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ANCRAGES DANS LES SOLS GROUND ANCHORS

Avec la participation des Comités français de:
- Mécanique des Solis.
- Mécanique des Roches.
- Géologie de l'Ingénieur.
ANCRAJE DANS LES SOLS
GROUND ANCHORS

SESSION SPÉCIALE N° 4

Organisateur : P. HABIB (France)
Co-organisateur : A. CROCE (Italie)
AVANT-PROPOS

La Session Spéciale n° 4 du Congrès International de Mécanique des Sols à Tokyo en juillet 1977 portait sur l'étude des tirants d'ancrages. Elle a été organisée par P. Habib, Directeur de la Revue Française de Géotechnique et par A. Croce (Italie). La qualité des communications présentées justifiait leur publication et c'est notre Revue qui s'en est chargé. Nos lecteurs trouveront ainsi dans ce numéro d'une épaisseur un peu exceptionnelle la documentation la plus récente sur les ancrages.

P. LONDE,
Directeur du Comité de Rédaction.
INTRODUCTION

By A. CROCE
Presidente della
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I am honoured to open the Specialty Session n° 4 of the Ninth International Conference on Soil Mechanics and Foundation Engineering.

The Subject of this Session is:

GROUND ANCHORS

This technique is more and more employed in Civil Engineering to assure safe execution of deep or very deep excavations, to construct piers, to solve complex foundation problems.

At the beginning, more or less twenty years ago, ground anchors appeared to be particularly suitable for rocky formations where structural discontinuities —like bedding planes, joints and similar— could be liable to constitute areas of less resistance.

It was essentially a means to correct defects of a natural body, that on the whole was a compact mass with high resistance to tensile stress.

Doubts on the durability of anchors restricted the use of this technique to temporary supports of natural or artificial slopes.

At that time the capability of anchors to be employed in soils was practically ignored.

In the meantime technologies improved in two distinct, but adjacent fields.

Boring machines, new and new injection materials, sophisticated techniques to introduce and control mixtures in rocks and soils were developed.

Secondary, diaphragm walls —which initially were considered as a mean to counteract seepage— developed quickly in highly reliable static structures.

Wide horizons were opened to ground anchors.

In the same period, Civil Engineering was challenged to design heavy constructions with dimensions unforeseen and unforeseeable a few years before.

Old structural support systems failed to give valid answers. New solutions were required.

The general designer very soon understood advantages offered by ground anchors from his point of view.

The structural engineer very soon became aware he should face problems of interactions between soil or rock and structure.

The general as well as structural designers now rely completely upon the skill of geotechnicians. A heavy burden, therefore, is on our shoulders.

- It is up to us to select a mathematical model capable of representing the true situation.

Where and how is it possible to perform reliable calculations?
This is, maybe, the most critical question since anchors are to be used not only in domestic homogeneous, isotropic rock or soil formations, but also in structurally complex formations where anchors will be more and more needed.

- It is up to us to establish correct values of physical and mechanical parameters. We have, therefore, to plan and perform valuable laboratory as well as \textit{in situ} testing.
- It is up to us to adopt proper technologies both for execution and durability of anchors.
- Last but not least, it is up to us to perform measurements and to understand their results in time while the construction is running, and afterwards.

The subject «GROUND ANCHORS» is extensive; only certain aspects will be discussed in this Session.

Prof. P. HABIB, Directeur du Laboratoire de Mécanique des Solides and Président of the International Society for Rock Mechanics, organized this Session with his well known, deep and wide experience in many fields of Geotechnics.

We are grateful to Prof. HABIB as well as the Authors of communications presented to this Session: they spent effort and time to share with us the best of their knowledge.
Le thème de la Session Spéciale n° 4 avait pour but de traiter les ancrages de barres ou de câbles placés dans des trous forés, puis scellés par des mortiers coulés ou injectés. Le Rapporteur est tout à fait désolé que le texte du thème n’ait pas été plus explicite. Après vingt ans de propositions, provenant de nombreux pays, relatives à des ancrages par plaques encastrées dans le sol et enterrées ont été faites et ont dû être refusées. Cependant, deux communications sur ce sujet ont été retenues parce qu’elles apportent une vision inattendue et des résultats généralisables.

Au total, dix-sept communications ont été présentées, soit presque le même nombre que pour la Session Spéciale n° 15 du CONGRES INTERNATIONAL DE MECANIQUE DES SOLS de Mexico en 1969, dernier Congrès International où ce thème a été examiné. À huit ans d’intervalle, la comparaison est intéressante et montre à quel point la technique a évolué.

Ainsi, en 1969, les problèmes qui paraissaient les plus inquiétants étaient le fluage des sols et la corrosion des aciers. Il est bien clair qu’à l’heure actuelle ces problèmes sont, sinon maîtrisés, du moins suffisamment connus pour ne plus être inquiétants, et l’on sait prévoir aujourd’hui des déplacements différences acceptables, et réaliser la protection des aciers de façon suffisante pour éviter les accidents par corrosion.


Les communications sur les ancrages, reçues à l’occasion du Congrès de Tokyo, n’épuisent pas le sujet, mais permettent de connaître les soucis actuels de ceux qui étudient ou réalisent des tirants.

Malgré leur grand intérêt, elles ne pourront pas être toutes présentées au cours de la Session Spéciale n° 4, faute de temps, et je vais en faire ici un résumé ordonné en forme de synthèse critique de l’état des connaissances actuelles.

J’ai dit, il y a déjà huit ans, que le développement des tirants et ancrages s’était fait dans un dûnement théorique complet, mais avec un empirisme raisonné. C’est toujours vrai actuellement et de nombreux auteurs construisent des modèles réduits pour élucider certains aspects du mode de fonctionnement des ancrages.

Nous avons quatre communications qui présentent des résultats acquis sur modèles réduits. Prenons l’exemple des tirants trop courts ; s’ils sont scellés dans le coin de Coulomb, leur efficacité est nulle. Par contre, si les tirants sont très longs, ce n’est peut-être pas la peine de les allonger davantage. Entre les deux, il y a une longueur optimale qu’Habib, Luong, Tcheng et Auger (France) ont cherché à déterminer en partant de l’idée que la surface de rupture avait une forme géométrique imposée par la nature du sol et fonction des forces extérieures. Ils ont utilisé des modèles en sable dans une similitude très pauvre, avec un modèle simple rigide-plastique qui n’est pas suffisant pour déterminer les déformations avant la ruine du prototype mais uniquement la charge de rupture, ce qui permet ensuite de définir la longueur optimale du tirant.

Basset (Grande-Bretagne) a utilisé lui aussi une similitude simple mais déjà plus élaborée, avec des modèles élastoplastiques en argile. Son but est d’essayer de préciser au moyen d’essais rapides le mode de fonctionnement d’élargissements localisés de l’ancrage. Leur distance ne doit pas être inférieure à 2,9 diamètres. Le terme $N_e$ est alors de 8,5, mais du fait de l’apparition d’un vide derrière l’écoulement, il faut admettre seulement $N_e = 6$ pour les essais de longue durée. La souplesse du câble d’ancrage permet une rupture progressive de chaque élargissement et adoucit la courbe efforts-déplacements en tête. Il reste encore, bien entendu, à définir la sécurité au fluxage et la charge admissible qui ne sont pas dans la similitude.

Kananyan et trois autres auteurs Russes utilisent une similitude particulière dite des modèles approximatifs. Pour cela, on construit des modèles géométriquement semblables pour des ancrages, pour des ancrages champignons, pour des plaques enterrées placés dans le sol réel et après chaque essai, on trace
la courbe réduit efforts-déplacements (c'est-à-dire des efforts divisés par l'effort limite et des déplacements divisés par le déplacement limite). On constate que cette courbe varie peu avec l'échelle du modèle. On cherche alors la loi de la force limite et celle du déplacement limite en fonction de l'échelle et on extrapole jusqu'au réel, ce qui donne une bonne approximation de la courbe réelle efforts-déplacements.

Enfin, Boon et Craig (Grande-Bretagne) ont réalisé des essais sur des ancrages en forme de disques, placés dans un sable, et avec une similitude parfaite grâce à des modèles en centrifugeuse. Ils ont fait apparaître un problème très général, qui est la difficulté de faire des modèles réduits en sable au risque d'avoir des résultats bien loin du réel. Ils ont montré que les formules les meilleures donnent des résultats qui sont trop grands, au moins le double de la résistance à la rupture vraie : les modèles ordinaires doivent donc être maniés avec précaution. Souhaitons donc que des tirants d'ancrage, au sens où nous l'entendons dans la Session Spéciale n° 4, soient examinés un jour en centrifugeuse. Mais il ne faudra pas oublier de simuler la mise en place et sa technologie et notamment les pressions d'injection.

Lors du Congrès de Mexico, j'avais signalé tout l'intérêt des essais en vraie grandeur pour l'analyse du comportement des tirants. Cette recommandation reste toujours valable et nous avons reçu des communications particulièrement enrichissantes.

Evangelista et Sapiio (Italie) instrumentent avec des extensomètres électriques deux barres Dividag verticales placées dans un bain de mortier de 22 cm de diamètre qui assure le scellement dans un sol argileux. Ils font une analyse non linéaire (avec une loi de comportement hyperbolique) et intègrent la formule de Mindlin comme si le scellement n'était qu'un vulgaire pieu, mais en admettant que le mortier du scellement casse dès que le béton supporte une traction de 5 MPa (30 kg/cm²). Ils obtiennent ainsi une relation entre la force et le déplacement en tête qui est bonne, au moins au début. Par contre, la résistance à la rupture est tout à fait surévaluée, ce qui montre que le scellement latéral n'est pas uniforme. D'après les essais d'arrachage, il ne serait pas plus élevé que le tiers ou le quart de C₉.

Fujita, Ueda et Kusabuka (Japon) présentent eux aussi une méthode de calcul non linéaire de la courbe d'arrachement des tirants. Pour estimer la résistance au frottement latéral et la longueur du bulbe actif, ils utilisent un modèle élastoplastique simple à une dimension et la comparaison avec les essais in-situ permet d'évaluer les différentes constantes des formules proposées. Une comparaison est faite entre ces constantes et le nombre N de l'essai de penetration standard (SPT) qui permet une évaluation à 20% de la courbe efforts-déplacements jusqu'à la rupture.

Bustamante, Delmas et Lacour (France) essayaient jusqu'à rupture, puis terrassent pour déterminer, dix tirants dans une argile grasse pour évaluer une charge critique Tₚ (différente de la charge limite Tₘ du rupture du scellement) d'où l'on déduit ensuite la traction de service ou charge admissible Tₐ. C'est une illustration de la nouvelle norme française proposée par les « Recommandations » du Bureau Sécuritas. Cette communication contient énormément de résultats. Par exemple, le flouage sous la charge critique Tₐ a atteint 1 mm ; sous la charge admissible Tₐ, il est seulement de 0.4 mm ; le rapport Tₘ/Tₚ est supérieur à 1.5 ; l'effet de groupe est sensible lorsque les tirants sont scellés à 2 m les uns des autres ; la résistance des tirants est proportionnelle, non pas à la pression d'injection, mais au volume injecté à condition de limiter les pertes ; d'où l'intérêt d'une injection de scellement en plusieurs passes. Dans ces conditions, le cisaillement mobilisé peut atteindre trois fois C₉.*

Ostermayer et Scheele (RFA) essaient en grandeur nature au laboratoire, puis évidemment déterreron, trente tirants scellés sous pression, sous 5 m de sable graveleux ou de graviers sableux à différentes compressions et avec différentes longueurs de scellement, et ils donnent une relation avec le SPT. Avant la rupture, on voit apparaître des effets visqueux dans le matériau pulvérulent : au fur et à mesure que la charge augmente, les contraintes évoluent et se redistribuent vers le pied du scellement. A l'approche de la rupture, il suffit d'attendre pour voir cette redistribution se produire. L'effet est d'autant plus sensible qu'on est proche de la rupture et il peut mener à la ruine différée. L'origine de cette viscoplasticité est un problème pour toute la Mécanique des Sols, même si le phénomène n'est pas douteux.

Je signale au passage que Wernick (RFA) estime dans sa communication que dans le cas des sables, les forts cisaillements correspondant au frottement latéral sont dus à l'effet de la dilatance dans la bande étroite autour du scellement où le glissement se produit. Il souligne que l'hypothèse de la coaxialité des tenseurs contraintes et accroissements de déformations devrait être revue du fait de la dilatance.

Littlejohn, Bruce et Deppner (Grande-Bretagne) analysent cinquante-sept tirants de forte capacité dans un grès stratifié, donc des tirants au rocher. Il s'agit d'un sujet marginal à la Mécanique des Sols, mais les résultats méritent qu'on s'y attarde pour de nombreuses raisons. Le poids d'un cône de demi-angle au sommet égal à 43°, à partir du pied du scellement est insuffisant pour donner les forces constatées. La rupture est essentiellement guidée par les joints verticaux et horizontaux. Littlejohn propose la formule P (kN) = 600 d² (d = profondeur en mètre). La rupture se produit à l'interface coulis-tendon, bien que cette aire soit plus grande que celle de l'interface coulis-roche. Il faut donc ne pas mettre trop de tendons dans le même scellement, et il est conseillé de ne pas dépasser une section d'acier supérieure à 10% de l'aire de la section droite du scellement.

Tous les essais cités permettent de prévoir la résistance d'un ancrage par analogie.

Kramer (RFA) va plus loin encore dans un travail très original. Abandonnant tout espoir de calculer la résistance d'un ancrage, il utilise les résultats de cent trente essais, une moitié dans l'argile, l'autre dans les sables, pour faire de la statistique et des corrélations. Il établit une formule dont l'excès maximal avec les essais ne dépasse pas 60 kN. Comme toute statistique, il faut rester dans le domaine d'application, mais à partir d'un grand nombre d'essais, le résultat paraît intéressant.

Enfin, pour terminer avec les essais en place, il faut signaler la communication de yen et Young (USA), qui présentent des essais très sophistiqués d'ancres marines enterrées à cinq diamètres, ce qui est une grande profondeur pour une ancre maritime. C'est une profondeur de 4 mètres pour un ancrage de Génie Civil. Ce texte est donc tout à fait hors du sujet ; mais il est extrêmement intéressant car les auteurs ont montré le rôle de la succion derrière l'ancrage et celui de la pression interstitielle devant l'ancrage qui atteint 30% de la résistance au début du chargement, pour des
essais d’arrachage en conditions non drainées dans des argiles molles. En fait, ce sont presque des vases, déposées sous leur poids propre, consolidées par surcharge, avec une teneur en eau importante mais constante ; il s’agit d’argiles toujours peu résistantes, avec des coûthés de l’ordre de 1 à 10 kPa (10 g/cm² à 100 g/cm²). L’ancrage étant chargé à 1/4, 1/2, 3/4 de la charge de rupture instantanée, on observe d’abord une consolidation puis un fluage. A 1/4 de la résistance instantanée, les déplacements cessent rapidement. Compte tenu de la très faible résistance de ces matériaux, on peut donc dire que le fluage d’un ancrage ne commence à devenir un danger qu’à partir d’un certain seuil et que ce seuil n’est jamais petit devant la résistance instantanée.

Une communication traite de technologie et une autre décrit une grande réalisation. Tomiolo (Italie) décrit le nouveau tirant de la Société Rodio. C’est un tirant à câble protégé par une feuille plastique avec scellement dans une plaque de pied en acier. L’injection se fait à partir d’un tube à manchettes central. L’avantage est la ré-injection toujours possible et le fait que le bulbe de scellement travaille en compression, donc sans risque de fissuration du mortier par traction. La transmission des contraintes dans le sol a été mise en évidence par des mesures extensométriques.

Fenoux (France) a présenté la réalisation d’une enceinte spectaculaire en parois de 30 m, soutenues par des tirants de 2 000 kN (200 t) pour la fouille de 17,5 m de l’usine électro-nucléaire de Blaye. Les tirants ont été tendus à la moitié de la charge de service, de façon à éviter de grands mouvements d’aller et retour lors de la mise en tension, puis pendant le dragage. Au cours de l’exécution de la fouille, les tirants et les parois très instrumentés, ont été suivis avec soin, ce qui est nécessaire pour un ouvrage de cette importance.

Mazurkiewicz et Najder (Pologne) ont étudié l’influence du mode de réalisation sur le comportement des ancrages dans des sols sableux avec intercalations d’argile. Ce qui est essentiel, c’est de bien enlever la bentonite. Pour le reste, si la réalisation est bien faite, les résultats sont semblables et les paramètres significatifs sont les conditions du sol autour du corps injecté. Les auteurs ont constaté que la durée qui sépare la prise du scellement de l’essai d’arrachage n’a pas d’influence sur la résistance.

La seule communication qui ait présenté un calcul du comportement sol-paroi-tirant d’ancrage en fonction de la raideur de l’appui et de la rigidité de la paroi est celle de Popescu et Ionescu (Roumanie). Ils utilisent un module de réaction non-linéaire et la comparaison avec des calculs par la méthode des éléments finis est très acceptable, tout en utilisant des calculs beaucoup moins coûteux. Il est probable que ce résultat est surtout valable pour les contraintes dans la paroi ou les tirants.

Enfin, la communication de da Costa Nunès et Dia (Brésil) est plus une leçon de Mécanique des Sol que l’exposé sur les tirants d’ancrage. Ils décrivent le remarquable sauvetage d’une paroi ayant présenté des déplacements atteignant jusqu’à 25 cm. La force portante de chaque tirant a été vérifiée et la force résiduelle a donné la poussée du sol, ce qui a permis ensuite les renforcements de tirants nécessaires.

En conclusion de ce rapide tour d’horizon, on peut dire que les essais systématiques en vrai grandeur constituent aujourd’hui encore la source la plus riche d’enseignement du comportement des tirants. Certes, de tels essais sont chers, et il faut profiter de l’occasion de grands travaux pour faire des essais en place. L’intérêt des réunions internationales est évidemment de montrer des réalisations ou des technologies nouvelles dans le vaste domaine du Génie Civil, mais, c’est surtout de mettre en commun des expériences coûteuses faites un peu partout au monde et c’est la raison pour laquelle je voudrais remercier ici, après nos hôtes Japonais, tous les auteurs de communications et tous ceux qui ont bien voulu nous tenir informés de leurs travaux.
GENERAL REPORT

by

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Special Session n° 4 dealt with the topic of anchoring bars or cables placed in bored holes, then bedded by poured or injected mortars. The Rapporteur deeply regrets that the topic was not more explicitly defined in the programme text. Because of this lack of precision, about ten proposals from numerous countries dealing with ground anchors by plates set in and embedded in the soil were sent in, but had to be refused. However, two papers on this subject were retained since they contributed unexpected light on the problem, also results which could be generalized.

Altogether 17 papers were submitted, that is almost the same number as for Special Session n° 15 at the I.S.S.M.F.E. congress in Mexico in 1969, the most recent international congress where this topic was examined. The comparison is interesting after an eight year interval and shows the extent of technical development.

In 1969 the problems which seemed most disturbing were the creep of soils and corrosion of steels. It is clear that at the present time these problems are, if not mastered, at least sufficiently known, to be no longer sources of anxiety. It is now possible to forecast acceptable deferred displacements and to ensure appropriate protection of steels to avoid accidents through corrosion.

Technical development is also obvious on the information level. In 1969 the Rapporteur could not present a valid bibliography through lack of documentation. In 1977 he still could not do so but this time because there were too many documents. To quote two examples, I would recall that the Institution of Civil Engineers organized in London in September 1974 a conference on the topic «Diaphragm walls and ground anchors» and in September 1976, that is two years after, a seminar on exactly the same subject. The second example is in France, where the Bureau Securitas edited a publication in 1972 entitled «Recommandations concernant la conception, le calcul, l'exécution et le contrôle des tirants d'ancrage» (recommendations for the design, computation, construction and inspection of anchorage ties) and five years later in 1977 a completely updated edition proved necessary to take into account the experience acquired during application of the first set of rules.

The papers on ground anchors sent in for the Tokyo Congress are not exhaustive, but reveal the present concern of those designing or constructing ties.

In spite of their importance, it will not be possible to present them all during the Special Session n° 4 because of lack of time, so I will sum them up here in a classified critical outline of the state-of-the-art.

I stated eight years ago that the development of ties and ground anchors had been achieved in the face of complete dearth of theoretical knowledge but backed up by an empirical rationale. This is still true to-day and numerous authors build small-scale models to clear up certain aspects of ground anchor behaviour. We have four papers which present results obtained from small-scale models. In the case of ties which are too short for example, if they are fixed in the Coulomb wedge, their effectiveness is reduced to zero. On the other hand, if the ties are too long, it is perhaps not worth lengthening them more. Between the two there is an optimal length which Habib, Luong, Tcheng and Auger (France) have endeavoured to determine using as a basis the idea that the failure surface has a geometrical shape imposed by the type of soil and dependent on external forces. They used models in sand with very poor similitude with a simple rigid-plastic model which is not sufficient to determine the deformations before the prototype's failure, but only the failure load, which however offers the possibility of defining the optimal length of the tie.

Bassett (G.-B.) also used simple similitude however a little more sophisticated with elastoplastic models in clay. His aim using quick tests is to try to obtain precise information on how localised widenings perform on ground anchors. Their distance must not be less than 2.9 diameters. The term N, is then 8.5 but because a void appears behind the widening, a value of $N_e = 6$ must be accepted for long-term testing. The flexibility of the anchor cable ensures progressive failure of each widening and slows down the load-displacement curve at the top. Naturally there still remains to define the creep safety and the acceptable load which are not included in the similitude.

Kananyan and three other Russian authors use a particular approach which they call «approximate model similitude». Geometrically similar models are built for ground anchors, flared anchors, for buried plates in real soil, and after each test the reduced load-displacement curve is traced (that is to say the loads divided by the limit load and the displacements divided by the limit displacement). This shows that this curve varies little with the model scale. The laws of the limit force and of the limit displacement are investigated according to the scale, and extrapolation carried out to reach real conditions, which gives a good approximation of the practical load-displacement curve.

Boon and Craig (G.-B.) have carried out tests on disc-shaped ground anchors placed in sand with perfect similitude achieved through centrifugal models. They have underlined a very general problem which is the difficulty of building small-scale models in sand with the risk of results well out of the realm of practice. They have shown that the best formulae give results which are high, at least double the real failure strength; ordinary models must therefore be handled with care.

Let us hope then that ground anchor ties, as we mean them in Special Session n° 4, will one day be examined by the centrifugal method. But it will be essential not to forget to simulate placing and its technology and especially the grouting pressures.
During the Mexico Congress, I pointed out all the interest of full-scale testing in the analysis of tie performance. This recommendation holds and we have received particularly enlightening papers.

Evangelista and Sapio (Italy) equip with electric strain gauges two vertical Dividag bars placed in a 22 cm 2 mortar paste bedding in a clayey soil. They draw up a non-linear analysis (with a hyperbolic behaviour law) and integrate the Mindlin formula as if the bedding were nothing but an ordinary pile but admitting that the bedding mortar breaks as soon as the concrete is put to a tensile load of 3 MPa (30 kg/cm²). In this way they obtain a relation between the force and the top displacement which is valid, at least at first. On the other hand, the failure strength is quite overrated, which shows that the side shear is not uniform. From the pull-out tests, it would not be higher than 1/3 or 1/4 of Cw.

Fujita, Ueda and Kusahuka (Japan) also show a non-linear design method for the tie pull-out curve. To estimate side friction and the length of the bulb of active pressure, they use a simple unidimensional elastoplastic model: the comparison with in situ tests makes it possible to evaluate the different constants of the formulae proposed. A comparison is made between these conditions and the number N of the standard penetration test which gives an evaluation to 20% of the load-displacement curve up to failure.

Bustamante, Delmas and Lacour (France) carry out tests to failure and then uncover to investigate the behaviour of ten ties in heavy clay to assess a critical load Tc (different from the limit load Ti of the bedding failure) whence they deduce the service tensile load or Ti. This is an illustration of the new French standard proposed by the Bureau Securitas Recommendations. This paper includes a wealth of results. For example, the creep under critical load Ti reached 1 mm, under the allowable load Tc it was only 0.4 mm, the Tc/Ti ratio is higher than 1.5; the group effect is felt when the ties are fixed at 2 m intervals; the strength of the ties is proportional, not to the injection pressure, but to the volume injected provided the losses are limited, hence the importance of fixing grouting in several runs. In these conditions, the mobilized shear may reach three times Cw.

