentati con celle estensimetriche, con inclinometri e servoaccelerometri per il rilievo delle rotazioni in sommità (fig. 3). Nella fig. 4 vengono riportati i risultati della prova di carico in cui la forza orizzontale è stata applicata con incrementi di 0.2 MN sino ad un valore massimo di 4 MN. Il pannello A in corrispondena del carico di 1.6 MN subisce una variazione di comportamento con spostamenti cinque volte maggiori di quelli osservati nel primo ciclo sino a 1.2 MN.

Dalle letture estensimetriche (fig. 5), in corrispondenza della sezione di maggiore sollecitazione, si osserva che gli estensimetri posti nelle parti tese subiscono una brusca variazione che si può interpretare come l'inizio dello stadio fessurativo nel calcestruzzo.

Nella prova di carico non si è raggiunto il collasso della struttura sebbene lo spostamento in sommità abbia raggiunto il valore massimo di 0.24 m. Risulta interessante osservare come in un primo momento l'elemento si comporti rigidamente interessando il terreno in profondità e al momento della fessurazione si abbia una ridistribuzione dell'azione orizzontale con maggiori sollecitazioni trasferite agli strati superficiali.

Il pannello B ha mostrato un comportamento praticamente elastico con spostamenti massimi in sommità di 25 mm con il carico di 4 MN.

Le misure inclinometriche degli spostamenti dei diaframmi (fig. 6) confermano le ipotesi del comportamento dei pannelli; in particolare per il pannello A si osserva che con carichi elevati sono interessati da grandi spostamenti solo i primi 6-7 m.

Per il pannello A i dati inclinometrici sono stati elaborati per ricavare i parametri di sollecitazione e di spostamento [GARASSINO *et al.* 1975]; le misure estensimetriche hanno permesso di ricercare tali parametri partendo dalle rotazioni delle sezioni, considerando il calcestruzzo dapprima in fase elastica e poi in fase fessurata [MACCHI e SIVIERO, 1974].

Dal confronto dei risultati ottenuti per il pannello A (fig. 8) i momenti flettenti desunti dai dati inclinometrici risultano superiori del 30-35% a quelli determinati con gli estensimetri.

Con il metodo delle curve P-y [REESE, 1980] i momenti flettenti sono maggiori del 25-30% di quelli determinati con le misure estensimetriche.

Le rotazioni in sommità (tab. I) calcolate impiegando le misure con gli estensimetri, con i servoaccelerometri, con i dati inclinometrici e calcolate con il metodo delle curve P — y confermano l'attendibilità delle misure di deformazione. Dalla elaborazione dei dati inclinometrici il modulo di reazione del terreno E_s risulta crescente con la profondità, fino a 3-4 m dal piano campagna, con un gradiente k funzione degli spostamenti (fig. 9) e con una diversa risposta del diaframma al variare dell'azione orizzontale.

Il gradiente del modulo tangente iniziale E_i, una volta normalizzate con delle iperboli le curve di reazione del terreno [REESE, 1974], risulta essere pari a 3×10^4 kN/m³, valore in accordo con i risultati proposti da REESE *et al.* [1974].

Dall'esame dei risultati ottenuti il comportamento di strutture interrate sollecitate con forze orizzontali sembra essere sensibilmente influenzato dalla loro rigidezza e dalle loro caratteristiche flessionali che condizionano il cinematismo con il quale il complesso terreno-struttura si avvicina alla fase plastica.

Le metodologie di calcolo usuali sembrano fornire dei risultati che rientrano in un campo di approssimazione accettabile; con deformazioni limitate e con spostamenti massimi in sommità dell'ordine del centimetro, la reazione del terreno si può globalmente rappresentare con il modulo tangente iniziale E_{si}, la cui scelta può essere fatta in base ai dati disponibili in letteratura [JAMIOLKOWSKI e GARASSINO, 1979].

Con deformazioni e spostamenti in sommità importanti la reazione del terreno si modifica; queste notevoli deformazioni infatti sono collegate alla rigidezza relativa della struttura e al livello di sollecitazione in essa raggiunto e comportano la formazione di un cinematismo del tipo a cuneo variabile con il tipo di struttura e di terreno.

In questa situazione la reazione del terreno si può globalmente rappresentare con il modulo tangente E_s funzione degli spostamenti. I valori di E_s, che si possono ricavare caso per caso elaborando i dati ottenuti nelle prove di carico, non possono prescindere dalle caratteristiche flessionali della struttura in relazione alla dipendenza della resistenza ultima del terreno dalle proprietà dello stesso nonché da quelle flessionali e di vincolo delle opere.

Per una corretta progettazione di tali opere si può far riferimento ai moduli tangenti e alle rigidezze iniziali dell'elemento sino a quando il livello di tensione risulta contenuto e nella fase non fessurata, successivamente all'aumentare delle azioni orizzontali ai moduli tangenziali [REESE *et Al.*, 1974] conviene sostituire quelli proposti da TERZAGHI [1955] considerando la rigidezza corrispondente al livello tensionale raggiunto.

Per opere di una certa rilevanza sembra indispensabile il ricorso a prove in vera grandezza con adeguata strumentazione.

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