Ostermayer and Scheele (GFR) have undertaken full-scale investigation in the laboratory where they test and then uncover for observation thirty ties bedded under pressure, under 5 m of gravelly sand or sandy gravel at different rates of compactness and with different fixing lengths. They then state this in relation to the soil pressure test. Before rupture, they note viscous effects on the granular material. As the load increases, the stresses develop and are redistributed towards the base of the bedding. This redistribution becomes obvious at a certain threshold and that this threshold is never low in the face of instantaneous strength.

Whilst on the subject, I should like to point out that Wernick (GFR) in his paper considers that in the case of sand, high shear corresponding to side friction is due to the dilatancy effect in the narrow strip around the bedding where sliding occurs. He emphasizes the need for reexamining the hypothesis of coaxiality of stress and increasing deformation tensors in the light of dilatancy.

Littlejohn, Bruce and Deppner (G.-B.) analyse 57 high bearing capacity ties in a stratified sandstone, this is therefore an example of rock based ties. This is a marginal subject for soil mechanics but the results are worthy of note for numerous reasons. The weight of a cone with top half angle equal to 45° from the bedding foot is not sufficient to produce the forces recorded. Failure is essentially dictated by vertical and horizontal joints. Littlejohn proposes the formula P (kN) = 600 d (cm) - 6, which is an improvement on the formula which occurs at the grout/tendon interface, although this surface is larger than that of the grout/rock interface. It is essential then not to put too many tendons in the same bedding and advisable not to exceed a steel section over 10% the surface of the bedding straight cross section.

All the tests quoted offer the possibility of forecasting the anchorage strength by analogy.

Kramer (GFR) goes even further with very original work. Abandoning all hope of calculating the strength of ground anchors, he uses the results of 150 tests, half in clay, half in sand, to draw up equations and correlations. He sets up a formula where the maximum variation with tests will not exceed 60 kN. As with all statistics it is preferable to stay in a practical range, but once a large number of tests have been undertaken, the results become interesting.

Finally to complete the subject of in situ tests, I must mention the paper by Yen and Young (USA) who describe very sophisticated tests with marine anchors embedded five diameters which is an improvement for an anchor, but rather shallow for civil engineering anchoring. This text is therefore quite beside the point, but it is extremely interesting since the authors show the role of suction behind the anchorage and that of pore pressure in front of the anchorage which reaches 50% of the strength at the beginning of loading for pull-out tests in non-drained conditions in soft clay. In fact they are almost muds, deposited under their own weight, consolidated by overloading with a high but constant water content. These are low-strength clays, with cohesion rates around 1 to 10 kPa (10 g/cm² to 100 g/cm²). With the anchor loaded to 1/4, 1/2, 3/4 instantaneous failure load, the result is first consolidation then creep. At a quarter of the instantaneous strength the displacements cease rapidly. Taking into account these materials low strength, we can state that ground anchor creep becomes dangerous only from a certain threshold and that this threshold is never low in the face of instantaneous strength.

Two other papers deal, one with the technology and the other with a description of a large undertaking. Tomiolo (Italy) describes the new tie at the Redio Company. It is a plastic-coated cable tie bedded in a steel foot plate. The injection was carried out with a central sleeved tube. The advantage is that reinjection is always possible and that the bedding bulb performs under compression, therefore there is no risk of cracking of mortar due to tensile stress. The transmission of stresses in the soil was revealed by strain gauge measurement.

Fenoux (France) presented a paper on the construction of a spectacular excavation with 30 m walls, supported by 2 000 kN (200 t) ties for the 17.5 m excavation at the Blaye nuclear power station. The ties were tensioned to half the service load, so as to avoid extensive forward and backward movements when tensioning, then during dredging. During excavation the behaviour of the ties and the walls fully equipped with measurement instruments was followed carefully, as is necessary for engineering works of this importance.
Mazurkiewicz and Najder (Poland) studied the influence of the construction method on the behaviour of anchors grounded in sand with insertions of clay. It is essential to carefully remove the bentonite. Apart from this, if the construction is properly carried out, the results are similar and the significant parameters are the soil conditions around the injected element. The authors have found that the lapse of time between the bedding setting and the pull-out test has no influence on the strength.

The only paper which presents computation of the soil-wall-anchorage tie behaviour in relation to the stiffness of the support and rigidity of the wall is that by Popescu and Ionescu (Romania). They use a non-linear reaction modulus and the comparison with the finite element calculation method is very acceptable, with the added advantage of being less costly. It is probable that this result is especially valid for stresses in the wall or the ties.

Finally the paper by Da Costa Nunès and Dia (Brasil) is more a lesson in soil mechanics than a paper on anchorage ties. They describe the remarkable salvage of a wall where displacements up to 25 cm had been recorded. The bearing capacity of each tie was checked and the residual force revealed the earth pressure, which made it possible to calculate the tie reinforcements necessary.

To conclude this quick survey, it seems possible to say that systematic full-scale testing still represents to-day the finest source of information on the behaviour of ties. Such tests are indeed costly and it is advisable to take advantage of opportunities of extensive construction to undertake in situ tests. The importance of international meetings is obviously to give information on new achievements and technology in the wide field of civil engineering but even more, such meetings help us to pool experience acquired throughout the world. This is why I should like to thank first our Japanese hosts and then all the authors of papers and those who have so kindly submitted information on their work.
UNDERREAMED GROUND ANCHORS
Ancrage dans le sol avec élargissement

by
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SOMMAIRE
Les ancrages dans le sol acquièrent une importance grandissante pour les appuis de murs de soutènement, aussi bien en terrains cohérents qu'en terrains non-cohérents. Pour les sols cohérents, le rendement de la portance d'un ancrage augmente avec la construction d'élargissements sur le fût. Les formules empiriques qui permettent de calculer cette charge utile proviennent de la théorie des pieux. L'étude examine, au moyen d'expériences en laboratoire, les caractéristiques suivantes des ancrages à élargissements de diamètre : influence des distances entre les élargissements, influence du nombre de ceux-ci et relation entre rigidité du sol et rigidité du câble de l'ancrage. L'observation des mécanismes de rupture montre que les éléments individuels de la formule empirique sont justifiés, ce qui permet de suggérer des valeurs pour les dimensionner.

SUMMARY
Ground anchors are becoming increasingly important for supporting retaining walls in both cohesive and non-cohesive soils. In cohesive soils the efficiency of an anchor's carrying capacity is improved by forming underreams on the shaft. Empirical formulae for calculating their load capacity have been developed from piling philosophy. The paper examines by controlled laboratory experiments the following features of underreamed anchors: The influence of the spacing of underreams, the influence of the number of underreams and the relationship between the soil stiffness and the anchor tendon stiffness. The observed failure mechanisms show that the format of the empirical formula is justified and values for design constants are suggested.

INTRODUCTION
The rapid increase in the economic importance of holding back permanent retaining wall structures by the use of anchor ties was indicated by Ostermayer (1974). The exponential increase suggested has continued unabated and in the author's personal experience appears to be coupled with a continuous demand for increased load capacity. Practical anchoring techniques fall into two distinct categories,

a) Pressure grouted or regrouted shafts, relying on the adhesion bond between the face of a grout mortar body and the soil (most commonly used in cohesionless soils and rock), or

b) Open drilled shafts in which the anchorage zone is deliberately expanded in diameter to form a series of bells or underreams (most commonly used in self supporting soils from fine cohesive clays to moderately hard rocks).

Although a considerable body of literature has now been collected, Littlejohn (1975), these predominantly concern either practical examples, wall-anchor interaction studies or fundamental laboratory tests of anchor plate systems in dry sand. Little fundamental study, with the exception of Ostermayer (1974), appears to be available to justify the empirical design formula (Littlejohn 1970a 1970b) recommended for the real construction methods.

The author has been working on both groups. But this paper will present only data obtained from a controlled laboratory examination of the behaviour of multi-underreamed anchors in a very low permeability, saturated, uniform, cohesive material.

THE UNDERREAMED ANCHOR PRINCIPLE

Straight shaft, pressure grouted anchors have been used in cohesive materials but boring the hole invariably results in the remoulding of a thin layer of soil adjacent to the final anchor member. The considerable, consequent reduction in strength can rarely be compensated for even by pressure grouting. Underreams cut out into undisturbed soil at intervals along the shaft induce a cylindrical shear failure although intact soil (fig. 1). Various patented cutting techniques are employed in the U.K. the blade cutter of Universal Anchorage Limited and the expanding brush of the Cementation Company are typical instances.
Dr. Littlejohn (1970a, 1970b) presented an empirical formula (1) for estimating the ultimate capacity of this type of anchor based quite logically on piling philosophy

\[ W_{\text{ult}} = N_e \cdot \left(D^2 - d^2\right) \frac{\pi}{4} \cdot c_u + \pi D \cdot l_u \cdot c_u + f_u \cdot \pi d \cdot l_u \cdot c_u \]  

end bearing + capacity of underream length + capacity of shaft length.

The author, Bassett (1970) in his discussion to this paper and in Bassett (1971) had derived a similar formula (2) based on the full scale testing of some 500 production clay anchors but the author included a reduction coefficient \( f_u \) to be applied to the capacity of the underream length and questioned Littlejohn's \( N_e \) value (see Littlejohn's reply (1970) c).

\[ W_{\text{ult}} = N_e \cdot \left(D^2 - d^2\right) \frac{\pi}{4} \cdot c_u + f_u \cdot \pi D \cdot l_u \cdot c_u + f_u \cdot \pi d \cdot l_u \cdot c_u \]  

Under the author's direction Potts (1973) set out to investigate both the fundamental components of this empirical equation and possible alternative analytical approaches. His work on the following is presented here:

(i) The examination of the behaviour of a single underream in order to measure the Terzaghi bearing capacity factor \( N_e \) and to assess the influence of the cavity which forms below an underream on rapid loading.

(ii) The examination of the mechanism of failure to justify the format of the empirical equation (2).

(iii) The examination of the influence of varying the spacing of underream bells.

(iv) The effect on the load capacity of increasing the number of underreams and to relate this effect to the ratio between the stiffness of the anchor tendon system and the equivalent response stiffness of the soil deformation mechanism.

**EQUIPMENT AND TEST PROCEDURE**

All the tests described were performed on a saturated, remoulded, reconsolidated, London clay with the following specification:

- Water content: 43%
- LL: 86%
- PL: 23%
- PI: 63%
- Liquidity Index: 0.35
- Undrained triaxial shear strength: 40 KN/m².

This is in good agreement with Skempton and Nor- they (1953). The data indicated an undrained \( E \) (at 1/2% strain) = 15 \times 10^6 N/m².

The anchor was formed of 30 mm diameter, stiff, aluminium discs some of which incorporated a strain gauged load transducer. These discs represented the underream bells and were connected by various lengths of rod, hollow hypodermic tube, or stiff springs to represent the tendon system. The load carrying zone formed by the discs was deeply embedded (depth/ diameter > 15) in the clay contained in a 380 mm dia. \times 900 mm high cylindrical container (fig. 2). The lateral pressure in the soil was initially fixed and monitored by instrumented bolts.

All tests were performed at a constant pull out rate of 1.7 \times 10^{-2} mm/s. For saturated, remoulded London clay this is effectively rapid and undrained.
THE SINGLE UNDERREAM

Two initial tests were performed on single discs the first with a hollow tube tendon venting the area below the disc to atmosphere, the second with a solid stem. The results are shown in figure 3 as curve A and curve B respectively.

The vented disc, A, reaches its initial failure in end bearing at approximately 1 mm displacement. Curve B shows a similar basic format the pressure difference between Curve A and B representing the suction on the underside of the disc is shown in figure 4.

where $P_s$ is the pressure required to expand the cavity and $\frac{c}{a}$ is the ratio of the cavity volume to the volume of the plastic zone.

\[ i.e. \quad \frac{c}{a} = \left( \frac{E}{1 - \nu} \right) \frac{2\,c_v}{C} \]  

(iii)

for $\nu = 0.5$ (undrained), $c_v = 40$ kN/m$^3$, $E = 15 \times 10^5$ N/m$^2$

$P_s = 188$ kN/m$^2$

A suction pressure of one atmosphere is developed after only 0.15 mm displacement and reaches a peak in excess of this at 0.25 mm there after progressively decreasing. This is presumably due to evaporation at the free clay surface and a reduction in suction as water vapour fills the cavity. It is significant that a vacuum cannot be maintained even at this high rate of strain.

Potts (1973) adopted the expanding plastic cavity approach to provide a theoretical peak value for the ultimate load capacity of this single underream. He adopted from the work of Bishop, Hill and Mott (1945) the equation:

\[ P_s = \frac{2}{3} c_v \left( 1 - 3 \log_2 \frac{c}{a} \right) \]  

(i)

and over the area of the underream disc for a half cavity is equivalent to a force of: $137$ N.

This value is drawn on figure 3 and is the theoretical estimate for the maximum value of curve A. The agreement is seen to be reasonably good. Swain (1976) has developed a similar theoretical argument but used a more extended approach to give the complete load-displacement relationship up to the initial failure.

The Conventional Terzaghi bearing capacity and the piling approach to this problem suggests a value of $N_c = 9$. (This value of $N_c$ is used by Littlejohn 1970a 1970b). The experimental value for Curve A is $N_c = 5.4$ and for the total load including the suction component in Curve B is $N_c = 8.6$. 

![Fig. 3. — Load-displacement for a single underream.](image)

![Fig. 4. — Suction-displacement below a single underream.](image)
VERTIFICATION OF THE MECHANISM POSTULATE IN THE EMPIRICAL FORMULAE AND THE INFLUENCE OF UNDERREAM SPACING:

As all the subsequent test series described in this paper allowed no venting below the bottom underream the higher value of Ne = 8.4 was adopted to include the suction component.

To check the failure mechanism an underream spacing of 1.5 D, consistent with practice, was adopted. This model was tested using a split sample, the clay surface being marked with a fine horizontal grid. During the test this grid was examined and typical photographs are shown in figure 5.

The ruptures seemed to justify the empirical approach, assuming two major components (1) end bearing on the top underream and (11) a waisted plug like component for the main anchorage length. The pictures did however show a distinct indication of an end bearing passive cone above all the underreams before the plug failure was fully formed. This observation suggested that a transfer from the plug failure mechanism to a number of separate end bearing failures would almost certainly occur when the load capacity between two underreams reached the value of an end bearing failure, perhaps even earlier.

i.e. if the distance between two underreams L is thought of as a dimensionless ratio \( \times \) diameter of underream (D)

\[ L = \frac{L}{D} \times D \]

then for equal capacity of a plug failure and an end bearing failure

\[ f_u \cdot c_u \cdot \pi \cdot D \left( \frac{L}{D} \right) _{\text{crit}} \cdot D = N_e \cdot c_u \cdot (D^2 - d_s^2) \cdot \frac{\pi}{4} \]

or \[ \left( \frac{L}{D} \right) _{\text{crit}} = \frac{N_e}{4f_u} \cdot \left( \frac{D^2 - d_s^2}{D^2} \right) \]

as \( d_s \) is small compared to D then

\[ L_{\text{crit}} = \frac{N_e}{4f_u} \]

Section 5 of this paper will indicate the determination of \( f_u = 0.63 \) and for this value and \( N_e = 8.5 \)

The critical spacing ratio is equal to 5.4. Using the fully vented cavity value \( \left( \frac{L}{D} \right) _{\text{crit}} \) reduces to 2.1.

A series of unvented, rapid loading tests were carried out using 3 underream discs at spacings of 1.5 D: 2 D: 2.25 D: 3 D and 4 D. The results are shown in figure 6.
The theoretical increase in ultimate load up to \( \frac{L}{D} = 3.4 \) is also shown. The predicted form is closely adhered to but deviation to an end bearing failure is earlier than expected i.e. at approx. 2.9 D. This was in fact expected as the full Ne value of 8.5 should not apply to the 2nd and 3rd underreams in end bearing because their capacities will obviously be reduced by the suction zone forming below the disc above. At the experimental \( \left( \frac{L}{D} \right)_{\text{cr}} \) of 2.9 this suggests that if \( N_e = 8.5 \) applies to the top underream plate a value of only 6.6 is possible on the two lower plates. Separate measurements of the loads on each underream at a spacing of 3 D confirm this figure 7.

The peak capacity of a \( L = 3 \times D \) set up was not achieved until a displacement of 4 mm had occurred — whereas study of all Potts (1973) data showed that a 1.5 D spacing achieved peak after approximately a 2 mm displacement.

The maximum effective spacing appeared to be 2.9 D but in view of the earlier comments on loss of capacity, \( N_e \) reducing to 5.4, with time as seepage relieves the vacuum and in order to limit displacements under load, a practical spacing of \( 2 - 2.25 \) D should be adopted. This conclusion is in very close agreement with Mohan, et al (1969) on multi-underream piles.

THE MAGNITUDE OF THE FRICTION \( f_u \) & INFLUENCE OF TENDON STIFFNESS

In view of the data in section 4 the dual mechanism was certainly mobilised at \( \frac{L}{D} \) ratios of less than 2. A further series of tests were therefore performed at the \( \frac{L}{D} \) ratio of 1.5 with 2, 4, 6, 8 and 10 underream plates connected with

(i) solid rods, and (ii) stiff coil springs.

The peak results of the solid rod series are shown in figure 8. The dotted line represents the load capacity given by Littlejohn’s equation (1), full \( c_u \) mobilised on the whole underream plug i.e. \( f_u \) in equation (2) = 1.

The experimental data indicates a reduction to an \( f_u \) of between 0.63 and 0.79.

This result appears to justify the authors original postulation in (1970) but it does not identify the cause. The waisted shape, i.e. the reduced D, of the failure plugs seen in figure 5b and the influence of the underream tips seem possible causes but detailed work by Swain (1976) on the strain patterns round underreams suggests that the rapid migration of excess pore pressure above the underreams to the plug shearing zone causes noticeable softening, even at rapid loading rates.

Figure 8 does however indicate a progressive and virtually linear rise in ultimate capacity with each
Additional underream. If however additional data showing the load capacity at various displacements is added to figure 9; then at an overall safety factor of 2 it is apparent that very little is gained by the use of more than 6 underreams.

It was suspected that the significance of displacement would be further accentuated in very stiff soils. In the absence of equipment capable of testing marl or shale the soil properties were kept consistent and this feature was examined by reducing the stiffness of the interconnecting tendon member.

The test series described above was repeated, $L/D = 1.5$, with stiff coil springs in place of the solid rods (*).

(*) The stiffness ratio of $K_{soil} : K_{tendon}$ for the rods was 1 : 20 where $K_{soil}$ is the mean load-displacement slope for the soil at 1/2 mm displacement and $K_{tendon} = $ the elastic load-displacement value.
An idealised summation of the behaviour of a 3 underream anchor with weak spring connection is shown in figure 10. (Potts 1973).

The total load curve shows three distinct changes in slope as each successive underream fails. In figure 11 the concept is extended to 2, 3 and 4 underream groups.

The corresponding experimental data is given in figure 12. The correct trend is observed, little of significance resulting from the use of more than two underreams. The displacements in this series are some ten times larger than those previously measured. This is due to the unrealistically soft spring tendon members, the stiffness ratio is $K_{soil} : K_{tendon} = 7 : 1 (*)$.

In real practice the stiffness ratio of say London Clay to cable anchor tendon is thought to be between 1 : 5 and 1 : 3 and of Keuper Marl to cable anchor tendon nearer 2 : 1 or 1 : 1. So the indications from the soft spring tests may in fact be more representative than those of the solid bar series.

CONCLUSIONS

The tests have indicated the validity of the composite empirical equation (2) provided that the $\frac{L}{D}$ spacing of the underream bells is maintained definitely below 2.9 and if displacements are a limiting factor it is recommended that a value of $\frac{L}{D}$ of approximately 2 should be adopted. Its use also implies that the stiffness ratio between the soil and the anchor tendon allows the whole system of underreams to be developing the failure mechanism when the initial end bearing failure condition is reached by the top underream.

The end bearing factor $N_e$ appears to be approximately 8.5 for rapid short duration loads but it is recommended that a value reduced to 6 should be adopted for permanently loaded situations.

The research described above continues to be developed to broaden its scope. Also several real problems still remain unanswered. A particularly important one is the effects of cyclic loading and very long term creep «settlement» on the carrying capacity and deformations and these are now being investigated.

REFERENCES


MODEL GROUND ANCHORS UNDER GRAVITATIONAL AND CENTRIFUGAL ACCELERATIONS

Modèles réduits d'ancrage au sol en gravité normale ou en centrifugeuse

by

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SOMMAIRE

On traite de deux séries d'essais dans lesquels des plaques circulaires d'ancrage de 25 mm de diamètre, enfouies dans des couches de sable sec à l'intérieur d'un conteneur de 0.770 x 0.770 x 0.500 m, ont été testées sous un régime de contraintes correspondant aux ancrages in situ.

Les niveaux des contraintes ont été obtenus, soit par un vide à l'intérieur de la maquette dont la surface supérieure était scellée par une membrane en caoutchouc, soit par des forces centrifuges.

On présente des courbes efforts-déplacements pour des plaques d'ancrage verticales, tirées vers la paroi du conteneur, ainsi que le changement de la distribution de la contrainte normale sur ces parois pour des valeurs différentes de la position de l'ancrage.

On trouve que la flexibilité du conteneur est critique pour le résultat des essais sous vide. Par contre, les modèles en centrifugeuse où les mouvements latéraux sont bloqués, donnent des résultats plus réalistes.

En utilisant cette technique, on a mesuré des variations importantes de la force d'arrachement, au fur et à mesure que l'ancrage passe d'une position horizontale à une position verticale. Les forces d'arrachement sont inférieures à celles présentées normalement par d'autres auteurs.

SUMMARY

Two series of model experiments are reported in which circular plate anchors, 25 mm diameter, buried in beds of dense dry sand within a container 0.770 x 0.770 x 0.500 m, have been tested at effective stress levels appropriate to field anchors.

These stress levels have been obtained by evacuating the void space within the container which was sealed at the surface by a flexible membrane and exposed to external atmospheric pressure or by the action of inertial accelerations generated within a centrifuge.

Load-deflection data are presented for vertical anchor plates pulled towards the boundaries of the container together with changes in normal pressure distribution at these boundaries for a range of initial anchor positions.

Flexibility of the container is shown to have a critical effect on the results from the evacuated models.

The centrifuge models in which boundary deflection is restrained are shown to give more realistic results and using this technique significant changes in anchor pull-out load have been measured for variations in anchor inclination from horizontal to vertical. The pull-out loads have generally been lower than those predicted by other researchers.

INTRODUCTION

The ultimate resistance to tensile loading of buried plate anchors in sand has been studied by many authors and of the design methods proposed, that developed by Meyerhof and Adams (1968) and Meyerhof (1973) has the widest use, being applicable to anchors of any shape at any inclination from horizontal to vertical.

Most of the data which have been used to justify this or any other design method of general application have been provided by small scale laboratory models. However, scale effects in model work on cohesionless soils give rise to uncertainty in the extrapolation of results to field situations. The reduction of the mobilised angle of friction $\phi$ under increasing mean principal stress levels is well known — typically for a dense sand a reduction of the order of 5 degrees in $\phi$ per log cycle of effective stress. Associated with this reduction is a transition from brittle to plastic failure in soil elements and a trend towards increased displacements at failure, relative to model size.

De Beer (1970), Graham (1974) and Aboshi (1975) among others have reported a change from overall shear failure in small models of foundations under compressive loading, to punching shear at larger sizes and higher stresses. Mikasa and Takada (1973) have performed centrifugal model tests on both shallow...
and deep foundations under compressive loads and have presented visual evidence of the difference in failure mode of the same model between tests at unit gravity and others at 60 g. Krebs Ovesen (1975) has summarised the findings of his own centrifuge experiments along with those of Mikasa and Takada and of Cherkasov et al. (1970) and has shown that the variation in model failure loads associated with the different modes of failure at different accelerations is consistent with substantial reductions in $\sigma$ with stress level increases. The results of centrifuge tests by Yamaguchi et al. (1976) on surface and shallow foundations show similar effects.

The implication is that more realistic design information for tension carrying foundations may be obtained from model experiments carried out at or close to field stress levels. In this context the centrifuge model is an attractive alternative to full scale experimentation.

The S.E.L. centrifuge has a capacity to accelerate up to 2,000 kg of soil to 140 g, i.e. an acceleration factor $N = 140$, and has been used well within this limit in a programme of testing model anchors in sand at inclinations from horizontal to vertical. A bolted aluminium container 0.770 m square $\times$ 0.500 m deep, with 16 mm walls was used for most of the experiments. The container was filled with air dry Mersey River sand deposited from a roller spreader. The range of porosities measured after deposition was 0.355 — 0.359 corresponding to a relative density of approximately 95%. The anchor plates were 25 mm diameter, 8 mm thick, attached to steel rods 4.75 mm diameter and were placed in every case at a depth 300 mm, below the upper, level soil surface at an inclination which varied from test to test but always coaxial with the load to be applied. Full details of the experimental techniques are given by Boon (1975).

**TESTS AT UNIT GRAVITY**

In order to carry out preliminary experiments at field stress levels without using the centrifuge the upper sand surface was covered by a thin rubber membrane and the void space of the soil mass was subjected to a partial vacuum.

Horizontal anchors were pulled by a pneumatic jack towards a wall of the container instrumented with total stress transducers. The anchor itself passed through a lubricated sleeving at the wall and the jack reaction was taken by a mounting frame of steel sections. Fig. 1 shows load-deflection results for a number of plate anchors initially located at different distances from the wall with a suction of 66 kN/m$^2$ applied in each case. The anchor shaft load in the absence of a plate was found to be consistently less than 0.08 kN. Defining failure arbitrarily at a deflection of 30% of the plate diameter the failure load is seen, fig. 2, to be sensibly independent of location relative to the wall in the range 6 - 12 plate diameters.

In a further series of tests on inclined anchors the applied suction was lowered to 57 kN/m$^2$ and the pneumatic jack, suitably mounted on the reaction frame, pulled the anchor rod either through a sleeve in the metal wall or through the rubber membrane. Results, fig. 3, show that the failure load increased as the inclination from the horizontal increased. The test arrangement is shown in fig. 4.

![Fig. 1](image1.png)

![Fig. 2](image2.png)

![Fig. 3](image3.png)
Under the action of the applied suction the box walls deflected inwards by up to 0.4 mm and the total stresses measured on the boundaries indicated initial $K$ values ($K = \sigma_H/\sigma_v$) of approximately unity. The results of the pull-out tests are consistent with an initial hydrostatic pressure distribution yielding a bed of essentially uniform strength at all depths and the higher loads on vertical anchors are associated with increased gravity effects in this direction.

TESTS UNDER INCREASED ACCELERATIONS

The SEL centrifuge has a rigid rotor 7.32 m in diameter which moves in a horizontal plane, generating a radial acceleration. In order to keep the free soil surface perpendicular to the acceleration, as in the field, this surface must be in a vertical plane and to achieve this with minimal disturbance it is necessary to fit a thin membrane over the surface and to apply a small suction to the sand mass until sufficient radial acceleration has been generated to hold the soil in place.

The model box was packed into a rigid container and with all four walls fully supported internally the suction was applied before the whole assembly was lowered into the test position, fig. 5. The suction was released when accelerations reached 5 g and the anchors tested under a steady accelerations of 26 g at the radius of the anchor plate, 2.94 m. Changes in wall pressure over the acceleration increment after
releasing the suction were consistent with $K = 0.25$. At 26 g the models simulated field anchors 0.65 m in diameter at depths of 7.4 m.

Failure loads for a series of 'horizontal' anchors were again sensibly independent of initial position within the range tested, fig. 6, but the changes in measured wall pressure were greatly influenced by this variable, fig. 7. Detailed study of a single test with the anchor plate initially 8 diameters from the wall shows the development of pressure changes across and
down the wall, fig. 8. The changes are symmetrical across the wall at the anchor level but are substantially higher below the anchor than above.

Limited results for tests on inclined anchors also carried out with $N = 26$, fig. 9, show a significant reduction in pull-out loads at inclinations of 45° and 90° from those measured on horizontal anchors. This figure also shows the failure loads predicted by the method of Meyerhof (1973) for a mean unit weight under centrifugal acceleration of 415 kN/m³ and a unique angle of friction of 41° measured in triaxial compression, for Mersey River sand at an appropriate mean stress level. In every case the prediction exceeds the observed failure load and while the difference can be reduced by considering anchor loads at displacements greater than the plate diameter a considerable discrepancy remains.

The prediction, developed from the results of Meyerhof and Adams (1968) assumes implicitly a limit to the zone of soil involved in the failure, based on observation of model tests at low stresses under unit gravity. However, this zone may be significantly reduced in size at higher stress levels with a subsequent lowering of the failure loads. No direct observation of the size of the 3-D zone of soil movement around circular plate anchors in the centrifuge has been possible, but observations on identical strip anchor models, pulled vertically in a plane strain configuration under different accelerations confirm that this zone is considerably reduced in the same manner as observed by Mikasa and Takada (1973) for the compressive loading situation.
In order to assess directly the effect of changes in stress level on the failure of an anchor a 25 mm bar 152 mm long was pulled vertically, in a plane strain configuration in a box also 152 mm wide, from a depth of 340 mm towards the surface of a bed of River Mersey sand at the same initial density as used previously. The test was repeated at a number of different accelerations in the range 1 - 54 g. For ease of comparison fig. 10 shows the measured anchor loads divided by the acceleration factor as a function of the anchor deflection. Defining failure at a deflection of 10% of the width in these tests the dependence of the parameter \( N_{qu} \) and associated values of \( \varnothing \), given by the analysis of Meyerhof (1973), on acceleration factor \( N \) is clear, fig. 11 and table 1.

**TABLE 1**

<table>
<thead>
<tr>
<th>Acceleration Factor</th>
<th>Mean ( \sigma_v ) (kN/m(^2))</th>
<th>( N_{qu} ) (Observed)</th>
<th>( \varnothing ) (Deduced Meyerhof)</th>
<th>( \varnothing ) (Tong)</th>
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<td>26.5</td>
<td>43.5</td>
</tr>
<tr>
<td>54</td>
<td>157</td>
<td>3.3</td>
<td>23.0</td>
<td>42.5</td>
</tr>
</tbody>
</table>

Table I shows the mean vertical stress in the sand in each test before loading the anchors and the values of \( \varnothing \) indicated for this material at the appropriate relative density from the plane strain element tests of Tong (1970) with minor principal stress of this magnitude. Allowing for this variation in friction angle the range of \( N_{qu} \) values predicted would be only one third of that observed. The difference appears to arise from the fact that the factors given by Meyerhof are based upon failure modes observed only in small model tests at unit gravity, i.e. low stresses. As Mikasa and Takada (1973) have shown, these will exaggerate the volumes of soil in the failure zone associated with geometrically similar models at higher accelerations and stress levels.

**CONCLUSIONS**

The design of field anchors in sand, based on data obtained from laboratory model experiments at low stress levels will be in error on account of the difference in stress level between the two situations. Adjustment of the design for changes in the angle of friction assessed in element tests at appropriate stress levels may partly eliminate the error but this will not necessarily take account of differences in failure mode if the design is still based on model test results with correct friction angles but incorrect stress levels. The centrifuge modelling technique provides a means of obtaining design data from models subjected to the failure modes appropriate to the correct stress levels.

**REFERENCES**


COMPORTEMENT DES TIRANTS PRÉCONTRAINTS DANS UNE ARGILE PLASTIQUE

Behaviour of prestressed anchors in plastic clay

par

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SOMMAIRE

Les possibilités remarquables qu'offrent les tirants dans les domaines les plus divers du génie civil, conduisent à envisager leur mise en œuvre dans les sols argileux. Toutefois, le manque d'études systématiques relatives à leur tenue à long terme dans ces matériaux, oblige encore les constructeurs à ne pas les utiliser comme organes définitifs.

A partir d'essais en vraie grandeur effectués sur dix tirants scellés dans une argile plastique, les auteurs se sont proposés d'étudier les différents aspects de leur comportement.

Le programme de recherche comportait également le déterrement des tirants.

Les résultats obtenus à ce jour apportent un début de réponse aux problèmes suivants:

1) Validité de la méthode de détermination de la traction de service $T_A$, basée sur la recherche de la traction critique $T_C$.

2) Importance et dispersion des tractions limites $T_L$.

3) Comportement du tirant au sein d'un groupe.

4) Incidence des paramètres de l'injection sur la tenue du bulbe de scellement et la formation de ce dernier.

5) Mécanisme de transmission de l'effort de traction le long du scellement.

6) Corrélation entre les caractéristiques géotechniques de l'argile et la résistance du tirant.

Le programme d'étude comportait le déterrement des tirants et notamment de leur partie scellée. Comme il n'a pas été procédé à l'excavation de certains tirants toujours soumis à des essais de longue durée, on ne présentera qu'une partie des résultats obtenus à ce jour.

SUMMARY

The outstanding performances offered by anchors in the various fields of Civil Engineering leads to an extension of application in clayey soils.

However, the lack of systematic study concerning the long term behaviour in this type of soil is a limitation for the designer to use them as permanent anchorages.

From full scale tests carried out on 10 anchors set in a plastic clay, the authors of this paper study the different aspects of the carrying behaviour of the anchors.

The research program included also the unearthing of the anchors.

Results obtained bring partial answers to the following topics:

- scattering of the values of ultimate and working loads;
- validity of the testing procedure based on the determination of a critical load;
- behaviour of a single anchor as part of a group of anchors;
- stress distribution along the grouted length;
- influence of the main grouting factors on the construction of the grouted part.

1. INTRODUCTION
2. RECONNAISSANCE ET CARACTERISTIQUES GEOTECHNIQUES DE L'ARGILE

Préalablement à la mise en œuvre des tirants d’essais et dans le but de définir les propriétés de l’argile, on a exécuté :
- un sondage carotté avec prélèvement d’échantillons intacts ;
- un essai de pénétration statique (Ø 45 mm) ;
- trois sondages pressiométriques avec essais tous les mètres ;
- la gamme complète d’essais d’identification et de cisaillement sur échantillons intacts.

Au droit du plot d’essai, et sur toute la hauteur des bulbes de scellement, on rencontre l’argile des Flandres (Ypresien). Le tableau I en synthétise les caractéristiques géotechniques essentielles déterminées en laboratoire.

Les essais en place, qui confirment d’ailleurs l’homogénéité de l’argile, ont respectivement donné pour le même niveau :
a) Résistance de pointe mesurée au pénétromètre statique : \( R_p = 1 \text{ à } 1.5 \text{ MPa} \)
b) Pression limite : \( p_Y = 0.5 \text{ à } 0.7 \text{ MPa} \)
Module pressiométrique : \( E = 4 \text{ à } 6 \text{ MPa} \).

| Tableau 1 |
|---|---|---|
| Prof. (m) | Essais identification | Essais de cisaillement | Compressibilité |
| | \( \gamma \) (kN/m³) | \( W \) (%) | \( W_L \) (%) | \( I_p \) | \( C'_{\text{su}} \) (kPa) | \( C' \) (kPa) | \( \phi' \) (°) | Etat de Consolidation |
| 5 | 185 | 30 | 80 | 50 | 60 | à | à | Argile surconsolidée |
| à | à | à | à | | | 38 | 12 |
| 7 | 195 | 40 | 90 | 60 | 80 | | | |

3. IMPLANTATION, MISE EN ŒUVRE ET EQUIPEMENT DES TIRANTS

Les essais ont porté sur un total de dix tirants du type TMD (brevet Sif-Bachy) [1], répartis sur trois plots (fig. 1) :
- plot A (tirants A₁, A₂, A₃, A₄) ;
- plot B (tirants B₁, B₂, B₃) ;
- plot C (tirants C₁, C₂, C₃) ;

Le tableau 2 résume les caractéristiques de chaque tirant ainsi que ses paramètres d’injection.

Certains tirants ont été équipés de tubes-logements (Ø 27/30 mm) permettant de recevoir un extensomètre amovible [2] ainsi que de deux tubes (Ø 10/12 mm) logeant chacun un fil d’invar reliant les extrémités du bulbe aux dispositifs de mesure en surface.
<table>
<thead>
<tr>
<th>TIRANT</th>
<th>Longueur scellée $L_s$ (m)</th>
<th>Longueur Libre ($L_L$) (m)</th>
<th>Section armature</th>
<th>Pression moyenne (MPa)</th>
<th>Volume de coulis $C/E$ (m$^3$)</th>
<th>Dosage du coulis (CIE)</th>
<th>Nombre de phases</th>
<th>Coloration du coulis de scellement ($^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A$_1$</td>
<td>6</td>
<td>6.20</td>
<td></td>
<td>0.5</td>
<td>0.955</td>
<td>2.27</td>
<td>3</td>
<td>Aucune</td>
</tr>
<tr>
<td>A$_2$</td>
<td>0.5</td>
<td>0.958</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A$_3$</td>
<td>0.5</td>
<td>0.890</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A$_4$</td>
<td>0.5</td>
<td>0.858</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B$_1$</td>
<td>0.5</td>
<td>0.325</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B$_2$</td>
<td>0.5 à 1.0</td>
<td>0.5</td>
<td>0.498</td>
<td>2.27</td>
<td>1</td>
<td></td>
<td></td>
<td>N + R</td>
</tr>
<tr>
<td>B$_3$</td>
<td>0.5</td>
<td>0.773</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C$_1$</td>
<td>0.5</td>
<td>0.725</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C$_2$</td>
<td>0.5</td>
<td>0.848</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C$_3$</td>
<td>0.5</td>
<td>0.773</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(*) Celle-ci inclut un séparateur gonflable de 1 m.  
(2) R = Rouge J = Jaune N = Noir.

4. PROGRAMMES ET DISPOSITIFS DES ESSAIS

L’un des buts essentiels de l’expérimentation a consisté à vérifier si la méthode de détermination de la traction de service ($T_A$), basée sur l’interprétation des mesures de fluage du scellement, s’appliquait au cas des argiles plastiques.

On rappellera brièvement le principe de cette méthode [3], [4].

On établit tout d’abord, pour différents niveaux de traction, l’évolution des déplacements absolus de l’extrémité du tirant ($\Delta L$) en fonction du logarithme du temps ($\log t$). Les relations obtenues, qui donnent en fait la mesure du fluage du scellement, présentent certaines propriétés :

- elles restent pratiquement linéaires ($^*$) jusqu’à une valeur critique de la traction ($T_c$), et cela indépendamment de la vitesse de chargement ;
- leur pente ($\alpha$) croît avec le taux de traction ($T_I$).

La traction critique ($T_c$) peut être déterminée graphiquement. Le diagramme pente ($\alpha$) traction ($T_I$) est linéaire jusqu’à la traction critique ($T_c$) et s’incurve au-delà de cette valeur, ce qui correspond à une accélération du fluage.

La valeur de la traction de service ($T_A$), appelée aussi traction admissible, est prise égale à une fraction de la traction critique ($T_c$). Les recommandations françaises actuellement en vigueur [5] préconisent un coefficient minorateur égal à 1.1 ($^{**}$). Les normes allemandes, par contre, stipulent d’adopter comme valeur de la traction de service ($T_A$) un taux de traction pour lequel le fluage reste inférieur à 6 mm pour un intervalle de temps de l’ordre de cinquante ans [6].

Les programmes de chargement retenus (tableau 3) consistent en une succession de paliers de traction croissants, d’égaux durées et intensités. Au total, quinze essais ont été réalisés, dont quatorze du type statique et un du type cyclique.

La précision de la mesure des déplacements de la tête de l’armature et des extrémités du bulbe, par rapport à des points fixes, est de l’ordre du 1/100 de millimètre. Les efforts appliqués ont été mesurés au moyen de dynamomètres à jauges, intercalés entre les massifs de réaction et les vérins. L’ensemble des appareils de mesure utilisés a fait l’objet d’un étalonnage préalable. La figure 2 offre une vue générale des dispositifs de mesure et de mise en traction utilisés en surface.

($^{**}$) Ces mêmes recommandations préconisent un coefficient de 1.5 sur la traction limite ($T_L$). Il y a lieu d’adopter comme traction de service ($T_A$), la plus petite des valeurs obtenues par l’application de ces deux coefficients minorateurs.

(*) A l’exception des toutes premières minutes d’application de la charge.
Fig. 2. — Vue générale du plot C. Exécution d’un essai de groupe sur trois tirants.

### TABLEAU 3

<table>
<thead>
<tr>
<th>Tirant</th>
<th>Essai n°</th>
<th>Traction maximale atteinte au cours de l’essai (1)</th>
<th>Durée des paliers</th>
<th>Incrément de traction par palier (kN)</th>
<th>Détèrrement (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A₁</td>
<td>1</td>
<td>$T_c$</td>
<td>1 h</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>$T_L$ (chargement cyclique)</td>
<td>1 h</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>$T_L$</td>
<td>1 h</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td>A₂</td>
<td>1</td>
<td>$T_L$</td>
<td>1 h</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>$T_L$</td>
<td>1 h</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td>A₃</td>
<td>1</td>
<td>$T_c$</td>
<td>1 h</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Calage à long terme &lt; $T_c$</td>
<td>1 an</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td>A₄</td>
<td>1</td>
<td>$T_c$</td>
<td>1 h</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>$T_L$</td>
<td>6 h</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td>B₁</td>
<td>1</td>
<td>$T_L$</td>
<td>1 h</td>
<td>100</td>
<td>+</td>
</tr>
<tr>
<td>B₂</td>
<td>1</td>
<td>$T_L$</td>
<td>1 h</td>
<td>100</td>
<td>+</td>
</tr>
<tr>
<td>B₃</td>
<td>1</td>
<td>$T_L$</td>
<td>1 h</td>
<td>100</td>
<td>+</td>
</tr>
<tr>
<td>C₁</td>
<td>1</td>
<td>$T_L$</td>
<td>30 mn</td>
<td>75</td>
<td>+</td>
</tr>
<tr>
<td>C₂</td>
<td>1</td>
<td>$T_L$</td>
<td>1 h</td>
<td>150</td>
<td>+</td>
</tr>
<tr>
<td>C₃</td>
<td>1</td>
<td>$T_L$</td>
<td>30 mn</td>
<td>75</td>
<td>+</td>
</tr>
</tbody>
</table>

(1) $T_c$: traction critique; $T_L$: traction limite.
(2) A ce jour, les tirants du plot A n’ont pas été excavés.
5. L'INTERPRETATION DES MESURES DE FLUAGE ET LA VALIDITE DE LA METHODE

Les figures 3 a et b présentent un exemple caractéristique de relation $\Delta l - \log t$ et de détermination graphique des tractions critiques ($T_c$), pour le tirant $A_4$.

On remarque :
- la quasi linéarité (à l'exception des toutes premières minutes des relations $\Delta l - \log t$ jusqu'à la traction critique ($T_c$);
- la similitude des relations $\Delta l - \log t$, correspondant à la tête du bulbe et l'extrémité des torons, ce qui prouve bien que l'observation des déplacements des torons en surface est bien représentative de ce qui se passe au niveau du scellement (fig. 3 a);
- l'existence d'un coude caractéristique sur les graphiques $a_i - T_i$, au-delà duquel les fluages augmentent non seulement très rapidement avec le temps mais cessent d'être prévisibles en raison de la non-linéarité de la relation $\Delta l - \log t$ (fig. 3 b);
- enfin, que les différentes relations caractéristiques ont exactement les mêmes allures que celles obtenues pour des tirants scellés dans des matériaux pulvéruents.

L'interprétation des essais effectués sur $A_4$ (fig. 4), sollicité une première fois par paliers de une heure, puis par paliers de six heures conduit à des valeurs
de la traction critique $T_c$ du même ordre (850 kN). Ceci étant la preuve que $T_c$ ne dépend pas de la vitesse de chargement, on voit qu'il semble possible de prédire le comportement futur du scellement à partir de conclusions tirées d'un essai d'arrachement ne durant tout au plus qu'une dizaine d'heures.

Il faut toutefois faire remarquer, que le fait de ne posséder qu'une seule expérimentation de ce type, en limite la portée.

L'ensemble des essais auxquels a été soumis le tirant A3 montre qu'un tirant calé à une traction légèrement inférieure (825 kN) à celle de sa traction critique $T_c$ (850 kN) n'indique pas de perte de tension notable (fig. 5). En effet, en l'espace de quatre mois, le tirant A3 a perdu environ 100 kN sur lesquels 65 kN sont dus à l'incidence de la mise en tension du tirant A4 situé à proximité immédiate (voir à ce sujet § 7).

Le tableau 4 présente les valeurs des tractions caractéristiques $T_c$ et $T_A$ pour les tirants des plots A et B, ainsi que les valeurs des fluages correspondants, calculés pour des intervalles de 10 à 100 mn.

Les essais ont montré par ailleurs :
- que de faibles dépassements de la traction critique $T_c$ (de l'ordre de 10% de cette dernière) n'entraînent aucune diminution de la capacité d'ancrage ;
- que des sollicitations de types cycliques d'amplitudes modérées (de l'ordre du 1/10 de la traction limite $T_L$) ne détériorèrent pas la tenue d'un scellement ;
- qu'une fois la traction limite $T_L$ atteinte, un tirant perdait environ 50% de sa capacité d’ancrage initiale.

<table>
<thead>
<tr>
<th>Tirant</th>
<th>Traction critique $T_c$ (kN)</th>
<th>$f_{TC}$ (mm)</th>
<th>Traction de service $T_A$ (kN)</th>
<th>$f_{TA}$ (mm)</th>
<th>Traction limite $T_L$ (kN)</th>
<th>$T_L/T_A$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>1 050</td>
<td>0.4</td>
<td>950</td>
<td>0.3</td>
<td>1 200</td>
<td>1.30</td>
</tr>
<tr>
<td>A2</td>
<td>850</td>
<td>0.4</td>
<td>760</td>
<td>0.3</td>
<td>1 100</td>
<td>1.45</td>
</tr>
<tr>
<td>A3</td>
<td>850</td>
<td>0.5</td>
<td>750</td>
<td>0.3</td>
<td>1 200</td>
<td>1.60</td>
</tr>
<tr>
<td>A4</td>
<td>850</td>
<td>0.5</td>
<td>760</td>
<td>0.4</td>
<td>1 000</td>
<td>1.35</td>
</tr>
<tr>
<td>B1</td>
<td>600</td>
<td>0.5</td>
<td>550</td>
<td>0.3</td>
<td>800</td>
<td>1.50</td>
</tr>
<tr>
<td>B2</td>
<td>700</td>
<td>0.6</td>
<td>620</td>
<td>0.5</td>
<td>900</td>
<td>1.45</td>
</tr>
<tr>
<td>B3</td>
<td>850</td>
<td>0.6</td>
<td>760</td>
<td>0.5</td>
<td>1 100</td>
<td>1.45</td>
</tr>
</tbody>
</table>

($f_{TC}$ : fluage correspondant à la traction critique $T_c$.
($f_{TA}$ : fluage correspondant à la traction de service $T_A$.

6. LES VALEURS DES TRACTIONS LIMITES ET LEUR DISPERSION

Le tableau 5 présente l'ensemble des valeurs des tractions limites $T_L$ obtenues pour la totalité des tirants.

<table>
<thead>
<tr>
<th>Tirant</th>
<th>$A_1$</th>
<th>$A_2$</th>
<th>$A_3$</th>
<th>$A_4$</th>
<th>$B_1$</th>
<th>$B_2$</th>
<th>$B_3$</th>
<th>$C_1$</th>
<th>$C_2$</th>
<th>$C_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traction limite $T_L$ (kN)</td>
<td>1 200</td>
<td>1 100</td>
<td>1 200</td>
<td>1 000</td>
<td>800</td>
<td>900</td>
<td>1 100</td>
<td>950</td>
<td>750</td>
<td>900</td>
</tr>
</tbody>
</table>

($f$ Cette valeur est estimée car le tirant $A_3$, toujours soumis à un essai de longue durée, n'a pas pu être sollicité jusqu'à la rupture de son scellement.
On constate que dans l’ensemble, les valeurs des tractions limites \((T_L)\) sont particulièrement élevées compte tenu de la modestie des longueurs de scellement \((l_i = 6 \text{ m})\) et de la médiocrité relative du sol.

Les résultats du plot A sont particulièrement intéressants. En raison de la quasi-similitude de leurs paramètres d’injection (voir tableau 2), il est possible de se faire une idée sur la dispersion des tractions limites \((T_L)\). Il apparaît que pour des tirants dont l’exécution du scellement a fait l’objet d’un maximum de soins, l’écart entre tractions limites \((T_L)\) extrêmes peut atteindre 200 kN, ce qui situe ces mêmes tractions à environ ±10 % de la valeur moyenne (soit 1 125 kN).

Seule l’observation visuelle des bulbes de scellement qui pourrait expliquer la raison de ces écarts, n’a pas pu être effectuée pour l’instant. L’étude de la résistance à long terme du tirant A3 interdisant toute excavation du plot A.

7. RESISTANCE DU TIRANT AU SEIN DU GROUPE

Le même tableau 5 indique, pour le plot C, des tractions limites \((T_L)\) nettement inférieures (en moyenne 25 % de moins que celles du plot A) pour des paramètres d’injection somme toute très voisins (voir tableau 2).

Les trois tirants de ce plot, dont les entre-axes sont de 1 m, ont fait l’objet d’un essai de groupe consistant à les mettre en tension simultanément tout en mesurant le flûge du scellement de chaque tirant (fig. 2).

Le programme de chargement observé, présentait quelques particularités (fig. 6) :
- le taux de traction appliqué aux tirants latéraux \(C_1\) et \(C_3\) est resté constamment supérieur à celui du tirant central \(C_2\);
- lors de l’application d’un nouvel incrément de tension aux tirants \(C_1\) et \(C_3\), la traction de palier correspondante est rigoureusement maintenue sur le tirant central \(C_2\).

L’analyse et la comparaison des relations \(\Delta T - \log t\) a montré :
- que toute application de nouveau palier de charge aux tirants latéraux \(C_1\) et \(C_3\) provoque une augmentation nette du déplacement du scellement du tirant central \(C_2\), mais cela uniquement lors des premières minutes. On observe par la suite une stabilisation, tout au moins jusqu’à la traction critique \((T_c)\);
- que dans l’ensemble, à taux égal de traction, le flûge du tirant central \(C_2\) est légèrement plus important que celui des tirants voisins et sa traction limite \((T_L)\) inférieure, d’environ 20 %, à celle des tirants \(C_1\) et \(C_3\). Le déterrrement des bulbes à permis de vérifier que cet écart de capacité d’ancrage ne pouvait être nullement attribué à la géométrie du bulbe : le diamètre moyen du tirant central \(C_2\) étant même légèrement supérieur à celui du tirant latéral \(C_1\). La raison semble bien être la proximité des tirants. L’excavation a montré que les surfaces latérales des bulbes du plot C se trouvaient parfois à moins de 70 cm.

Parallèlement, l’interprétation des résultats obtenus sur le tirant A3 confirme l’incidence défavorable de la mise en tension d’un tirant sur un tirant voisin déjà calé.

Le tirant A3, soumis à un essai de longue durée (six mois) a été calé à une traction voisine de 825 kN. L’arrachement du tirant A3, situé à 2 m du tirant calé A1, a provoqué une perte de tension irréversible de ce dernier tirant de l’ordre de 65 kN, soit 8 % environ de la tension de blocage (fig. 5).

Un autre essai, effectué sur le tirant A3, situé cette fois à 4 m d’A3, n’a entraîné aucune chute de tension (fig. 5).

L’ensemble des observations effectuées sur les plots C et A semblent indiquer que, dans notre cas précis, un écartement de 2 m entre tirants constitue une valeur en-dessous de laquelle il n’est pas prudent de descendre si l’on ne tient pas à diminuer considérablement la capacité d’ancrage des tirants.

8. LES PARAMETRES DE L’INJECTION ET LA RESISTANCE ULTIME DU TIRANT

On envisageait d’étudier initialement sur le plot B, pour des longueurs et des quantités de coulis injectées identiques, l’incidence d’une variation de la pression d’injection sur la traction limite \((T_L)\).

La réalisation des tirants du plot A, mis en œuvre en premier, montra que la pression d’injection se stabilisait toujours et pour l’ensemble des phases, aux alentours de 0,5 MPA (pression voisine de la pression limite \(p_0\)).

On décida alors de faire uniquement varier le nombre de phases d’injection, les quantités de coulis injectés devant rester identiques pour les trois tirants. Mais là encore, l’apparition de résurgences conduisit à limiter les quantités de coulis prévues, manifestement trop élevées.

On se trouva finalement obligé d’étudier l’effet conjugué du nombre de phases et des quantités injectées, sur la résistance ultime.
Les essais de traction effectués sur les tirants des plots A et B, ainsi que l’observation des tirants déterrés de ce dernier plot, montrent :

- que si les tractions $T_L$ et $T_c$ d’un tirant augmentent en principe avec la quantité de coulis ($V_s$) entrant effectivement dans la constitution du scellement (fig. 7a), au-delà d’une certaine quantité injectée ($V_i$), ces mêmes tractions cessent de croître (fig. 7b) ;
- que les tractions $T_L$ et $T_c$ augmentent avec le nombre de phases d’injections (fig. 7a et 7b). Les tirants du plot A injectés en trois phases et le tirant B3, également injecté en trois phases présentent, malgré des quantités injectées ($V_i$) quelque peu différentes, des tractions $T_L$ et $T_c$ du même ordre. Le partage de la quantité globale de coulis à injecter en plusieurs phases favorise, semble-t-il, la constitution d’un bulbe de scellement plus important et par conséquent capable de mobiliser des efforts de traction plus élevés.

Pour ce qui est des deux tirants C1 et C2 également déterrés (*), la proportionnalité de la relation « capacité d’ancrage-quantité injectée » est moins évidente. Si, d’une part, les tractions limites ($T_L$) des tirants C1 et B3 sont bien du même ordre (900 kN) pour des quantités de coulis effectivement très voisines (0.28 contre 0.25 m$^3$), la résistance ultime ($T_L$) du tirant central C2, dont le volume réel du bulbe est égal à 0.58 m$^3$, n’a pas excédé 750 kN.

On voit par là toute l’incidence du facteur « écartement entre tirants » sur une relation du type « capacité d’ancrage - quantité de coulis ».

9. L’INJECTION ET LA FORMATION DU BULBE DE SCELLEMENT

L’observation du mécanisme de formation des bulbes a été également effectuée sur les tirants du plot B : les coulis ayant été colorés pour mieux discerner les différentes phases d’injection.

On a pu établir pour chaque tirant détréssé, à partir du relevé des diamètres, une comparaison entre la quantité de ciment injectée à partir de la surface ($V_i$) et la quantité de ciment présente dans le bulbe autour de l’armature ($V_s$). Le tableau 6 indique le relevé des différents paramètres (fig. 8).

On remarque pour les tirants du plot B :
- que B3, injecté en une phase (0.077 m$^3$ de coulis par manchette) présente un maximum de déperditions ;
- que B2, injecté en deux phases (0.0385 m$^3$ de coulis par manchette et par phase) présente une déperdition plus faible ;

(*) Le tirant C3 n’a pas pu être détréssé pour des raisons de stabilité de fouille.
TABLEAU 6

<table>
<thead>
<tr>
<th>Tirant</th>
<th>Quantité de coulis injecté $V_i$ (m$^3$)</th>
<th>Quantité de coulis autour de l'armature $V_a$ (m$^3$)</th>
<th>Déperdition (%</th>
<th>Diamètre moyen du bulbe (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B_1$</td>
<td>0.310</td>
<td>0.090</td>
<td>env. 70</td>
<td>16</td>
</tr>
<tr>
<td>$B_2$</td>
<td>0.450</td>
<td>0.220</td>
<td>env. 50</td>
<td>24</td>
</tr>
<tr>
<td>$B_3$</td>
<td>1.200</td>
<td>0.420</td>
<td>env. 65</td>
<td>24</td>
</tr>
<tr>
<td>$C_1$</td>
<td>0.650</td>
<td>0.250</td>
<td>env. 60</td>
<td>23</td>
</tr>
<tr>
<td>$C_2$</td>
<td>0.780</td>
<td>0.545</td>
<td>env. 30</td>
<td>36</td>
</tr>
</tbody>
</table>

(*) Les valeurs indiquées ne tiennent pas compte des quantités injectées dans le séparateur.

- que $B_1$, injecté en trois phases (0.088 m$^3$ de coulis par manchette et par phase) présente à nouveau une déperdition de l'ordre de celle du premier tirant.

Ces observations restent difficiles à interpréter. Leur analyse incite toutefois à penser que le partage des quantités injectées en plusieurs phases, réduit quelque peu les déperditions.

On est cependant frappé par les taux élevés de déperditions du coulis. Celles-ci, provoquant le claquage de l'argile, se présentent sous forme de nappes horizontales d'épaisseurs variables. Ces nappes, se propageant à quelques mètres du scellement, finissent par se ramifier en réseaux extrêmement tenus, épousant le système de fissuration naturel de l'argile. On est aussi en droit de se demander si les différents claquages et apophyses n'influencent pas sur la résistance globale du tirant par l'amélioration des propriétés de l'argile, notamment au contact immédiat du bulbe.

Les observations effectuées sur le plot $C$, inciteraient aussi à croire que le facteur "écartement entre tirant" a une répercussion certaine sur l'importance des déperditions : le tirant $C_2$, injecté entre un tirant terminé ($C_3$) et un tirant déjà partiellement injecté ($C_1$), accuse le plus faible taux de pertes.

La répartition du coulis des phases consécutives est quelconque. Le coulis de la première phase ayant fait prise est brisé, puis repoussé vers l'extérieur ou contourné par le coulis des phases suivantes (fig. 8 et 9). On obtient bien, après chaque phase, une augmentation notable du diamètre.

**TIRANT $B_2$**

**Bulbe de scellement**

**SEPAREATEUR**

**TIRANT $C_1$**

**Bulbe de scellement**

**SEPAREATEUR**

Fig 8. — Géométrie des bulbes de scellement des tirants $B_1$ et $C_1$.
10. LA TRANSMISSION DE L’EFFORT DE TRACTION LE LONG DU SCELEMENT

La répartition des déformations unitaires $\Delta l/l$, mesurées à l’aide de l’extensomètre logé au sein du bulbe (fig. 10), a la même allure que celle observée sur les pieux arrachés. Les difficultés liées à la détermination du module équivalent du bulbe, n’ont pas permis pour l’instant d’établir les courbes de mobilisation du frottement latéral.

Il faut signaler que lors du déterrement, on a pu observer au contact du bulbe des surfaces brillantes et striées, qui pourraient être des surfaces de cisaillement. Celles-ci prouveraient en quelque sorte la similitude des mécanismes de ruptures des tirants et des pieux arrachés.

Enfin, l’observation du bulbe a mis en évidence l’extrême fissuration du coulis de la zone de scellement et cela sur toute la longueur.

11. LES CARACTERISTIQUES GEOTECHNIQUES DE L’ARGILE ET LA RESISTANCE DU TIRANT

Dans la mesure où la rupture s’effectue au contact de la surface latérale du bulbe, on peut tenter de déduire des valeurs de frottement latéral unitaire $T_{\text{max}}$ correspondantes.

Le tableau 7 présente ces valeurs pour le plot B.

<table>
<thead>
<tr>
<th>Tirant</th>
<th>Traction limite $T_{\text{L}}$ (kN)</th>
<th>Diamètre moyen (cm)</th>
<th>Surface latérale du scellement (*) (m$^2$)</th>
<th>Frottement latéral unitaire $T_{\text{max}}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B_1$</td>
<td>800</td>
<td>16</td>
<td>env. 3.00</td>
<td>265</td>
</tr>
<tr>
<td>$B_2$</td>
<td>900</td>
<td>24</td>
<td>env. 4.50</td>
<td>200</td>
</tr>
<tr>
<td>$B_3$</td>
<td>1 100</td>
<td>24</td>
<td>env. 4.50</td>
<td>240</td>
</tr>
</tbody>
</table>

(*) Celle-ci inclut la surface du séparateur gonflable.
On voit combien ces valeurs sont élevées, trois fois plus élevées en moyenne que les valeurs de cisaillement mesurées en laboratoire (60 à 80 kPa). De pareilles valeurs pourraient s'expliquer par une augmentation du taux de résistance de l'argile au voisinage immédiat du bulbe, provoquée par l'injection (voir § 9).

12. CONCLUSIONS

En dépit du trop faible nombre d’essais réalisés, notamment ceux portant sur de longues durées, on retiendra comme conclusions principales :
- que les tirants injectés peuvent être utilisés comme ancrages provisoires et définitifs dans les argiles plastiques, à condition de ne pas être sollicités au-delà d'une traction de service \( (T_A) \) définie comme précédemment (voir § 4);
- qu'il est possible, pour une argile plastique, de prédire à partir de conclusions tirées d'un essai à court terme, des enseignements relatifs au comportement à long terme d'un ancrage injecté (voir § 5);
- que la traction limite \( (T_L) \) d'un tirant isolé scellé dans une argile plastique, reste essentiellement proportionnelle à la quantité effective de coulis entrant dans la constitution du bulbe (voir § 8);
- enfin, qu'il existe pour les tirants scellés dans ce type de sol, un phénomène « d'effet de groupe », se traduisant par une diminution sensible de la capacité d'ancrage en deçà d'une valeur critique de l'écartement (2 m dans notre cas) entre tirants (voir § 7).

13. REFERENCES BIBLIOGRAPHIQUES

EXPERIMENTAL VERIFICATION OF ANCHORED CURTAIN WALL
Vérification expérimentale d'un rideau ancré

by

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P.H. VIEIRA DIAS
Engineer - Division Chief - Tecnosolo S.A.

INTRODUCTION

The project referred to here is an experimental verification of the stability of a large anchored excavation wall, more than 20 m high and with a perimeter of about 430 m. The excavation was made in residual soil, partially collapsible, for the new headquarters building of the Central Bank of Brazil, located in the Capital city of Brasilia.

The design of the wall was based on soil parameters obtained from laboratory tests made on undisturbed samples. Toward the end of the construction it was determined that the walls were moving and in particular one wall gave evidence of horizontal movement that reached 25 cm (more than 1% of the height), moving into the excavation (wall No 2).

Fig. 1. — Photograph of the excavation and the anchored wall.
To investigate the movement, the owner contracted with a consultant. As there was disagreement between the owner’s consultant and the design engineer regarding the soil parameters used for design (cohesion $c$ and friction angle $\phi$), the owner engaged the senior author to arbitrate the question. His suggestion to perform experimental verification of the anchorage loads was accepted by all parties concerned. The existing wall had not been instrumented.

**METHODOLOGY**

a) The residual loads in the anchorages were determined in two typical sections of each of the four walls by testing with a hydraulic jack and calibrated gage (Costa Nunes, 1966).

Incorporation loads in the anchorages were then reduced to slightly less than the residual loads (about 5% less), and afterwards retested until the residual load increased between tests. The lower load, thus obtained was considered to correspond to the bearing pressures. The limiting load for each anchorage was also determined. This is the largest load applied for which the movements were still stabilized.

b) From the results obtained from testing 42 anchorages, and at least five re-post-tensioning tests each, diagrams of the existing pressures were obtained.

As to the distribution of the earth pressures over the walls, these will closely approximate uniform distribution with the minimum pressures (always the highest anchorages) varying about 20% from the average pressures.

c) These diagrams were increased by the influence of the surcharge to be placed in the future behind the wall, and compared with the limiting loads of the anchorages that were also determined experimentally, considering also relaxation loads. The necessity of reinforcing the walls was determined. Using the safety factors of 1.75 fixed by Brazilian Standards (NB-565) for permanent anchorages. Consequently, three of the four walls were reinforced; the maximum increase was 13% over the capacity of the anchorages for wall No 2, which had suffered the undesirable movement (see table 1).
d) The relaxation factors of the anchorages were always less than 0.5 mm, well below the value of 1.0 mm, considered as adequate for sandy soils by the Brazilian Standards. The German Standard, DIN 4125, allows values of 2 mm, for any soil. It is observed that the relaxation factors were determined for loads between 40 and 60 tons.

**ANALYSIS OF REINFORCEMENT REQUIRED**

The sequence of calculations used for each wall, considering the average loads in the anchorages, is the following:

a) Determination of $C_a$ (minimum residual load, determined experimentally);

b) Determination of the increase of loads $C_s$, caused by the surcharge;

c) Determination of the load $C_a + C_s$;

d) Determination of the limiting test load, $1.75 (C_a + C_s)$, necessary;

e) Determination of the limiting test load $C_l$, experimentally;

f) Verification of the necessity, or not, of reinforcement, comparing the loads from items d) and e):
   - if $1.75 (C_a + C_s) \leq C_l$, reinforcement is not necessary;
   - if $1.75 (C_a + C_s) > C_l$, we pass to the following step;

g) Calculation of reinforcement necessary, in percentage:

$$\frac{1.75 (C_a + C_s)}{C_l} - 1 \times 100;$$

The calculated values are presented on table I.

**TABLE I**

Summary of calculated anchorage loads and wall reinforcement necessary (loads are in ton)

<table>
<thead>
<tr>
<th>WALL</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_a$</td>
<td>48.9</td>
<td>37.0</td>
<td>42.1</td>
<td>41.1</td>
</tr>
<tr>
<td>$C_s$</td>
<td>—</td>
<td>3.0</td>
<td>3.0</td>
<td>2.4</td>
</tr>
<tr>
<td>$C_a + C_s$</td>
<td>48.9</td>
<td>40.0</td>
<td>45.1</td>
<td>43.5</td>
</tr>
<tr>
<td>$1.75 (C_a + C_s)$</td>
<td>85.6</td>
<td>70.0</td>
<td>79.0</td>
<td>76.1</td>
</tr>
<tr>
<td>$C_l$</td>
<td>82.7</td>
<td>62.2</td>
<td>72.5</td>
<td>70.8</td>
</tr>
<tr>
<td>compare $d$ to $e$</td>
<td>$&gt;$</td>
<td>$&gt;$</td>
<td>$&gt;$</td>
<td>$&gt;$</td>
</tr>
<tr>
<td>reinforcement required (additional capacity)</td>
<td>3%</td>
<td>13%</td>
<td>9%</td>
<td>7%</td>
</tr>
</tbody>
</table>

**INSTRUMENTATION**

As the project schedule already had been substantially delayed it, was undesirable to interrupt the construction while performing the investigation or necessary reinforcement of the wall. It was resolved, therefore, to instrument the wall and to continue with construction, providing an alarm system for any eventuality. The instrumentation included mechanical cells of the «Interfels plates» type and extensometers,
as well as the method of re-post-tensioning that already had been used extensively, all of which accompanied the changes in actual loads bearing on the anchorages before, during, and after execution of reinforcement.

The instrumentation confirmed as correct the reinforcement measures previously recognized, and indicated the necessity of the removal of the surcharge of construction materials that had been stored next to the walls.

RECALCULATION OF THE EARTH PARAMETERS

Although beyond the study objectives, the results permit the determination of values of the parameters $c$ and $\varnothing$, once the unit weight is known, as well as the distribution of earth pressures over the walls. Table II summarizes the values of $c$ and $\varnothing$ and the coefficient of active horizontal earth-pressure $K_{ah}$.

<table>
<thead>
<tr>
<th>$\varnothing$</th>
<th>10°</th>
<th>15°</th>
<th>20°</th>
<th>25°</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_{ah}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>wall 1</td>
<td>0.97</td>
<td>0.65</td>
<td>0.51</td>
<td>0.40</td>
</tr>
<tr>
<td>2</td>
<td>6.44</td>
<td>4.24</td>
<td>2.99</td>
<td>1.78</td>
</tr>
<tr>
<td>3</td>
<td>7.40</td>
<td>5.40</td>
<td>4.30</td>
<td>3.26</td>
</tr>
<tr>
<td>4</td>
<td>7.17</td>
<td>5.12</td>
<td>3.99</td>
<td>2.91</td>
</tr>
<tr>
<td>$c$ values</td>
<td>7.21</td>
<td>5.18</td>
<td>4.05</td>
<td>2.98</td>
</tr>
</tbody>
</table>

It was verified that the induced values of $c$ and $\varnothing$ were clearly less than some of those obtained from laboratory tests on undisturbed samples.

CONCLUSIONS

In the opinion of the authors, the method of re-post-tensioning, in the absence of a representative number of instrumented anchorages, offers the possibility of determining the approximate distribution of earth pressures acting on anchorage curtain walls and the factor of safety of the construction.

REFERENCES


COSTA NUNES (A.J. da) and VIEIRA DIAS (P.H.). — «Some Results from the Systematic Verification of the Load Capacity of Anchorages», 10th Brazilian Congress on Large Dams, Curitiba, Brasil (1975).


BEHAVIOUR OF GROUND ANCHORS IN STIFF CLAYS
Comportement de tirants d’ancrage en argiles raides

by
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SUMMARY
The results of a full scale investigation on instrumented anchors, bored through a typical pliocenic stiff clay formation, are reported and analyzed. The variation with depth of the normal stress in the reinforcement bars, found to be non-linear up to failure load, is predicted by a non-linear numerical analysis of the interaction among the reinforcement, the grout mortar and the surrounding soil taking into account the cracking of the mortar. The best correlation with experimental results, however, is obtained with values of soil and mortar properties somewhat different from laboratory values.

A similar correlation of load-upheaval curves has been obtained, by the same analysis and with the same values of parameters, only for the first stage of the curves and not for the final stage, near to failure load.

This inconsistency is discussed, pointing out the need for further theoretical and experimental investigations.

1. INTRODUCTION

In a recent paper (Sapio, 1975) the results of a full scale investigation on 3 test anchors bored through a typical pliocenic clay formation of Southern Italy were reported.

Calling $\tau_m$ the average adhesion at failure between the grouted length of the anchor and the soil, and $c_u$ the undrained cohesion of the soil as measured in laboratory tests on undisturbed samples, values of the ratio $\alpha = \tau_m/c_u$ ranging between 0.28 and 0.36 were measured (see table 1).

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Nominal diameter $\varnothing$</th>
<th>Grouted length $L$</th>
<th>Depth below ground level</th>
<th>Ultimate uplift load $Q_u$</th>
<th>Adhesion coefficient $\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>220 mm</td>
<td>3.80 m</td>
<td>7.20 ÷ 11.00 m</td>
<td>20 tons</td>
<td>0.28</td>
</tr>
<tr>
<td>B</td>
<td>220 mm</td>
<td>7.20 m</td>
<td>7.80 ÷ 15.00 m</td>
<td>50 tons</td>
<td>0.36</td>
</tr>
<tr>
<td>C</td>
<td>220 mm</td>
<td>12.80 m</td>
<td>6.20 ÷ 19.00 m</td>
<td>80 tons</td>
<td>0.33</td>
</tr>
</tbody>
</table>

In the mean time, the values of normal stress in the reinforcement bars were measured by means of strain gauges, glued to the bars at different depths.

The stress variation with depth was far from linear not only at low stress level, as found for instance by Berardil (1967) and Adams and Klym (1972), but also at failure.

Such a finding is in contrast with the usual hypothesis of uniform shear stress distribution on the lateral surface of the anchor at failure. Accordingly,
the need for a further analysis of the behaviour of the anchors was underlined, considering the mutual interaction of the steel bars, the grout and the surrounding soil.

A step towards this goal is attempted in this paper; only two of the three anchors are considered, since in the third one the strain gauges failed at the beginning of the uplift test.

2. FIELD INVESTIGATION

2.1. Subsoil properties

Field experiences were carried out at Taranto (Italy) in a formation of overconsolidated stiff clays of pliocenic age, typical of the region (Apulia), where it is found in wide areas with rather uniform characteristics.

At the test site the formation is overlain by overburden soils, for a thickness of 3.5 m, and by a layer of partially cemented sand 2 m thick.

The clay is of medium to high plasticity; its grain size distribution is reported in fig. 1.

Physical properties, as determined on undisturbed samples, are listed in table II. Typical oedometer curves are reported in fig. 2; as it may be seen, the clay appears to be heavily overconsolidated.

Table II

Physical properties of clays from lab. tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (m)</th>
<th>Porosity n</th>
<th>Water content w</th>
<th>Unit weight γ t/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.50 - 7.90</td>
<td>43.7</td>
<td>30.8</td>
<td>1.94</td>
</tr>
<tr>
<td>2</td>
<td>9.80 - 10.20</td>
<td>38.5</td>
<td>22.9</td>
<td>2.06</td>
</tr>
<tr>
<td>3</td>
<td>12.00 - 12.40</td>
<td>36.5</td>
<td>21.0</td>
<td>2.10</td>
</tr>
<tr>
<td>A</td>
<td>13.45 - 13.75</td>
<td>37.5</td>
<td>22.0</td>
<td>2.08</td>
</tr>
<tr>
<td>4</td>
<td>14.20 - 14.60</td>
<td>40.6</td>
<td>24.1</td>
<td>2.01</td>
</tr>
<tr>
<td>B</td>
<td>14.70 - 15.10</td>
<td>39.9</td>
<td>24.3</td>
<td>2.04</td>
</tr>
<tr>
<td>6</td>
<td>20.60 - 21.00</td>
<td>37.1</td>
<td>20.6</td>
<td>2.07</td>
</tr>
<tr>
<td>7</td>
<td>25.00 - 25.40</td>
<td>38.6</td>
<td>22.9</td>
<td>2.06</td>
</tr>
<tr>
<td>Averages</td>
<td></td>
<td>39.3</td>
<td>23.6</td>
<td>2.05</td>
</tr>
</tbody>
</table>

Stress-strain curves were fitted by a hyperbola (Kondner and Zelasko, 1963), fig. 3 a, whose parameters \( a = 1/E_1 \) and \( b = 1/(\sigma_1 - \sigma_3) \), were obtained by the linear plot of \( \varepsilon/(\sigma_1 - \sigma_3) \) versus \( \varepsilon \) (fig. 3 b). Initial undrained tangent modulus \( E_1 \) ranges between 166 and 500 kg/cm²; the ultimate deviator stress \( (\sigma_1 - \sigma_3) \), derived by the hyperbolic interpolation is, on the average, 14% higher than the deviator stress at failure \( (\sigma_1 - \sigma_3) \). The fitting obtained with the hyperbola is rather good, as it may be seen in fig. 3 b.

Table II

Stress-strain curves were fitted by a hyperbola (Kondner and Zelasko, 1963), fig. 3 a, whose parameters \( a = 1/E_1 \) and \( b = 1/(\sigma_1 - \sigma_3) \), were obtained by the linear plot of \( \varepsilon/(\sigma_1 - \sigma_3) \) versus \( \varepsilon \) (fig. 3 b). Initial undrained tangent modulus \( E_1 \) ranges between 166 and 500 kg/cm²; the ultimate deviator stress \( (\sigma_1 - \sigma_3) \), derived by the hyperbolic interpolation is, on the average, 14% higher than the deviator stress at failure \( (\sigma_1 - \sigma_3) \). The fitting obtained with the hyperbola is rather good, as it may be seen in fig. 3 b.

Fig. 1. — Grain size distribution of clay.

Fig. 2. — Oedometer curves of clay.

Fig. 3. — Stress-strain curves of clay.
Fig. 3. — Triaxial compression tests. Stress-strain curves fitting by a hyperbola.

Fig. 4. — Shear box tests. Stress-strain curves fitting by a hyperbola.
Overburden soils

Sand partially cemented

Pliocene clayey formation

Fig. 5 — Test anchors.

Some unconsolidated undrained shear box tests were also performed; typical results are listed in table IV. A hyperbolic interpolation was attempted for these tests (fig. 4); the values of initial tangent modulus $\beta_i$ and ultimate strength $\tau_u$ thus obtained are also reported in table IV.

A comparison between tables III and IV shows that the hyperbolic interpolation is more suited for triaxial than for direct shear test results.

### TABLE III

<table>
<thead>
<tr>
<th>Sample</th>
<th>$E_i$ kg/cm²</th>
<th>$(\sigma_1 - \sigma_2)_u$ kg/cm²</th>
<th>$(\sigma_1 - \sigma_2)_f$ kg/cm²</th>
<th>$(\sigma_1 - \sigma_2)_u / (\sigma_1 - \sigma_2)_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>166</td>
<td>4.00</td>
<td>3.55</td>
<td>1.13</td>
</tr>
<tr>
<td>b</td>
<td>200</td>
<td>3.31</td>
<td>2.88</td>
<td>1.10</td>
</tr>
<tr>
<td>c</td>
<td>227</td>
<td>3.45</td>
<td>3.06</td>
<td>1.13</td>
</tr>
<tr>
<td>3a</td>
<td>208</td>
<td>7.14</td>
<td>5.98</td>
<td>1.19</td>
</tr>
<tr>
<td>b</td>
<td>263</td>
<td>7.69</td>
<td>6.65</td>
<td>1.16</td>
</tr>
<tr>
<td>6a</td>
<td>500</td>
<td>7.14</td>
<td>6.35</td>
<td>1.12</td>
</tr>
<tr>
<td>b</td>
<td>500</td>
<td>7.14</td>
<td>6.30</td>
<td>1.13</td>
</tr>
<tr>
<td>c</td>
<td>385</td>
<td>7.14</td>
<td>6.03</td>
<td>1.18</td>
</tr>
</tbody>
</table>

### TABLE IV

Elaboration of the shear box test results

<table>
<thead>
<tr>
<th>Sample</th>
<th>$\beta_i$ kg/cm²</th>
<th>$\tau_u$ kg/cm²</th>
<th>$\tau_i$ kg/cm²</th>
<th>$\tau_u / \tau_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aa</td>
<td>10.17</td>
<td>2.88</td>
<td>2.34</td>
<td>1.23</td>
</tr>
<tr>
<td>b</td>
<td>10.75</td>
<td>3.52</td>
<td>2.37</td>
<td>1.48</td>
</tr>
<tr>
<td>c</td>
<td>11.49</td>
<td>3.97</td>
<td>2.44</td>
<td>1.63</td>
</tr>
<tr>
<td>Ba</td>
<td>11.90</td>
<td>2.99</td>
<td>2.23</td>
<td>1.34</td>
</tr>
<tr>
<td>b</td>
<td>12.50</td>
<td>4.67</td>
<td>2.98</td>
<td>1.57</td>
</tr>
<tr>
<td>c</td>
<td>13.51</td>
<td>3.79</td>
<td>2.64</td>
<td>1.44</td>
</tr>
</tbody>
</table>

2.2. Test anchors

The two anchors considered in this paper are represented in fig. 5. The first one (anchor A) has a grouted length of 3.8 m and a total length of 11 m; the second one (anchor B), respectively of 7.2 m and 15 m.

Both anchors were drilled with rotary bit and a provisional steel casing 220 mm in diameter. At the bottom of the hole, a layer of fine sand, 50 cm thick, was poured before introducing the reinforcement, that consists of two Dividag rods 32.7 mm in diameter fastened to spacing rings.

The bars are instrumented with strain gauges as shown in fig. 5.

The grout mortar was obtained by mixing 1 m³ of fine sand, 1 200 kg of R. 325 cement and 800 l of water; it was poured through a tremie pipe lowered to the hole bottom.

During the mortar casting, the steel casing was progressively raised and finally left in place above the grouted length of the anchor.

Some specimens, prepared in laboratory with the same mix used in the field, gave the following average 28 days strength:

- compression strength: 310 kg/cm²
- bending strength: 36 kg/cm²

The reinforcement steel has the following characteristics:

- yield-stress: $86 \times 10^3$ kg/cm²
Uplift tests were carried out nearly two months after the construction of the anchors. The load was applied by means of a couple of hydraulic jacks, via a test frame with a 100 ton capacity. The displacements were measured by means of 4 dial gauges, and independently by optical levelling.

Three loading and unloading cycles were performed for each test, the last one kept to failure load.

The results obtained are shown in fig. 6 as curves of the vertical displacement $\Delta$ versus applied uplift load $Q$. The load distribution on the steel bars, derived from the strain gauges readings, is reported in fig. 7. As already said, such a distribution is non-linear up to the ultimate uplift load.

tensile strength : $105 \times 10^2$ kg/cm$^2$
Young's modulus : $2.2 \times 10^6$ kg/cm$^2$

The anchors were constructed by Fondedile S.p.A., Naples.

2.3. Uplift tests

Uplift tests were carried out nearly two months after the construction of the anchors. The load was applied by means of a couple of hydraulic jacks, via a test frame with a 100 ton capacity. The displacements were measured by means of 4 dial gauges, and independently by optical levelling.

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3. THEORETICAL INTERPRETATION

3.1. Hypothesis

To interpret the observed behaviour of the test anchors, the subsoil was treated as a homogeneous, isotropic, elastic half space, except for a thin layer surrounding the anchor where yield may eventually occur.

Yield occurrence, in terms of total stress, has been postulated when the shear stress at the interface between the anchor and the soil reaches a limit value; this yield stress has been assumed to be constant over the anchor length and function of the undrained cohesion of the soil.

In the calculation the above mentioned thin layer was assumed to coincide with the interface between the grout and the soil; shear displacement at this interface was related to corresponding shear stress by a hyperbolic law.

The overall conditions being undrained, a Poisson’s ratio equal to 0.5 was assumed for the soil.

The anchor body was treated as a homogeneous body whose deformability equals that of the grout-reinforcement system. Moreover, the occurrence of tension cracks in the upper part of the anchor was considered by assuming that, if the tensile stress in the grout exceeds the tensile strength, the normal load is resisted only by the steel reinforcement while the surrounding grout keeps the only function of transmitting the shear stress from the soil to the reinforcement.

3.2. Calculation procedures

The calculation procedure adopted is based on the discretization procedure developed for the study of pile foundations (Poulos and Davis, 1968; Mattes and Poulos, 1969; Evangelista, 1976); the anchor, actually, may be treated as a pile with a non-reacting base.

According to such procedures, the anchor was subdivided into K elements; for the i-th element an equation containing the unknown shear stresses on all the K elements and the displacement of the i-th element may be written. A further equation is obtained by the condition of overall vertical equilibrium. In matrix form, the displacement $\Delta_i$ of the soil corresponding to the K elements are connected to the interface shear stresses $\tau$ by the equation:

$$\begin{align*}
\{\Delta_i\} &= [S] \{\tau\}
\end{align*}$$

where $[S]$ represents the matrix of the influence coefficients of the soil, obtained by numerical integration of Mindlin’s formula. Calling $\Delta_i$, the displacement of the soil at the anchor base, and putting

$$\Delta'_i = \Delta_i - \Delta_0$$

can be expressed:

$$\begin{align*}
\Delta'_i &= [S'] \{\tau\}
\end{align*}$$

The displacements $\Delta_i$ of the elements of the anchor, under the action of the axial load $Q$ and of the tangential stress $\tau$ may be written:

$$\begin{align*}
\{\Delta_i\} &= \Delta_i + \Delta_0 - [A] \{\tau\} + \{\Delta^0\}
\end{align*}$$

where $\Delta_0$ is the unknown mutual displacement between the grout and the soil at the anchor base (fig. 8); the term $-[A] \{\tau\}$ represents the deformation of the anchor due to the stress $\tau$; $\{\Delta^0\}$ represents the deformation due to the load $Q$.

To construct matrix $[A]$ and to calculate deformations $\{\Delta^0\}$, the anchor was considered as a homogeneous cylinder. If tension cracks occur, the anchor has an equivalent modulus $E_1$ in the fissured zone, and a different modulus $E_2$ in the unfissured one. They are expressed respectively:

$$E_1 = \frac{4E_{SR}Q_{SR}}{\pi D^2} ; \quad E_2 = \frac{4(E_{SR}Q_{SR} + E_M\Omega_M)}{\pi D^2}$$

where:

$E_{SR}$, $Q_{SR}$ the Young’s modulus and the total area of reinforcement bars;

$E_M$, $\Omega_M$ the Young’s modulus and the area of the mortar;

$D$, the nominal diameter of the anchor.

The compatibility equations are expressed:

$$\{\Delta_j\} = \{\Delta_j\} + \{\tau^s\}$$

(1)

where $\{\tau^s\}$ represents the shear displacement between the mortar and the soil. By substituting:

$$\{[S'] + [A] \{\tau\}\} = \Delta_0 + \{[\Delta^0]\} - \{\tau^s\}$$

(2)

The displacement $\tau$ was related to the shear stress at the interface by a hyperbolic equation of the type:

$$\tau_i = \frac{\tau}{1 - \tau_\beta/\tau_0}$$

(3)

where $\beta$ is the initial gradient of the $\tau$, $\tau_0$ curve, and $\tau_0$ is the asymptotic limit value of $\tau$. A similar interface behaviour has been used by Clough and Duncan (1971) and by Desai (1974).

The equilibrium equation is expressed:

$$Q = [B] \{\tau\}$$

(4)

where:

$$\tau = \frac{\pi DL}{K} \sum_{i=1}^{K} \tau_i$$

Eqs. (2) and (4) offer the solution to the problem.

Eq. (3) being non linear, the whole system is non linear; it has been solved in increments, by considering loading steps $\Delta Q$ and writing eq. (3) in incremental form:

$$\Delta \tau_i = \frac{1}{\beta} \left(1 - \frac{\tau_i}{\tau_0}\right)$$

(5)

$\tau_i$ being the value of the interface shear stress just before the load increment $\Delta Q$. Since matrix $[A]$ and vector $\{\Delta^0\}$ vary with the length of cracked zone, after each loading step the occurrence and the extent of this zone is checked, and $[A]$ and $\{\Delta^0\}$ are eventually recalculated.

The solution of the system (2) and (4) gives the $K$ values of $\tau_i$; simple equilibrium considerations allow then the determination of the axial load $N$ in any section of the anchor.

The fraction $N_{SR}$ of $N$ taken up by the steel bars in the uncracked length is:

$$N_{SR} = \frac{E_{SR}Q_{SR}}{E_{SR}Q_{SR} + E_M\Omega_M}$$

On the contrary, in the cracked length the value of $N_{SR}$ is unknown, the location of the cracks being unknown. Nevertheless, the range of possible values of $N_{SR}$ may be defined; they vary between a minimum $N_{SR,min}$ (fig. 9) occurring if the mortar fully develops its tensile strength, and a maximum $N_{SR,max}$ corresponding to a fully cracked mortar. Of course, one may write:

$$N_{SR,min} = N - \sigma_{PM}\Omega_M; \quad N_{SR,max} = N$$

where $\sigma_{PM}$ is the tensile strength of the mortar.
3.3. Results and discussion

In order to compare the experimental results with those obtained in the calculations, the following physical parameters must be known:
- Length L, diameter D and depth \( L_s \) of the anchor;
- Percentage of reinforcement, expressed by the ratio \( \rho_{SR}/\rho_M \);
- Undrained modulus of the soil \( E_0 \);
- Young’s modulus of the steel, \( E_{SR} \), and of the mortar, \( E_M \), in tension;
- Tensile strength of the mortar, \( \sigma_{FM} \);
- Initial tangent to the curve connecting shear stress \( \tau \) and displacement \( \delta \) at the interface between mortar and soil;
- Limit value of \( \tau \), corresponding to the asymptote of the \( \tau \) -- \( \delta \) curve.

Besides geometrical parameters, the only defined property is the Young’s modulus of the steel bars, that was determined in the usual way and equals \( 2.2 \times 10^6 \) kg/cm².

Laboratory values of \( E_M \) and \( \sigma_{FM} \) are respectively of the order of \( 2 \times 10^5 \) kg/cm² and 30 kg/cm²; they have been assumed as valid in the calculations, notwithstanding the obvious differences in curing conditions between the laboratory and the site.

\( E_0 \), \( \beta \) and \( \tau_a \) have been determined, by trial and error, on the basis of best correlation to experimental results.

As a first trial, \( E_0 \) was assumed equal to the mean value of \( E_i \) (table III) and \( \beta \) to the mean value of \( \beta_i \) (table IV).

\( \tau_a \) was assumed: \( \tau_a = \alpha \cdot c_u \) with \( c_u \) taken from laboratory measurements (table III and IV) and \( \alpha \) (adhesion coefficient) from table I (\( \alpha = 0.28 \) for anchor A; 0.36 for anchor B).

The results thus obtained being unsatisfactory, successive trials have shown that a reasonably good correlation is obtained by assuming:

\( E_0 = 700 \) kg/cm²; \( \beta = 50 \) kg/cm²;
\( \tau_a = 2.75 \) kg/cm².
Fig. 10. — Load on steel bars. Comparison between theoretical and experimental values.

Fig. 11. — Calculated values of $\tau$ at failure.
The results obtained with such a set of parameters are reported in fig. 10 together with field data; the agreement may be seen to be rather good, except for the points near the anchor top for loads below the values producing tension cracks. Such differences could be explained with a decrease of the modulus $E_M$ in this highly stressed zone.

The value $c_r = 2.73$ kg/cm$^2$ corresponds to the mean value of undrained cohesion $c_u$ of the soil determined in laboratory, on samples 3, A, B and 6 falling below top levels of the anchors, while the value of $E = 700$ kg/cm$^2$ is derived from the same value of $c_u$ following the suggestions of Poulos (1972) for bored piles in clay. The value of $\beta = 50$ kg/cm$^3$ is rather different from the average laboratory value determined by means of shear box tests; in this connexion the differences between laboratory direct shear test on undisturbed samples and the field behaviour of the interface between the mortar and the soil must be recalled.

As already said, the calculations offer the possibility of predicting the load-displacement response of the top of the anchor. With the values of parameters discussed above the initial part of the load-displacement curve is predicted very well; on the contrary, for both anchors, the upheaval at high loads are grossly underestimated and the values of failure load overestimated.

It seems that the usual hypothesis of uniform shear stress $\tau = \sigma$, at failure does not depict the actual phenomenon. This is confirmed by the fact that different values of $\sigma$ $(0.28 \pm 0.56)$ are derived by the three test anchors of different length, but identical in any other respect.

A further confirmation may be obtained from the results of the calculations carried out to determine the stress distribution in the reinforcement bar. In fig. 11, the diagrams of the calculated values of $\tau$ at failure are reported for both anchors A and B; it is to be remembered that these shear stresses are compatible with the measured values of the stress in the steel rods. It may be seen that the distribution of $\tau$ is far from uniform, and the maximum value is lower than $c_u = c_r$. Finally, it is interesting to point out that the pattern of $\tau$, decreasing downwards, is in contrast with the usual interpretation of failure in terms of total stress. It may be argued that an effective stress analysis could be more suited, and could account for some drainage at the interface between the soil and the mortar, due to the relatively high mortar permeability.

In terms of effective stress the shear stress $\tau$ becomes a function of the radial normal stress at the interface; it is interesting to point out that, according to Mindlin formulas, the radial stress decreases downwards being compressive at the top and tensile at the bottom of the anchor.

4. CONCLUSIONS

Such a discrepancy is probably related to the hypothesis of a uniform distribution of $\tau = \sigma$, assumed in the analysis of failure in terms of total stress.

Further theoretical and experimental investigations are needed to elucidate this point; it appears that an interpretation in terms of effective stress could offer considerable advantages.

REFERENCES


ENCEINTE ÉTANCHE DE LA CENTRALE ÉLECTRIQUE DE BLAYE : 
ESSAIS ET MESURES SUR LES TIRANTS D’ANCRAGE 

Ground anchors: tests and measurements 

par 

G.-Y. FENOUX 
Directeur des Etudes de Solétanche - Paris - France 

SOMMAIRE 

Le projet de fondation de l’usine E.D.F., sur radier semi-enterré, comporte la substitution par du sable compacté à sec des couches de vase en place (1 million de mètres cubes à évacuer). 

Pour ce faire, une enceinte étanche en paroi mou­

lée a été retenue. Dégagée sur 17.5 m de hauteur, 

supportant outre la poussée des terres sur cette 

hauteur celle de l’eau sur 15 m, la paroi a 30 m de 

hauteur. Elle est ancrée en pied dans le substratum 

(graviers et marnocalcaire) et tenue en tête par des 

tirants. 

De 200 t de force de service, de 55 m de longueur 

totale (dont 25 m d’ancrage), au nombre de 832, les 

tirants ont fait l’objet d’essais et de mesures, tant 

préalables qu’au cours du chantier. 

Testés à 1.2 la charge de service, les tirants ont 

été bloqués à 0.5 fois celle-ci (100 t), afin de limiter 

le mouvement en arrière de la paroi. Par la suite, on 

attendait un mouvement en avant (vers la fouille) de 

la paroi en cours de terrassement avec mise en 

charge complémentaire progressive des tirants. 

Les mouvements de la paroi et les efforts dans 

les tirants ont été suivis avec attention et comparés 

avec les estimations préalables. 

EXPOSE DU PROBLEME 

Le site du Blayais, situé en bordure rive droite de 

la Gironde, à environ 50 km au nord-ouest de Bor­

deaux, a été retenu par l’EDF pour la construction 

d’une centrale électrique nucléaire. 

Il s’agit d’une zone marécageuse, avec la coupe 

schématique suivante : 

de + 2.5 NGF à —13.0 : vase (φ = 0, c = 0.4 à 

4 t/m²) ; 

de —13.0 à —19.0 : alluvions sablo-graveleu­

ses (graves) (φ = 35°) ; 

de —19.0 à —26.0 : marnocalcaire de l’éo­

cène (φ = 0, c = 15 t/m²) ; 

en-dessous de —26.0 : sable de l’éocène. 

On rencontre deux nappes : une nappe de surface, qui 

varie de 1 m environ avec la marée au voisinage immé­
diat du terrain naturel ; une nappe profonde baignant 

l’éocène avec un niveau statique voisin de +1.50 

N.G.F. Bien qu’hétérogène, le marnocalcaire est, en 

grand, peu perméable. 

Afin de mettre le site hors d’eau et rendre possible 

la circulation, le terrain naturel a été partiellement 

remblayé à la cote + 4.50, laquelle correspond sensi­

blement à celle de la digue de protection des crues de 

la Gironde. 

Les ouvrages à fonder, sur radier enterré, sont calés 

daux niveaux variant entre —13 N.G.F. (station de 
pompage), —7.50 (bâtiment combustible) et —2.50 
(groupe et salle des machines). 

Le parti de fondation adopté : radier, implique une 

substitution complète des couches de vase, toute idée 
de consolidation ne résistant pas à un examen même 
superficiel. Il s’agit donc de réaliser une enceinte des­
cendue au toit des graves (17.5 m de hauteur totale 
de +4.50 à —13), dégageant un rectangle de 
210 x 265 m pour 2 groupes de 900 MW, étanche, 
stable latéralement et vis-à-vis des sous-pressions, qui 
permette d’ériger à sec la station de pompage et, sur 
remblai compacté de 6 à 10 m d’épaisseur, les 
bâtiments réacteur, combustible, massif de groupe, salle 
des machines.
CHOIX D'UNE SOLUTION

La solution de base consiste à faire « comme d'habitude », lorsqu'il y a de la place, une fouille talutée avec un écran étanche périphérique (solution talutée) (fig. 1).

L'écran étanche, en paroi, a 28 m de hauteur totale. Le terrassement des vases est réalisé sous l'eau pour éviter tout glissement des talus à 2/1. Une recharge en sable, également réalisée sous l'eau, fait passer la pente des talus de 2.1 à 3.1. Une couche de tout-venant, déposée à sec, assure une protection contre l'érosion des intempéries. Une telle solution représente 40 à 45 000 m² d'écran, 1 600 000 m³ de terrassement, 300 000 m³ de remblai sous l'eau.

La solution type paroi verticale, dont plusieurs formes sont envisageables, supprime 600 000 m³ d'enlèvement de vase et de remblaiement. Elle écarte toute impasse sur la stabilité des talus, limite le risque d'anomalie d'étanchéité naturelle du fond marnocalcaire (16.5 ha dans la solution de base, 6, seulement, dans la variante), et rapproche de 80 m la station de pompage de la Gironde.

La paroi verticale « classique » se dessine plane avec des tirants forés précontraints (fig. 2). Les efforts étant considérables (17.5 m de poussée des terres et 15.5 de poussée d'eau : un véritable barrage de basse chute), quatre lignes de tirants sont nécessaires. Dès lors, le terrassement est à faire à sec, en ménageant au fur et à mesure les plateformes utiles pour forer les tirants. Cette solution se caractérise de ce fait par des délais voisins du double des délais normaux de la solution de base.

D'où le choix de la solution retenue, avec terrassement par dragage sous l'eau (le terrassement sous l'eau a priori économique équilibrant une paroi à contreforts plus lourde qu'une paroi plane) qui permet de respecter les délais impartis.

DESCRIPTION DE LA SOLUTION

En plan, la paroi s'inscrit dans un rectangle de 275 x 219 m (fig. 3) en bordure immédiate de la digue de protection de la Gironde.

Le schéma d'exécution (fig. 4) que les délais imposent implique que le soutènement soit stable dès le début de la phase 2, c'est-à-dire le dragage. La paroi équilibre les efforts de poussée en s'appuyant en tête sur des tirants et en pied sur le terrain en place. Le moment maximum atteint de ce fait plus de 1 000 tm par mètre linéaire, la paroi est du type à « contrefort » (fig. 5).

Chaque panneau (plot, ou contrefort) a 4.70 m de développement et doit, pour être équilibré, recevoir en tête une réaction horizontale de 5 900 KN. Cette réaction est assurée par quatre tirants inclinés en moyenne à 40° sur l'horizontale et de 1 950 KN de tension unitaire en service (fig. 6).

L'équilibre général et la disposition des couches conduisent à prévoir des tirants d'une longueur totale de 55 m dont 30 m de longueur libre et 25 m de scellement.

La paroi a une hauteur totale moyenne de 31 m, la longueur de fiche (13.5 m) étant intermédiaire entre les résultats des calculs en « butée simple » et en « semi-encastrement ». L'épaisseur normale de 0.80 m est portée à 1 m dans les contreforts pour des raisons essentiellement pratiques : passage des tirants en tête, disposition des armatures dans les zones de recouvrement.
Fig. 3. — Vue en plan de l'enceinte.

Fig. 4. — Schéma d'exécution.

1) Exécution de la paroi et des tirants

Gironde

Paroi moulée à contreforts

3) Pompage de l'eau

2) Terrassement de la fouille par dragage

4) Construction génie civil
L'ensemble du dispositif (fig. 3) est complété par vingt-quatre puits filtrants, situés à l'intérieur de la paroi et profonds de 31,5 m. Utiles pour évacuer le débit de fuite de l'encombre, ces puits sont essentiels pour la stabilité du fond. En effet, si le marnocalcaire est peu perméable en grand, son hétérogénéité structurelle rend imaginaire une succession de couches minces très étanches séparées par des lentilles sableuses très perméables. En supposant qu'une couche étanche se trouve située au voisinage du toit du marnocalcaire (vers la cote — 20 N.G.F. par exemple), elle aurait à supporter toute la sous-pression (15 à 16 t/m²). N'ayant, au-dessus d'elle, que 6 à 7 m de terres (4 à 5 t/m²), elle ne serait pas stable et risquerait de claquer. Sans grande conséquence sur le plan des débits d'exhaure, un tel incident serait par contre catastrophique car il décomprimerait les couches de fondation.

Ainsi, le rôle des puits profonds est d'assurer une répartition régulière des écoulements et des pressions sur l'épaisseur des marnocalcaires.
**ESSAIS PREALABLES SUR LES TIRANTS**

**VUE EN PLAN**

**TIRANTS D'ESSAI**

**PRINCIPE**

Les tirants, inclinés à 40° en moyenne sur l'horizontale, doivent traverser de fortes épaisses de vases pour atteindre les horizons d'ancrage potentiels. Ceux-ci sont constitués par les graves alluvionnaires et par le marnocalcaire sous-jacent très diversifié.

Un premier objet des essais est la définition de la résistance d'ancrage mobilisable dans ces terrains et, corrélativement, la détermination de la longueur de scellement nécessaire.

Un second objet est l'appréciation de la tenue des formations superficielles (remblais) et des vases pour définir le programme des mises en tension des tirants de l'enceinte. L'essai correspondant consiste à solliciter horizontalement une barrette moulée.

**DISPOSITIF D'ESSAI**

Le dispositif d'essai est délibérément implanté dans une des zones les plus défavorables mises en évidence par les reconnaissances préalables à l'est de la future enceinte.

Il comprend (fig. 7) :
- deux barrettes 2.20 x 0.80 m, hautes de 30 m, parallèles, à 6 m l'une de l'autre ;
- un chevêtre en béton armé (7.80 x 2.20 x 1 m) couronnant les barrettes ; pour la deuxième partie des essais, ce chevêtre est conçu pour être ultérieurement coupé en deux, transversalement ;
- deux tirants symétriques, inclinés à 40°, ne différant l'un de l'autre que par la longueur du scellement : 20 ou 25 m ;
- un dispositif de mesure, permettant de mesurer les allongements des tirants et les mouvements tant horizontaux que verticaux du chevêtre (fig. 8), ainsi que les efforts dans les tirants.

Les tirants, du type I.R.P. 12 T 15, ont les caractéristiques suivantes :

Section d'acier : 1 680 mm²
Type d'acier : 153/175 kg/mm²
Allongement : 0.03 mm/ml/t
Résistance minimale garantie à la rupture (R.M.G.) 2 940 kN (294 t)
Tension minimale garantie à 0.1 % (T.M.G.) 2 580 kN (258 t)
Tension extrême de linéarité (P.R.) 2 150 kN (215 t)

Les diagrammes de mise en tension montrent que les deux tirants tiennent instantanément sous une charge de 2 150 kN (1.2 fois la charge de service), que la position du point d'ancrage fictif du tirant nord est correcte, que celle du tirant sud est au-delà au milieu de la longueur scellée, et que la tension critique (critère Cambefort-Chadeisson) s'établit vers 1 900 kN pour ce tirant (fig. 9 et 10).
L’essai de sollicitation horizontale (fig. 11) montre que l’on doit attendre des mouvements importants dès que la vase est sollicitée en butée au-delà de 500 kN environ par mètre linéaire de paroi.

• EXPLOITATION

En premier lieu, on décide d’adopter, pour tous les tirants, une longueur de scellement de 25 m.

En second lieu, on constate que si on précontraint les tirants à leur valeur de service (1 930 kN), et sollicite ainsi en butée les vases à hauteur de 1 250 kN par mètre linéaire de paroi pendant toute la durée de l’exécution, on va engendrer un mouvement notable de la paroi vers la terre, suivi, lors du dragage et de la vidange, d’un mouvement en sens inverse. Cet « aller-retour » n’est pas très satisfaisant pour l’esprit : pertes de tension des tirants puis resollicitations, mouvements différentiels des plots avec cisaillement des joints, n’ont rien d’usuels.

Aussi, on choisit de limiter la tension de blocage à 50 % de la tension de service (c’est-à-dire 1 000 kN par tirant).

Fig. 11. — Essai de sollicitation horizontale barrette nord.

MISE EN TENSION DES TIRANTS

Les tirants sont essayés à 1.2 fois la tension de service en appliquant la méthode du cycle (Recommendations TA 77).

Des processus particuliers sont mis au point afin d’éviter au maximum des différences de sollicitation importantes entre plots voisins. A cette fin, on teste les tirants par paire, à cheval sur deux plots, en n’en sollicitant qu’un à la fois sur chaque plot.

Il importe que les quatre tirants d’un même plot aient sensiblement la même tension, 50 % de la charge de service, mesurée après le blocage de tous. Des essais sont conduits pour mesurer la perte de tension engendrée dans un tirant donné par la mise en service d’un autre du même plot. Cette perte est de l’ordre de 5 à 6 % de la charge de service. Ceci conduit aux valeurs successives de blocage suivantes : 60 % sur le tirant A, 55 % sur le tirant C, 52 % sur le tirant B, enfin 50 % sur le tirant D.

On doit ajouter enfin les essais usuels faits systématiquement tous les n tirants : étude fine du fluage et de la charge critique, essai spécial à 8 000 kN sur quelques plots (pendant quelques minutes).

DISPOSITIF DE CONTROLE

En dehors de tous les contrôles qu’il est d’usage de faire en cours de chantier, et qui constituent les règles de l’Art, un dispositif est mis en œuvre pour suivre paroi, tirants et rabattements pendant les phases cruciales du dragage, de la vidange et du terrassement à sec (fig. 12) :
- des potences métalliques (23) pour les déplacements horizontaux et les rotations en tête de la paroi ;
- des tubes verticaux (14) aux nœuds des contreforts pour mesures clinométriques ;
- des cales dynamométriques (Dynasolf et Glötzl) (33) pour les tensions dans les tirants ;
- un ensemble de points de mesure des niveaux piezométriques et des débits d’exhaure.

Ce dispositif trouve ses justifications en considérant d’une part, les dimensions exceptionnelles du soutènement, d’autre part, le processus particulier de mise en tension des tirants. On s’attend lors du terrassement à noter des déplacements de la paroi vers la fouille, d’une amplitude voisine en tête et en pied, de l’ordre de 6 à 8 cm selon la valeur des sollicitations réelles. Un tel mouvement, compatible avec l’environnement, doit permettre réellement l’obtention de la poussée minimale.
RESULTATS

Les figures 13 et 14 représentent, en fonction de l'avancement des travaux, les mouvements de la paroi et les tensions dans les tirants dans les plots 127 (paroi sud) et 195 (paroi ouest), choisis comme caractéristiques des observations faites sur vingt-trois profils.

On remarque :
- la difficulté que l'on éprouve, sur un chantier, à conduire des mesures dès l'origine des travaux..., on installe le dispositif, telle ou telle manœuvre le met hors de service, on recommence... et on perd les petits mouvements d'origine (1 cm peut-être) ;
- le délai de réflexion de la paroi, qui amorce ses mouvements plusieurs jours après être sollicitée;
- l'influence du dernier mètre terrassé (à sec), qui représente à lui seul une augmentation considérable des efforts et des mouvements (plus de 10 %) ;
- la concordance entre les phénomènes attendus et la réalité observée.

Le figure 15 représente les mouvements maximum de la paroi. Sa forme en « as de carreau » est conforme à la structure : les quatre angles sont des points fixes. La paroi ouest (côté Gironde) a été plus sollicitée que les autres : terrassement à une cote plus basse (— 13.50 NGF), niveau d'eau de la nappe de surface plus élevé (+ 2.50 NGF). Le plot 50 est un cas particulier : un défaut d'étanchéité au droit d'un joint a été traité par injection derrière la paroi et le coulis injecté a poussé un peu trop sur celle-ci.

Les mesures d'inclinaison sont restées dans le domaine de l'imprécision (1/1 000 environ) et n'ont pas révélé des mouvements spectaculaires.

Le système de rabattement-décompression du fond de fouille fonctionne correctement. Les débits mesurés restent nettement inférieurs à 200 m³/h, débit attendu comme normal correspondant à 50 % de la capacité de pompage installé. Les niveaux piézométriques — en particulier un point de mesure au centre de la fouille — sont également très satisfaisants.

COMMENTAIRES

1) Il est, à première vue, assez étonnant que la vidange de la fouille sur 12.50 m n'ait entraîné qu'un faible accroissement de tension des tirants (moins de 10 tonnes) accompagné d'un faible avancement de la paroi (de l'ordre de 8 à 10 mm).
Fig. 13. — Mouvements paroi et tension tirants. Plot 195.

Fig. 14. — Mouvements paroi et tension tirants. Plot 127.

Fig. 15. — Mouvements de la paroi.
L'explication qui paraît la plus simple est de considérer que la paroi est quasiment un solide indéformable (la flexion propre de la paroi correspond à des flèches maximum de l'ordre de 1 cm, qu'on peut négliger ici). Ce solide indéformable est appliqué sur le sol par la mise en traction des tirants, côté extérieur de la fouille (phase 2) (fig. 4).

En fin de phase 1, le coefficient de poussée vaut environ 0,5 sur chaque face de la paroi. En fin de phase 2, ce coefficient a évolué d'une part à l'extérieur (butée sans déformation) vers $K_e = 1$, et d'autre part à l'intérieur (poussée minimale) vers $K_l = 0.25$

Quand on vidange la fouille (phase 3), on met en mouvement la paroi vers l'intérieur, $K_e$ glisse vers 0,25 valeur de la poussée : le $K_l$, balance à peu près la pression de l'eau qui vient d'être vidangée, d'où une faible augmentation de la force du tirant.

Cette analyse conduit à penser que l'on passe d'un équilibre de butée à l'équilibre de poussée au prix d'un très faible mouvement.

2) Le mouvement de la tête de la paroi et l'accroissement de la force dans le tirant sont en relation théoriquement linéaire.

Dans 50 % des profils étudiés, on vérifie aisément que la relation est conforme aux paramètres (longueur libre du tirant, caractéristiques de déformation de l'acier en fonction de sa tension).

Dans 50 %, on ne retrouve pas clairement cette relation, la traction dans le tirant restant en deçà.

L'explication de ce phénomène peut venir soit d'influence au niveau des têtes, soit d'un mouvement vertical descendant de la paroi. Nous ne possédons pas de mesures de cet éventuel mouvement, qui n'est pas impossible compte tenu des efforts engendrés.

3) En conclusion, nous souhaitons mettre en lumière l'augmentation de sécurité, sur la sollicitation réelle des tirants, que le processus adopté a offert. À notre connaissance, il s'agit du premier exemple, à cette échelle, d'un tel parti de construction et nous pensons qu'il doit pouvoir s'appliquer chaque fois qu'on est en présence d'une structure de soutènement simple dont les mouvements éventuels sont compatibles avec l'environnement.
A METHOD TO PREDICT THE LOAD-DISPLACEMENT RELATIONSHIP OF GROUND ANCHORS

Modèle pour calculer la relation charge-déplacement des ancrages dans les sols

by

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SOMMAIRE

Des mesures de contrainte effectuées pendant des essais d'ancrage ont montré que la relation entre la contrainte de frottement superficiel du bulbe d'ancrage et le déplacement de celui-ci est non lineaire et que la distribution de la force axiale du bulbe d'ancrage s'étend graduellement vers l'extrémité lorsque la charge augmente.

Afin de donner une explication à ce comportement de l'ancrage dans le sol, nous avons proposé une relation charge-déplacement de l'ancrage en supposant un modèle élastoplastique de relation entre la contrainte de frottement superficiel et le déplacement à un endroit quelconque sur le bulbe d'ancrage. En outre, afin d'appliquer cette formule aux cas réels, nous avons analysé de nombreuses données d'essais d'ancrage et nous avons examiné la relation charge-déplacement, la distribution de la force axiale, et les différentes constantes du sol comprises dans la formule. Nous avons pu ainsi trouver une relation entre les constantes du sol et la valeur N obtenue lors d'un essai de pénétration standard (S.P.T.).

Par conséquent, si la valeur N est donnée comme un résultat de reconnaissance du sol, il sera possible de prévoir la relation charge-déplacement de l'ancrage.

SUMMARY

According to results from measurements of stress obtained by tests of the ground anchor, the relation between skin frictional stress of the anchorage bulb and its displacement is non-linear, and the axial force distribution range of the anchorage bulb expands gradually towards the tip due to increase of the load.

In order to explain such behavior of the ground anchor, we have proposed a formula, which gives the load-displacement relationship of the ground anchor, by assuming an elasto-plastic model of skin frictional stress-displacement relation at a point on the anchorage bulb. Moreover, in order to apply this formula to actual cases, we have analyzed numerous data from anchor tests conducted on the site and we have examined the load-displacement relationship, the distribution of axial force, and various ground constants included in the formula. As a result, we have found out a mutual relationship between the ground constants and the N value of the standard penetration test.

Therefore, if N value is given as result of the soil investigation, it will be possible to predict the load-displacement relationship of the ground anchor.

INTRODUCTION

Accurate prediction of the load-displacement relationship is important for the design and construction of ground anchors in order to ensure the safety and economical efficiency of the structures utilizing the anchors.

Accordingly, in order to put forward a method for predicting this load-displacement relationship, studies were carried out; firstly, to find and formulate an analytical method that could adequately account for the behavior of the anchor; secondly, to find a method of estimating the ground characteristic values used in the formula from ground investigation data; and, thirdly, to confirm that the computed values obtained by means of the above prediction method are accurate enough for practical usage when compared with the measured values.

It is recognized, from considerable measurement data, that, unlike conditions in the conventional design methods hitherto employed, the distribution of anchor resistance against the external force is not uniform throughout the entire anchorage bulb, but that when the load applied to the anchor is light most of the load is conveyed to the top part of the anchorage bulb and is not transmitted efficiently as far as the tip part. In other words, as the load increases, the range of distribution of the axial force in the anchorage bulb spreads gradually from the top part of the anchorage bulb towards the tip part. Therefore, concerning the length of anchorage bulb that effectively functions as a resistance to the pulling force, it is necessary to conclude that there exists a critical length that corresponds to the stiffness of the anchorage bulb itself and the ground characteristics.
ANALYTIC MODEL AND MATRIX FORMULATION

As shown in fig. 1, the anchor is divided into a finite number of elements, and the nodal point of each element is supported by elasto-plastic spring of the ground.

The linear stiffness equation of the displacement method usually employed shown by the following equation.

\[ [\mathbf{D}] [\mathbf{K}] [\mathbf{D}]^T \{ \ddot{\mathbf{s}} \} = \{ \mathbf{P} \} \]  \hspace{1cm} (1)

where,

\[ [\mathbf{D}] [\mathbf{K}] [\mathbf{D}]^T: \text{stiffness matrix}; \]
\[ \{ \ddot{\mathbf{s}} \}: \text{displacement vector}; \]
\[ \{ \mathbf{P} \}: \text{load vector}. \]

The skin frictional stress \( \tau \) of the anchorage bulb, taken as proportional to axial displacement \( \delta \) raised to the \( m \)th power, can be expressed as

\[ \tau = C_s \delta^m \]  \hspace{1cm} (2)

where,

\( \tau \): skin frictional stress (kg/cm\(^2\));
\( C_s \): coefficient of skin frictional stress (kg/cm\(^{m+2}\));
\( \delta \): axial displacement of anchor (cm);
\( m \): constant.

The ground reaction \( R_i \) along the anchor axis at nodal point \( i \) can be written

\[ R_i = -K_{si}\delta^m \]  \hspace{1cm} (3)

where,

\( R_i \): ground reaction along anchor axis at nodal point \( i \) (kg);
\( K_{si} \): coefficient of elasto-plastic spring = \( 1/2 C_s U (L_{i-1} + L_i) \) (kg/cm\(^m\));
\( L_i \): length of anchor element \( i \) (cm);
\( U \): circumference of anchorage bulb (cm).

Addition of equation (3) to the load vector \( \{ \mathbf{P} \} \) of equation (1) gives

\[ [\mathbf{D}] [\mathbf{K}] [\mathbf{D}]^T \{ \ddot{\mathbf{s}} \} + [K_{si}] \{ \delta^m \} = \{ \mathbf{P} \} \]  \hspace{1cm} (4)

Equation (4) is the stiffness equation of the analytic model shown in fig. 1, but constant \( m \) has to be determined for actual anchor analysis. Here it is assumed that the following equation of Yamakado & Muta proposed for the skin frictional stress-displacement relationship for a pile will also hold true for an anchor (fig. 2).

\[ \delta < d, \tau = C_s \delta^{0.5} \]
\[ \delta \geq d, \tau = \tau_{\text{max}} = \text{const}. \]  \hspace{1cm} (5)

where,

\( d \): displacement at yield point (cm);
\( \tau_{\text{max}} \): maximum skin frictional stress (kg/cm\(^2\)).

From this assumption, we write equation (4) as follows:

\[ [\mathbf{D}] [\mathbf{K}] [\mathbf{D}]^T \{ \ddot{\mathbf{s}} \} + [K_{si}] \{ \delta^{0.5} \} = \{ \mathbf{P} \} \]  \hspace{1cm} (6)

Numerical analysis of equation (6), in which a nonlinear term is included in the left hand second term, has already been used generally, and so will not be discussed afresh here.
DETERMINATION OF GROUND CHARACTERISTIC VALUES AND LENGTH OF ANCHORAGE BULB

Material characteristics of the anchor can be found by tests conducted on the steel and cement materials used. Usually, however, the actual dimensions of an anchorage bulb constructed in the ground differ from the expected ones, and measurements are impossible without completely extracting or excavating the anchor.

Moreover, measurement of the soil characteristics of the actual ground in which the anchor is being constructed is difficult with the existing measurement techniques.

Accordingly, we propose an indirectly estimating method for coefficient of skin frictional stress of the ground and the effective length of anchorage bulb from the measured load-displacement curve, because ground characteristic values and actual effective anchorage bulb dimensions are great factors for analyzing anchor behavior.

Measured load-displacement curves are approximated by

\[ P_0 = K_0 \frac{G_0}{n} \]  

(7)

where,

- \( P_0 \): load;
- \( G_0 \): displacement at anchor head;
- \( K_0 \): constant;
- \( n \): constant.

Constants \( K_0 \) and \( n \) of equation (7) are considered as functions of coefficient of skin frictional stress \( C_s \), ratio \( r \) of anchor length \( L_a \) to effective length of anchorage bulb \( L_b \), and elastic modulus \( E \) and cross-sectional area \( A \) of the anchor material.

If, as stated, \( E \) and \( A \) are considered known quantities, we can consider constants \( K_0 \) and \( n \) as functions only of \( C_s \) and \( r \), and the following equation becomes applicable.

\[ K_0 = f_1(C_s, r) \]
\[ n = f_2(C_s, r) \]  

(8)

where,

- \( r \) : \( L_b / L_a (0 < r \leq 1) \), which will be termed ratio of effective length;
- \( L_a \) : anchor length = \( L_f + L_b \);
- \( L_f \) : effective free length of anchor;
- \( L_b \) : effective length of anchorage bulb.

GROUND CHARACTERISTIC VALUES

Ground characteristic values were obtained by analysing the test data of thirty ground anchors according to the above method the tests being carried out at the actual construction site. The ground anchors used for the analyses were all of the cement mortar (or paste) grouted type without reaming.

Fig. 4 and fig. 5 are the results of analyses of coefficient of skin frictional stress \( C_s \) and displacement at yield point \( d \) when the subject ground was dealt with as if it was a single-stratum, made to correspond with mean value \( \bar{N} \) of the \( N \) values of the ground around the anchorage bulb obtained by means of standard penetration tests.

In fig. 6, \( C_s \) of fig. 4 and \( d \) of fig. 5 are used to calculate by means of equation (5) the maximum frictional stress \( \tau_{max} \), which is made to correspond with mean \( N \) value \( \bar{N} \).

Further, the following equation was used to calculate the mean \( N \) value \( \bar{N} \) of multi-stratum ground, and to deal with the ground if it was a single-stratum.

\[ \bar{N} = \frac{\sum \alpha_i N_i}{L_b} \]  

(9)

where,

- \( \bar{N} \): mean \( N \) value for single-stratum ground;
- \( \alpha_i \): thickness of stratum \( i \);
- \( N_i \): \( N \) value of stratum \( i \).
The following conclusions were obtained from the results of the above analyses.

(1) By following the method described herein, it is possible to estimate the coefficient of frictional stress $C_s$ and effective anchorage bulb length $L_b$ from the measured load-displacement curve.

(2) In the expedient method for estimating the effective free length of the anchor from the load $P_o$-elastic displacement $\delta_{p_e}$ curve obtained by deducting from the measured displacement $\delta$ the residual plastic displacement $\delta_{p_r}$ at zero-load time when the load is removed, the estimate should be according to the slope of the initial linear zone of the load-elastic displacement curve (see fig. 7).

(3) There exists a critical length, for the effective anchorage bulb length of the ground anchor and even if the length is made greater than this, no evident increase in pulling resistance can be expected (see fig. 8).

(4) The coefficient of skin frictional stress $C_s$, displacement at yield point $d$, and maximum skin frictional stress $\tau_{max}$ for a single stratum can be estimated, using the mean N value $\bar{N}$ obtained from equation (9), by means of the following equation.

$$C_s = 0.114 \bar{N} - 0.508$$

$$\tau_{max} = 0.0584 \bar{N} + 0.546$$

$$d = (\tau_{max}/C_s)^2$$

CONCLUSIONS
Fig. 8. — Effect of anchorage bulb length $L_b$ on the load $P_c$-displacement $\delta$ relationship at anchorage bulb top (test data No. 2).

Five observations were made at different lengths of anchorage bulb. The results are shown in the figure. It is evident that as the length of the anchorage bulb increases, the load-displacement relationship becomes stiffer.

Fig. 9. — Comparison of measured load $P$ with predicted load $P'$, at yield point of load-displacement curve plotted on log-log scale.

(a) Good example, test data No. 2.
(b) Poor example, test data No. 8.

Fig. 10. — Comparison of analytical result of the load $P_c$-displacement $\delta$, curve with measured values.

5) By use of the ground constants calculated by means of equation (10), the load-displacement relationship calculated with equation (6) is sufficiently accurate for practical application (see fig. 9 and fig. 10).

AFTERWORD

It has been definitely shown by the above that this method of analysis is capable of fully describing the behavior of a ground anchor, and is an effective method in practice, too.

$N$ values of standard penetration tests in ground investigation data accompanying existing anchor test data were used for estimations of the ground characteristic values required for actual application of this method. However, it is considered insufficient to use $N$ values for this, so suitable ground investigation methods need to be developed.

Considerable information was gained in the course of this research but this will be presented in other papers.

REFERENCE

DÉTERMINATION DE LA LONGUEUR LIBRE OPTIMALE D'UN TIRANT D'ANCRAGE SOUTENANT UNE PAROI

Determination of the optimal free length of the line of an anchor supporting a wall

par

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SOMMAIRE

La longueur libre optimale des tirants d'ancrage d'une paroi verticale soutenant un massif de sable sec à surface libre horizontale a été déterminée par des essais en modèles réduits avec des matériaux pulvérulents d'angle de frottement interne \( \varphi_1 = 31° \pm 1° \) et \( \varphi_2 = 24° \pm 1° \). La similitude ne cherchait pas à représenter les déformations élastiques, mais simplement la rupture plastique. Une règle simple a pu être dégagée pour la longueur libre d'un tirant isolé ou pour celle des tirants d'une ligne horizontale unique d'ancrages.

SUMMARY

Scale models were used in order to find the optimal length of the lines of anchors for vertical walls retaining a mass of dry sand with a horizontal surface using granular materials having an internal angle of friction \( \varphi_1 = 31° \pm 1° \) and \( \varphi_2 = 24° \pm 1° \). The similitude did not aim at a representation of elastic deformation but only of plastic failure. A simple rule was found for the free length in the case of either a single anchor or a single horizontal line of anchors.

1. INTRODUCTION

La longueur libre des tirants d'ancrage des murs de soutènement n'est pas déterminée actuellement d'une façon très rigoureuse. Les différents règlements, codes de construction ou recommandations (1) proposent des méthodes de calcul peu satisfaisantes ou des règles empiriques que les maîtres d'ouvrage ont tendance à considérer comme des minimums. Pourtant, il n'y a pas intérêt à avoir des tirants trop longs : d'une part ils sont plus coûteux et d'autre part ils introduisent inutilement une plus grande souplesse en tête des tirants. Il est donc souhaitable de définir avec plus de précision la longueur libre optimale des tirants.

Prenons l'exemple d'un mur vertical soutenant un massif à surface libre horizontale et d'une rangée de tirants horizontaux destinée à s'opposer au renverse-

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ment du mur. Si les tirants sont ancrés à l'intérieur du coin de Coulomb, c'est-à-dire à l'intérieur du dièdre formé par le mur et par un plan incliné à \( \frac{\pi}{4} - \frac{\varphi}{2} \) sur la verticale et issu du pied du mur, leur efficacité est nulle. S'ils sont ancrés au-delà du plan faisant un angle \( \varphi \) avec l'horizontale, c'est-à-dire au-delà du talus naturellement stable, leur efficacité ne doit pas croître beaucoup pour des tirants plus longs. L'efficacité des tirants en fonction de leur longueur (fig. 1) apparaît donc comme une courbe tangente à l'origine à l'axe des abscisses et ayant une asymptote horizontale : elle possède donc un point d'inflexion qui peut être considéré comme donnant une longueur caractéristique. Il reste à définir la notion d'efficacité des tirants et à vérifier si la longueur caractéristique ainsi obtenue correspond à un changement qualitatif net de l'efficacité des tirants.

Nous nous proposons dans cette étude d'apporter une réponse expérimentale à cette question au moyen d'essais sur des modèles réduits. Les expériences ont été menées en parallèle au Laboratoire de Mécanique des Solides (LMS) et au Centre Expérimental du Bâtiment et des Travaux Publics (CEBTP) avec des matériaux pulvérulents différents pour obtenir des résultats qui puissent se compléter.

2. PRINCIPE DES ESSAIS

On a utilisé de simples modèles réduits en sable. L'échelle des longueurs étant égale à l'échelle des contraintes et les modules d'élasticité variant d'une façon non linéaire avec les contraintes, il n'y a pas de correspondance simple entre les déformations de la maquette et celles du prototype, c'est-à-dire un mur soutenant un massif du même sable. En conséquence, les petits déplacements qui précèdent la rupture des modèles réduits sont des phénomènes utiles à l'interprétation des essais, mais qui ne permettent en aucun cas de prévoir l'amplitude des mouvements du prototype. Ces maquettes doivent être considérées comme utilisant une similitude volontairement très pauvre, limitée au domaine plastique et à la rupture, à l'exclusion des petites déformations du domaine élastique ou du domaine linéaire.

Il faut définir une mesure de l'efficacité d'un tirant. Soit \( h_o \) la hauteur libre limite à partir de laquelle un
mur autostable s'effondre par renversement (fig. 2) et \( h \) la hauteur libre maximale du même mur, mais soutenu par un tirant. Le rapport \( \frac{h}{h_o} \) permet de définir une mesure de l'efficacité. En particulier \( \frac{h}{h_o} = 1 \) correspond à l'efficacité nulle. Le choix des tirants horizontaux se justifie pour obtenir une résistance du dispositif d'ancrage constante quelle que soit la longueur du tirant. Ce dispositif est constitué par un disque fixé à l'extrémité d'une corde à piano recouverte d'un souploisso. Le souploisso protège la tige du tirant du frottement du sable. Le disque symbolise le scellement des empilements de rouleaux selon la méthode de Schneebeli-Dantu, qu'un ancrage par plaque donne des résultats qualitativement comparables à un ancrage par frottement (fig. 3). Il est évident que la résistance d'un tel ancrage pour un tirant incliné aurait varié avec la longueur du tirant, c'est-à-dire avec la profondeur de l'ancrage, obligeant à modifier le disque et compliquant inutilement les essais.

Par un calcul de poussé classique, on a évalué sommairement la force du tirant permettant de retenir le mur, en fonction de la hauteur d'accrochage. Le tirant n'a pas été placé trop près de la surface du sable afin que l'ancrage soit obtenu avec un disque de rayon suffisamment petit pour que la perturbation du massif autour de l'ancrage ne soit pas trop affectée par le gradient de pesanteur. Il fallait cependant que la résistance de l'ancrage soit suffisamment grande pour que \( h \) soit nettement différente de \( h_o \), et que l'expérience soit sensible, mais aussi il fallait qu'elle ne soit pas trop grande et que la rupture se produise toujours par basculement du mur et non par chassement du pied : un changement de mode de rupture aurait en effet rendu l'interprétation des essais complètement illusoire. La profondeur et la force de l'ancrage ont été ainsi choisies par le calcul. La résistance du dispositif d'ancrage a été vérifiée par des essais d'arrachement en milieu indéfini et en adaptant le diamètre du disque par interpolation pour obtenir la résistance désirée. Enfin, au cours de chaque essai, les déplacements du mur ont été suivis de façon à s'assurer que le mouvement général restait un renversement du mur, même si le centre instantané de rotation était plus profond pour les murs ancrés que pour le mur autostable.

3. MODE OPERATOIRE

Les essais ont été menés de la façon suivante.

Le mur de soutènement est une plaque carrée rigide qui est mise en place et calée à droite et à gauche. Le sable est déposé en couches, par chute libre à hauteur constante à partir d'un tamis, symétriquement de chaque côté du mur et jusqu'aux deux tiers de la hauteur. Le ou les tirants munis de leurs plaques d'ancrage sont mis en place horizontalement. Le remplissage continue alors côté amont jusqu'à la hauteur du bord du mur. Les cales sont alors ôtées et on simule le creusement d'une fouille en enlevant le sable côté aval centimètre par centimètre, en notant les déplacements correspondants. Au début, les déplacements sont insensibles ; on peut considérer que la rupture est généralisée et que des changements de géométrie se sont produit lorsque la rotation du mur a atteint 10°. La courbe hauteur libre devant le mur a atteint 10°. La courbe hauteur libre devant le mur, en fonction de la hauteur d'accrochage. Le rapport d'un tel ancrage pour un tirant incliné aurait varié avec la longueur du tirant, c'est-à-dire avec la profondeur de l'ancrage, obligeant à modifier le disque et compliquant inutilement les essais.

Dans les essais du CEBTP, on a mesuré les traînons dans les tirants au moyen d'un anneau dynamométrique de 2 cm de diamètre intérieur formant rotule et placé à l'extérieur du mur, traversé diamétralement par le tirant et muni d'extensomètres à résistance électrique. Ce dispositif a permis de tracer la courbe des efforts dans le tirant en fonction du déplacement de l'écran, en plus de la courbe des déplacements de l'écran en fonction de la hauteur de la fouille. De même, ce dispositif a permis de mettre en évidence l'évolution de la tension dans un tirant initialement précontraint.

- Dimension de l'écran : 50 cm × 50 cm.

Des fentes verticales dans l'écran permettaient de placer le ou les tirants à la hauteur voulu sans risque d'encastrement de la tige, ou des tiges, dans le mur. Les fentes intuiliées étaient obstruées.

- Dimensions de la cuve d'essais : canal de 51 cm de large, de 2 m de long, de 0.70 m de profondeur. Le sable de billes de verre avait 50 cm de longueur, 25 cm de diamètre intérieur formant 2 cm de diamètre intérieur formant disque en laiton de 1 mm d'épaisseur et de 1 cm ou 1.2 cm ou 5 cm de diamètre selon les cas.

- Dimensions de la cuve d'essais : canal de 51 cm de large, de 2 m de long, de 0.70 m de profondeur. Du fait du caractère sphérique des grains et de leur état de surface très lisse, la variation de volume en cours de cisaillement est pratiquement nulle. L'angle de frottement interne correspondant est \( \varphi = 24^\circ \).

- Dimension des tirants : 1 mm.

- Ancrages : disques en plexiglas de 1 mm d'épaisseur et de 2 cm de diamètre.

- Dimensions de l'écran : 25 cm × 25 cm avec fentes pour placer le ou les tirants aux positions voulues.

- Dimensions de la cuve d'essais : le massif de sable de billes de verre avait 50 cm de longueur, 25 cm de largeur, 30 cm de profondeur (soit 5 cm de sable sous le mur).
4. INTERPRETATION DES ESSAIS

A partir de la courbe du déplacement du mur en fonction de la hauteur de la fouille, on peut obtenir différents critères de rupture comme on le fait habituellement avec les courbes efforts-déformations. On peut définir la rupture par l'asymptote du déplacement, par un déplacement arbitraire, par un rapport d'affinité des différentes courbes, etc., quoi qu'il en soit les courbes de la figure 4 montrent nettement l'influence de la longueur du tirant. Les courbes A et A' représentent deux essais identiques et donnent une idée de la dispersion des résultats qui est faible. La courbe B correspond à un tirant précontraint ; elle est à comparer à la courbe C d'un tirant de même longueur non précontraint. On voit que les petites déformations sont influencées et réduites par la précontrainte, mais que la résistance à la rupture n'est pratiquement pas affectée. Ce résultat, qui est classique pour un corps de Tresca ($\phi = 0$) élastoplastique semble pouvoir s'étendre aux structures (mur + tirant) dans un sol pulvérulent. Le tableau ci-dessous, interprété à partir des courbes de la figure 4, donne les résultats des essais d'un tirant unique, muni d'un disque d'ancrage de 3 cm de diamètre, en fonction de sa longueur, pour le sable de Fontainebleau, ainsi que la traction maximale dans le tirant.

TABLEAU I

<table>
<thead>
<tr>
<th>Longueur (cm)</th>
<th>Dénivellation (cm)</th>
<th>Traction (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>$h_0 = 35.6$</td>
<td>(mur autostable)</td>
</tr>
<tr>
<td>30</td>
<td>38.6</td>
<td>17.5</td>
</tr>
<tr>
<td>35</td>
<td>39.3</td>
<td>20.3</td>
</tr>
<tr>
<td>40</td>
<td>45.0</td>
<td>39.1</td>
</tr>
<tr>
<td>50</td>
<td>42.5</td>
<td>non</td>
</tr>
<tr>
<td>50</td>
<td>45.0</td>
<td>mesurée</td>
</tr>
</tbody>
</table>

La figure 5 représente la courbe de l'efficacité du tirant en fonction de sa longueur. La précision des résultats ne permet pas de placer exactement le point d'inflexion de la courbe ; on voit néanmoins que...
L'efficacité maximale est déjà atteinte lorsque la longueur du tirant dépasse 40 cm.

La figure 6 représente la courbe analogue de l'efficacité du tirant unique dans un massif de billes de verre, en fonction de la longueur du tirant. L'efficacité maximale est atteinte lorsque la longueur dépasse 20 cm.

Les essais précédents étudiaient le comportement d'un élément de mur soutenu par un seul tirant. Des essais ont été effectués avec une ligne horizontale de plusieurs tirants de façon à examiner l'influence du voisinage des tirants sur la longueur optimale, ce qui est plus réaliste si l'on veut prévoir ce qui peut se produire pour les structures les plus courantes. Le tableau ci-dessous donne les résultats des essais dans du sable de Fontainebleau avec une rangée de cinq tirants munis de disque d'ancrage de 1.2 cm de diamètre (donnant à peu près la même résistance totale que le tirant unique précédent), espacés de 10 cm ; les tirants marginaux étaient placés à 5 cm des bords.
TABLEAU II
Nappe de cinq tirants dans du sable de Fontainebleau.
Les chiffres indiqués dans ce tableau correspondent à un état de rupture généralisée obtenu après une rotation de l'ordre de 2.5°. Le phénomène de rupture est donc beaucoup plus net que dans le cas du tirant isolé.

<table>
<thead>
<tr>
<th>Longueur (cm)</th>
<th>Dénivellation (cm)</th>
<th>Traction dans le tirant central (N)</th>
<th>Traction dans un tirant latéral (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>40.0</td>
<td>4.9</td>
<td>4.4</td>
</tr>
<tr>
<td>39</td>
<td>38.6</td>
<td>8.4</td>
<td>8.1</td>
</tr>
<tr>
<td>45</td>
<td>46.0</td>
<td>10.8</td>
<td>non mesurée</td>
</tr>
</tbody>
</table>

La figure 7 représente la courbe de l'efficacité des tirants en fonction de leur longueur ; on voit que l'efficacité est maximale lorsque la longueur dépasse 45 cm.

La figure 8 représente une courbe analogue de l'efficacité d'une nappe de cinq tirants dans un massif de billes de verre, en fonction de leur longueur. L'efficacité maximale est atteinte lorsque la longueur dépasse 22.5 cm.

Si l'on joint la base du mur et les points où l'on a obtenu un ancrage d'efficacité maximale on peut calculer les angles minimaux suivants avec l'horizontale :

- pour le sable de Fontainebleau :
  tirant isolé .................................................. $\alpha = 40^\circ$
  nappe de tirants ............................................... $\alpha = 37^\circ$

- pour le sable de billes de verre :
  tirant isolé .................................................. $\alpha = 40^\circ$
  nappe de tirants ............................................... $\alpha = 37^\circ$

CONCLUSION

Les résultats obtenus dans deux séries d'essais complémentaires avec des matériaux distincts ayant notamment des angles de frottement interne très différents donnent des longueurs libres optimales des tirants d'ancrage très voisines. Ce résultat n'est pas étonnant dans la mesure où les formes des surfaces de glissement des structures (mur-sol pesant-tirants) sont voisines lorsque les ruptures se produisent par déversement du mur. On remarque en particulier que la longueur optimale des tirants est plus grande pour les groupes de tirants que pour les tirants isolés, ce qui était prévisible et qui est une manifestation de l'interférence des ancrages. La règle d'avant-projet qui consiste à placer le scellement des ancrages au-delà d'une ligne à 45° issue du pied du mur se trouve ainsi vérifiée, mais cette conclusion n'est confirmée par la présente étude que pour des massifs de terre à surface libre horizontale, avec parement vertical, sans force intérieure ou extérieure inclinée comme pourraient l'être, par exemple, les pressions de courant ou des sollicitations dynamiques.
BASE CALCULATION OF ANCHOR FOUNDATIONS USING APPROXIMATE MODEL TESTING

Calcul des fondations d'ancrage à l'aide des modèles approximatifs

by

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SOMMAIRE
Cette communication présente les résultats d'une similitude particulière dite des modèles approximatifs. Des modèles géométriquement semblables pour des ancrages, pour des ancrages champignons, pour des plaques enterrées placées dans le sol réel ont été réalisés pour étudier la capacité portante des ancrages. La loi de la force limite et celle du déplacement limite en fonction de l'échelle a été dégagée et extrapolée jusqu'au réel.

Cette méthode permet l'élaboration des abaques en coordonnées adimensionnelles répétées par les résultats de nombreux essais sur modèles à diverses échelles. Une telle approche minimise le nombre des essais en vraie grandeur.

SUMMARY
The conditions of approximate model testing are used to determine the bearing capacity and deformity of anchor foundations bases. The idea of approximate model testing is to model geometrical dimensions of foundations keeping dynamic similarity, the soil being the same both for model and the prototype. The models of anchor foundation structures such as slabs, inclined piles, cross-bar cylinders, mushroom-like and bored anchors were tested and the resultant data were represented in non-dimensional coordinates. The empirical method of the calculation of the anchor bearing capacity and deformity should be used.

This method offers the possibility of plotting coincidence diagrams in terms of non-dimensional coordinates after conducting several model tests in various scales. Such an approach minimizes the volume of full scale tests.

It is rather difficult to make analytical calculations of bearing capacities and deformations of anchor foundation bases, owing to the absence of axial symmetry and non-linear deformations.

That is why it is advisable to use an approximate modeling in order to determine the bearing capacity and anchor foundation deformity. In the 50 s the by D.E. Polshin. According to the concept, geometrical dimensions of the foundation were modelled with the dynamic similarity kept, and the soils the same both for the model and the prototype [1].

The systemizing of test results is done in non-dimensional coordinates which gives the opportunity to minimize and sometimes to eliminate the effect of the model on the final results determining the bearing capacity and natural foundation base deformation [2], [3], [4].

The models of the anchor foundation parts such as slabs, inclined piles, cross-bar cylinders, mushroom-like and bored anchors were tested and the resultant data were represented in non-dimensional coordinates [5], [6], [7], [8]. The methods of each model loading were taken to be equal according to the requirements of the kinematic and dynamic similarity. The anchors were subjected to pulling force step by step and were kept till stabilization of displacement in the point of load application was achieved.

During every model test no less than 10 experimental values for the diagram «load-displacement» were obtained as the steps of loading were nearly 1/10 of their maximum value. The tests were considered finished when either a sudden loss in bearing capacity with soil deformation or increasing displacement took place without additional loading.

The last loading step when the anchor displacement stabilization set in was taken as a maximum \( N_{\text{lim}} \), while the displacement \( U_{\text{lim}} \) corresponded to this loading. The loading \( N_{\text{lim}} \) was prior to the maximum value of pulling force.

The test results of different anchor foundation structures in sand and clay soils indicated that the dependence «load-displacement» is non-linear as the compaction and displacement deformation of base soil takes place simultaneously, one of the types predominating at the beginning and at the end of loading.

The «load-displacement» curves for different models and prototypes have in absolute values quite a different character, without giving the opportunity to generalize the dependence function for the «base-foundation» system. The diagram of the above dependencies in absolute values by test data does not provide their true significance even qualitatively. Thus, for instance, diagram curvature depends on the choice of scales of displacement and loading values. The natural dispersing of deformation experimental values for this or that
load in repeated tests influences the diagram curvature as well. The estimation of such diagrams, taking into account the variety of soil conditions, becomes more difficult for foundation models and prototypes of various constructions and dimensions.

When the diagram of «load-displacement» dependence is compiled in relative values, we have quite another picture. The ratio of acting loads \( N_i \) to their maximum value \( N_{\text{lim}} \) marked on the abscissa axis in the unique system of non-dimensional coordinates and the corresponding relations between \( U_i \) and \( U_{\text{lim}} \) displacements are marked on the ordinate axis. Such a systemizing of test results makes it possible to compare the dependences for anchor foundations of different scale and to get equations of relative parameter connections.

\[
\frac{U_i}{U_{\text{lim}}} = \frac{N_i}{N_{\text{lim}}} \quad \text{or} \quad C_i = f(C_i^o) \quad (1)
\]

The tests diagrams in non-dimensional coordinate were plotted in accordance with the anchor foundation structures in the scale range from 1:10 to 1:1 according to the tests in sand and clay soils (fig. 1, 2). The analysis of the diagrams showed their characteristics as follows:

1. The diagrams are characterized by convexity which shows the non-linear nature of anchor foundation base deformation and the growth of displacements during the next steps of load application.

2. The vague diagram curvature shows the foundation properties: the abrupt curve corresponds to shallow anchor foundations, soft soils having low mechanical characteristics; the flat curving corresponds to deeper foundation bedding and compact soils with much better mechanical characteristics.

3. The diagrams remain just the same for anchor foundation models both over the whole scale range and in a part of the range and this opens wide opportunities for extrapolation of test results.

4. The diagram curvature is not affected by the dimensions and the shape of foundations as well as the angle of pull-out, though these parameters influence the absolute values of maximum displacements and efforts.

5. The comparison of test results of pulling out rammed piles and cylindrical anchors put into leading wells shows that the curvature of «load-displacement» non-dimensional diagram is more abrupt in the first case than in the second one.

The accuracy of base calculations under approximate model testing conditions depends greatly on the definition of the foundation displacement maximum values. It is reasonable therefore to lessen the loading steps before foundation bearing capacity limit is achieved.

Fig. 1. — Average «load-displacement» dependence in approximate values for anchor foundations: a) right-angled inclined slabs with 80 x 500 mm in average-sized sand; b) cross-bar cylinders when being piled into the average-sized sand; c) round slabs in sand of average size and density; d) inclined piles in fine sand of average density and in hard dusty sandy loam; e) cross-bar cylinders when using the principle method of boring into the sand of average size and density; f) mushroom-like foundations in sand of average size and density and in a hard clay.
As the summarizing of available data shows, one must be sure that the test is conducted properly in order to minimize the dispersing of results, i.e. to see if all the adopted methods of model testing and the accuracy of effort and displacement measurements are observed carefully till the limit of foundation bearing capacity is achieved. The soil for model testing must have similar physical and mechanical characteristics. Fig. 3, 4, 5, 6, show diagrams of maximum values of stabilized pulling forces $N_{\text{lim}}$ and the corresponding displacements $U_{\text{lim}}$ depending on anchor model scale.

Dependence $N_{\text{lim}} = f(S)$ is curved and has power function character, but the dependence $U_{\text{lim}} = f(S)$ is almost linear, which simplifies the extrapolation of the foundation model test results.

This almost linear character of the dependence of maximum stabilized displacement followed by pulling out the anchors will remain the same even for larger prototypes. Such a conclusion is based on the thought that anchor soil base lies above the footing. «Load-scale» dependences are also convenient to represent in a unique system of non-dimensional coordinates for all the anchor foundation structures. The scale ratios or ratios of linear dimensions of prototypes and models are marked on the abscissa axis and corresponding ratios of their maximum loads are marked on the ordinate axis. Any foundation scale, natural dimensions included, can be taken as a single one in this coordinate system, other scales being prototypes.

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Fig. 4. — "Model scale-maximum displacement" for sloped boring anchors in soft sandy morainic loam.

Fig. 5. — Average "model scale-maximum displacement" dependences for anchor foundations. 1) cross-bar cylinders in sand; 2) cross-bar cylinders in morainic sandy loam; 3) inclined single piles in morainic sandy loam; 4) mushroom-like foundations in sand.

Dependences \( \frac{N_{\text{prot}}}{N_{\text{lim}}} = f(C_t) \) are represented in fig. 7, 8.

Fig. 6. — "Model scale-maximum displacement" for inclined bored anchors in soft sandy morainic loam.

Fig. 7. — Average "model scale-load" dependences calculated in approximate values for anchor foundations of different constructions (curved diagrams are compiled according to the equation 2 with \( n \) having different values).
Fig. 8. — «Model scale-loading» dependences in approximate values for inclined bored anchors (curved diagram is compiled according to the equation 2 where \( n = 2.6 \)).

Systemizing the obtained results it is possible to approximate them in the following expressions:

\[
\frac{N_{\text{prot}}}{N_{\text{lim}}} = C_L \quad (2)
\]

where \( C_L \) is the ratio of linear dimensions or prototype and model scales; \( n \) is the coefficient depending on anchor foundation construction and the kind of soil. Its values for anchor foundation models in sand and clay soils are found to be in the interval from 2 to 2.7. It is possible to take \( n = 2.15 \) for cylindrical and inclined piled anchors, \( n = 2.3 \) for anchor slabs and \( n = 2.6 \) for mushroom-like foundations with some stability of foundation in the first approximation.

Thus, using the principle of approximate similarity, with summarizing the model tests of anchor foundation structures in relative values, makes it possible to introduce the empirical method of calculations of natural foundation bearing capacity and deformity. The essence of this method can be represented in the following way.

Repeated (3 or 5 times) model tests of anchor foundation of definite construction in corresponding soil conditions are carried out and approximate model testing requirements are observed, the limit of foundation bearing capacity being exhausted by pull-out.

The values \( N_{\lim}^{\text{mod}} \) and \( U_{\lim}^{\text{mod}} \) are determined. Multiplying model test results by transitional coefficients according to dependencies (2) and (3), the ultimate points of prototype displacement and loading are found in absolute values. They are multiplied by non-dimensional coordinates of intermediate points of the «load-displacement» dependence and a diagram is plotted for natural foundation. The diagram gives the opportunity to find corresponding efforts or displacements caused by them with the necessary reliability. The calculated load is determined by means of permissible displacements.

The principle of the calculation of anchor foundation bases with the use of small-scale models test results will be shown in the following example.

**CALCULATION EXAMPLE**

It is necessary to calculate the displacement and stability of the model of the inclined bored anchor foundation having the following dimensions: full length 3.2 m, the root 6.4 m in diameter and 1.2 m in length, the inclination of the axis — 20°, the slope angle of the boring depth — 40°) in morainic sandy loam.

**SOLUTION**

1. Average «load-displacement» dependences were obtained for the model which is 4 times less than the anchor being calculated and having equal deflection from horizon and the slope angle depth; tests were repeated three times in identical soil. The non-dimensional parameters are given in table 1.

<table>
<thead>
<tr>
<th>( C_i^N )</th>
<th>( C_i^U )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C_i^N )</td>
<td>( C_i^U )</td>
</tr>
<tr>
<td>0.10</td>
<td>0.01</td>
</tr>
<tr>
<td>0.20</td>
<td>0.02</td>
</tr>
<tr>
<td>0.30</td>
<td>0.03</td>
</tr>
<tr>
<td>0.40</td>
<td>0.06</td>
</tr>
<tr>
<td>0.50</td>
<td>0.08</td>
</tr>
<tr>
<td>0.60</td>
<td>0.12</td>
</tr>
<tr>
<td>0.70</td>
<td>0.19</td>
</tr>
<tr>
<td>0.80</td>
<td>0.29</td>
</tr>
<tr>
<td>0.90</td>
<td>0.48</td>
</tr>
<tr>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The arithmetical mean values \( N_{\lim}^{\text{mod}} \) and \( U_{\lim}^{\text{mod}} \) according to the test results were 0.20 t and 0.30 mm respectively.

2. The maximum value of the base bearing capacity of calculated bored anchor and its displacement are determined in the following way.
\[ N_{\text{prot}}^\text{lim} = N_{\text{mod}}^\text{lim} \cdot \left( \frac{L_{\text{prot}}}{L_{\text{mod}}} \right)^{2.6} = 0.20 \left( \frac{320}{80} \right)^{2.6} = 7.3 \text{ t} \]

\[ \bar{U}_{\text{prot}}^\text{lim} = \bar{U}_{\text{mod}}^\text{lim} \cdot \left( \frac{L_{\text{prot}}}{L_{\text{mod}}} \right) = 0.30 \times 4 = 1.20 \text{ cm} \]

3. Compile the supposed diagram of «load-displacement» dependence for the prototype. For this purpose multiply maximum load and displacement values by the non-dimensional diagram coefficients from table I. The results are summarized in table 2 and the diagram \( \bar{U}_{\text{prot}}^\text{lim} = f (N_{\text{prot}}^\text{lim}) \) is compiled (fig. 9 a, curve 1).

There are diagrams obtained experimentally and giving the opportunity to compare calculation results (fig. 9 a, curve 2). This very figure shows the comparative diagrams for other models of bored anchors (S 1.5: 10 curves 3.4; S 2: 10 - curves 5.6; S 3: 10 - curves 7.8) in an identical soil.

Table 2

| \( N_{\text{prot}}^\text{lim} \) (t) | 0.73 | 1.46 | 2.19 | 2.92 | 3.65 | 4.38 | 5.10 | 5.83 | 6.56 | 7.30 |
| \( \bar{U}_{\text{prot}}^\text{lim} \) (cm) | 0.01 | 0.02 | 0.04 | 0.07 | 0.10 | 0.14 | 0.23 | 0.34 | 0.58 | 1.20 |

Figure 9 b shows comparative diagrams for mushroom-like anchor \( a = b = 110 \text{ cm}; h = 1.5 \text{ m} \) curves 1.2; \( a = b = 96 \text{ cm}; h = 1.31 \text{ m} \) — curves 3.4; \( a = b = 69 \text{ cm}; a = b = 94 \text{ cm} \) — curves 5.6; \( a = b = 55 \text{ cm}; h = 75 \text{ cm} \) — curves 7.8) in average-sized sand. Calculated (curve 1) and experimental (curve 2) «load-displacement» dependence for cylindrical cross-bar anchor with the diameter of 56 cm and 300 cm long in morainic sandy loam with the pull-out \( = 40^\circ \) are represented in fig. 9 c.

Calculated diagrams are obtained by the testing of models with a 1: 5 scale. Thus, the use of approximate values, while summarizing the results of approximate anchor foundation model testing, put in the first place the consideration of base soil properties and made it possible to solve a problem almost unsolvable for the practice of full identity and for analytical calculation methods.

Fig. 9. — Analytical and experimental «load-displacement» dependences:

a) for inclined bored anchor in soft sandy morainic loam;
b) for mushroom anchor foundations in sand;
c) for cross-bar cylindrical anchors in morainic sandy loam.
REFERENCES


DETERMINATION OF THE CARRYING CAPACITY OF GROUND ANCHORS
WITH THE CORRELATION AND REGRESSION ANALYSIS

Calcul de la force portante des tirants d’ancrage
à l’aide de l’analyse de la corrélation et de la régression

by

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SOMMAIRE

Au cours de la dernière décennie, on a effectué un grand nombre d’essais de tirants d’ancrage à Hannovre (Allemagne Fédérale). La communication décrit l’interprétation des observations recueillies par application des méthodes statistiques et propose deux équations pour la capacité portante des tirants d’ancrage : l’une pour les sols pulvérulents et l’autre pour les sols cohérents.

SUMMARY

In the last decade a great number of fundamental and suitability tests of ground anchors have been performed in Hannover, GRF. These tests have been supervised by the «Institut für Grundbau und Bodenmechanik, Technische Universität Hannover», which also performed the laboratory soil investigations. This contribution describes the evaluation of the gathered observations by application of statistical methods. The result of these investigations is the proposal of two equations to estimate the carrying capacity of ground anchors, one for anchors in non-cohesive soil and the other for anchors in cohesive soil.

INTRODUCTION

All equations estimating the carrying capacity of ground anchors published till now have been developed on the basis of theoretical considerations or model tests. Knowing these equations and taking into consideration the experience gained at site, it must be stated that the carrying capacity depends on a multitude of influence factors, which can be classified into the following groups:

- dimensions and construction specification of ground anchors;
- soil characteristics;
- state of stress acting on the grouted body of ground anchors.

It can be supposed that there is no functional relation between the carrying capacity and each of the influence factors, but only a stochastic relation. In natural science and in many branches of technology correlation and regression analysis are used to examine stochastic relations. Regression is a method for getting the best formula relating a dependent variable to independent variables and to determine the constants in this formula by minimizing the sum of squared deviations of observations from the regression equation. The regression equation can be a straight line (linear regression) or a curved line (nonlinear regression) and if there are more than one independent variable (simple regression) it is called multiple regression.

The interdependence between the variables is measured by the correlation coefficient. The simple correlation coefficient shows how much two variables are associated. It varies between +1.0 (perfect correlation) through 0 (no association) to —1.0 (perfect inverse correlation). The multiple correlation coefficient is a measurement for the association between a dependent variable and two or more independent variables. It varies between 0 (no correlation) to 1.0 (perfect correlation). The partial correlation coefficient measures the association between a dependent variable and only one independent variable, when the other independent variables are held constant. It is also measured in the range +1.0 to —1.0.

LIST OF SYMBOLS

The following notations are used in this paper:

\[ a_j \] : regression constants.
\[ A \] : carrying capacity of ground anchors in kN.
\[ B \] : relative importance factor.
\[ c' \] : cohesion of cohesive soil in kN/m².
\[ C_c = \frac{d_{30}}{d_{10}} \cdot d_{60} \] : coefficient of curvature.
\[ d_A \] : diameter of the grouted body.
\[ d_{\text{eff}} \] : effective diameter in mm.
\( d_{10} \): soil diameter at which 10% of the soil weight is finer.

\( d_{30} \): soil diameter at which 30% of the soil weight is finer.

\( d_{60} \): soil diameter at which 60% of the soil weight is finer.

\( D_1 \): percentage of soil with grain diameters \( d < 0.006 \text{ mm} \).

\( D_2 \): percentage of soil with grain diameters \( 0.006 \text{ mm} < d < 0.02 \text{ mm} \).

\( D_3 \): percentage of soil with grain diameters \( 0.02 \text{ mm} < d < 0.06 \text{ mm} \).

\( D_4 \): percentage of soil with grain diameters \( d > 0.06 \text{ mm} \).

\( D_5 \): percentage of soil with grain diameters \( d < 0.2 \text{ mm} \).

\( D_6 \): percentage of soil with grain diameters \( 0.2 \text{ mm} < d < 0.6 \text{ mm} \).

\( D_7 \): percentage of soil with grain diameters \( 0.6 \text{ mm} < d < 2.0 \text{ mm} \).

\( D_8 \): percentage of soil with grain diameters \( d > 2.0 \text{ mm} \).

\( F_m \): superficies of the bond-to-ground length in \( \text{m}^2 \).

\( h_m \): height of the overburden at the half of the bond-to-ground length in \( \text{m} \).

\( l_e \): consistency index.

\( I_p \): plasticity index.

\( k \): coefficient of permeability.

\( l_o \): bond-to-ground length in \( \text{m} \).

\( p \): maximum injection pressure.

\( U = d_{60}/d_{10} \): uniformity coefficient.

\( V_{\text{cm}} \): lime content in percent.

\( w \): natural water content in percent.

\( w_p \): plastic limit in percent.

\( x \): angle of inclination of ground anchors in grade.

\( \gamma \): unit weight in \( \text{kN} / \text{m}^3 \).

\( \psi ' \): angle of friction in grade.

\( \tau \): shear stress in non-cohesive soil in \( \text{kN} / \text{m}^2 \).

\( \tau_c \): shear stress in cohesive soil in \( \text{kN} / \text{m}^2 \).

**INFLUENCES ON THE CARRYING CAPACITY**

The results of about 130 fundamental and suitability tests were the basis of the following investigations. Only in some cases could the breaking load of the tested ground anchors be measured and therefore it was necessary to estimate the maximum load. By evaluating the stress-strain curves from the tests as proposed by T.F. Herbst (1971) and H. Grade (1974) the real placing bond-to-ground length could be found out. Using these determined bond-to-ground lengths the following maximum loads could be estimated: for anchors in non-cohesive soil according to H. Grade (1974) in the range of 614 kN to 1167 kN and for the anchors in cohesive soil as proposed by F.R. Hahn (1974) in the range of 181 kN to 954 kN.

The influence of the following factors on the carrying capacity was taken into consideration:

a) from the dimensions and construction specification of the anchors:
- bond-to-ground length;
- superficies of the bond-to-ground length;
- inclination of the anchors;
- maximum injection pressure;

b) from soil characteristics of non-cohesive soil:
- grain size distribution;
- uniformity coefficient;
- effective diameter;
- coefficient of permeability;

b) from soil characteristics of cohesive soil:
- grain size distribution;
- natural water content;
- consistency limits;
- lime content;

---

![Fig. 1. — Temporary anchor construction of the investigated typ.](image-url)
d) from the state of stress acting on the grouted body:

- normal stress;
- shear stress;

both estimated by different formulas.

The aim of these investigations was to find answers to the following questions:

1) Which factors can have an influence on the carrying capacity?
2) How intense is the dependence of the carrying capacity on individual factors?
3) What are the direct associations between the carrying capacity and the influencing factors?

This paper deals only with temporary anchors as shown in fig. 1.

CORRELATION- AND REGRESSION ANALYSIS OF GROUND ANCHORS IN NON-COHESIVE SOIL

The test results of 51 ground anchors were taken into consideration. Using the linear multiple regression and taking 8 influence-factors in each case multiple correlation coefficients between 0.83 and 0.97 were measured. The formula yielding the highest value included the following 8 independent variables:

\[ A = f(F_M, p, D_s, D_6, D_7, D_g, k, \tau) \]  
(1)

To determine the degree of association between the carrying capacity and only one of the 8 influence-factors of formula (1), partial correlation coefficients have been computed.

Comparatively the superfluous of the bond-to-ground length \( F_M \) has been exchanged for the bond-to-ground length \( l_o \). The results are shown in the following table:

<table>
<thead>
<tr>
<th>No</th>
<th>multiple investigated variables</th>
<th>simple correlation coefficient</th>
<th>partial correlation coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.972 A, F_M</td>
<td>0.471</td>
<td>0.543</td>
</tr>
<tr>
<td>2</td>
<td>0.928 A, l_o</td>
<td>0.350</td>
<td>0.459</td>
</tr>
<tr>
<td>3</td>
<td>0.972 A, p</td>
<td>-0.267</td>
<td>0.261</td>
</tr>
<tr>
<td>4</td>
<td>0.972 A, D_s</td>
<td>0.239</td>
<td>0.000</td>
</tr>
<tr>
<td>5</td>
<td>0.972 A, D_6</td>
<td>0.039</td>
<td>0.000</td>
</tr>
<tr>
<td>6</td>
<td>0.972 A, D_7</td>
<td>-0.269</td>
<td>0.000</td>
</tr>
<tr>
<td>7</td>
<td>0.972 A, D_g</td>
<td>-0.175</td>
<td>0.000</td>
</tr>
<tr>
<td>8</td>
<td>0.972 A, k</td>
<td>0.015</td>
<td>-0.789</td>
</tr>
<tr>
<td>9</td>
<td>0.972 A, ( \tau )</td>
<td>0.758</td>
<td>0.932</td>
</tr>
</tbody>
</table>

To known the influence of other soil characteristics the coefficient of permeability in the formula (1) has been exchanged for other factors. The following correlation coefficients have been computed:

<table>
<thead>
<tr>
<th>No</th>
<th>multiple investigated variables</th>
<th>simple correlation coefficient</th>
<th>partial correlation coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.884 A, U</td>
<td>-0.516</td>
<td>0.501</td>
</tr>
<tr>
<td>2</td>
<td>0.914 A, C</td>
<td>-0.049</td>
<td>-0.422</td>
</tr>
<tr>
<td>3</td>
<td>0.939 A, ( d_w )</td>
<td>0.055</td>
<td>-0.748</td>
</tr>
</tbody>
</table>

By using the regression analysis, the constants of the formula (1) were determined. But these constants do not adequately reveal the relative effect of the independent variables on the dependent variable because of differences in the units. The importance of the independent variables can be measured by the relative importance-factor of variables \( B \). It can be stated that the dependent variable is influenced more by the independent variable with a larger relative important factor.

This investigation shows that the whole grain size curve is very important for the carrying capacity, while the separate grain size groups have only a negligible influence.

As a result of a great number of calculations, the following equation to estimate the carrying capacity of ground anchors in non-cohesive soil is proposed:

\[ A = a_0 + a_1 \cdot F_M + a_2 \cdot D_s + a_3 \cdot D_6 + a_4 \cdot D_7 + a_5 \cdot D_g + a_6 \cdot k + a_7 \cdot \tau \]  
(2)

with

\[ F_M = \pi \cdot d_A \cdot l_o \]

and

\[ \tau = \frac{2 - \sin \varphi'}{2} \cdot \gamma \cdot h_m \cdot \tan \varphi' \]

The correlation analysis yielded a multiple correlation coefficient equals 0.96 and the following values for the constants of formula (2):

\[ a_0 = -2 \, 679.36 \]
\[ a_1 = +34.12 \]
\[ a_2 = +29.20 \]
\[ a_3 = +30.94 \]
\[ a_4 = +20.63 \]
\[ a_5 = +31.92 \]
\[ a_6 = -2 \, 051.48 \]
\[ a_7 = +9.73 \]

Using formula (2) to estimate the carrying capacity it must be observed that the grain size curve lies within the boundaries of fig. 2 and the values of the influence-factors do not exceed the following limits:
The test results of 57 ground anchors have been investigated. Using the linear multiple regression and taking 9 influence-factors into consideration in each case multiple correlation coefficients between 0.97 and 0.98 have been computed. The formula (3) yielded the highest value.

\[ A = f(F_M, p, D_1, D_2, D_3, D_4, l_b, V_w, \tau_e) \]  

(3)

The following table contains the multiple, simple and partial correlation coefficients of the above formula, in which the superficials of the bond-to-ground length \( F_M \) has been exchanged again for the bond-to-ground length \( l_b \):

<table>
<thead>
<tr>
<th>No</th>
<th>multiple correlation coefficient</th>
<th>investigated variables</th>
<th>simple correlation coefficient</th>
<th>partial correlation coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.983</td>
<td>A, F_M</td>
<td>0.508</td>
<td>0.752</td>
</tr>
<tr>
<td>2</td>
<td>0.983</td>
<td>A, L_b</td>
<td>0.057</td>
<td>0.569</td>
</tr>
<tr>
<td>3</td>
<td>0.983</td>
<td>A, p</td>
<td>0.340</td>
<td>0.304</td>
</tr>
<tr>
<td>4</td>
<td>0.983</td>
<td>A, D_1</td>
<td>-0.698</td>
<td>-0.157</td>
</tr>
<tr>
<td>5</td>
<td>0.983</td>
<td>A, D_2</td>
<td>-0.021</td>
<td>0.170</td>
</tr>
<tr>
<td>6</td>
<td>0.983</td>
<td>A, D_3</td>
<td>0.669</td>
<td>0.210</td>
</tr>
<tr>
<td>7</td>
<td>0.983</td>
<td>A, D_4</td>
<td>0.800</td>
<td>0.385</td>
</tr>
<tr>
<td>8</td>
<td>0.983</td>
<td>A, I_c</td>
<td>0.638</td>
<td>0.093</td>
</tr>
<tr>
<td>9</td>
<td>0.983</td>
<td>A, V_w</td>
<td>-0.141</td>
<td>-0.119</td>
</tr>
<tr>
<td>10</td>
<td>0.983</td>
<td>A, \tau_e</td>
<td>0.676</td>
<td>0.666</td>
</tr>
</tbody>
</table>

Subsequently the constants of formula (3) and the relative importance factors of the independent variables have been computed as already described above. The results indicate that the carrying capacity is most influenced by the superficials of the bond-to-ground length and by the consistency index.

Taking the results of these investigations as a basis, it can be proposed to estimate the carrying capacity of ground anchors in cohesive soil by the following equation:

\[ A = a_0 + a_1 \cdot F_M + a_2 \cdot D_1 + a_3 \cdot D_2 + a_4 \cdot D_3 + a_5 \cdot D_4 + a_6 \cdot I_c + a_7 \cdot \tau_e \]  

(4)

where

- \( F_M = \pi \cdot d_A \cdot l_b \)
- \( \tau_e = \frac{\cos^2 \alpha + \sin^2 \alpha (1 + 2 \cdot \tan^2 \varphi') + 2 \cdot \sin \alpha \cdot \cos \alpha \cdot \epsilon' \cdot \cos^2 \varphi'}{\epsilon' \cdot \cos^2 \varphi'} \)

The test results of 57 ground anchors have been investigated. Taking 9 influence factors into consideration in each case multiple correlation coefficients between 0.97 and 0.98 have been computed. The formula (3) yielded the highest value.

\[ A = f(F_M, p, D_1, D_2, D_3, D_4, l_b, V_w, \tau_e) \]  

(3)
The multiple correlation coefficient for this equation was 0.98 and the following values for the constants have been calculated:

\[ a_0 = + 721.51 \]
\[ a_1 = + 71.84 \]
\[ a_2 = -- 9.81 \]
\[ a_3 = -- 1.99 \]
\[ a_4 = -- 21.22 \]
\[ a_5 = + 10.34 \]
\[ a_6 = + 95.15 \]
\[ a_7 = + 2.56 \]

Estimating the carrying capacity of ground anchors in cohesive soil by using formula (4), the grain size curve must be within the boundaries of fig. 3 and the values of the influence-factors are not allowed to exceed the following limits:

\[ 0.98 \text{ m}^2 \leq F_m \leq 6.48 \text{ m}^2 \]
\[ 6.50 \text{ cm} \leq d_A \leq 16.80 \text{ cm} \]
\[ 4.0 \text{ m} \leq l_p \leq 15.00 \text{ m} \]
\[ 20 \% \leq D_1 \leq 76 \% \]
\[ 12 \% \leq D_2 \leq 27 \% \]
\[ 4 \% \leq D_3 \leq 27 \% \]
\[ 2 \% \leq D_4 \leq 34 \% \]
\[ 0.84 \leq I_p \leq 1.55 \]
\[ 50.7 \text{ kN/m}^2 \leq \tau_e \leq 165.3 \text{ kN/m}^2 \]

**CONCLUSIONS**

The evaluation of the test results of 51 ground anchors in non-cohesive as well as 57 ground anchors in cohesive soil by using the correlation and regression analysis shows the influence of ground anchor dimensions, soil characteristics and the shear stresses acting on the grouted body on the carrying capacity of ground anchors. On the basis of these investigations two equations to estimate the carrying capacity are proposed, one for ground anchors in non-cohesive soil and the other one for ground anchors in cohesive soil. The constants of these equations, including 7 parameters in each case, have been determined by means of the linear multiple regression analysis.

Both equations include the grain size distribution of the soil classified in 4 fractions, the superficies of the bond-to-ground length and the shear stress acting on the superficies of the bond-to-ground length. Another parameter in the formula for ground anchors in non-

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**Fig. 3.** — Boundaries of the grain size distribution of the investigated cohesive soil.

**Fig. 4.** — Differences between the carrying capacity of ground anchors in non-cohesive soil gained by evaluating the stress-strain curves (\( A_g \)) and estimated by the proposed formula (\( A_{reg} \)).
cohesive soil is the coefficient of permeability, whereas in the formula for ground anchors in cohesive soil the consistency index appears. Furthermore the unit weight, the angle of friction and the cohesion of cohesive soil appear in both equations. Estimating with the proposed equations the carrying capacity of the ground anchors used in the regression analysis, it is found that about 90 percent of the ground anchors have only differences of less than 60 kN from the loads estimated by evaluating the stress-strain curves from the tests as shown in fig. 4 and 5.

In spite of the good correlation demonstrated in fig. 4 and 5 it must be stated that it is not possible to abandon the acceptance test of each ground anchor at site due to stratification and nonhomogeneity of soil. Furthermore there is no possibility to take the different placing methods into consideration. The advantage of the proposed equations based on performed tests at site is that reliable loads can be estimated already during the stage of design without special expenditures or time delay.

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ANCHOR FIELD TESTS IN CARBONIFEROUS STRATA
Essais en place d’ancrage dans un sédiment carbonifère

by

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SUMMARY
The paper discusses the preliminary findings of full scale tests on 57 anchors installed in carboniferous sediments. The site geology is briefly described together with the anchor construction and testing methods adopted. To investigate the influence of anchor geometry on failure mode, anchors ranging in overall depth from 0.75 to 12 metres were tested with grouted fixed anchor lengths of 0.75 to 6 metres. The observed effects of depth of embedment, grout surcharge, tendon configuration, interstrand spacing and tendon density on anchor performance are discussed in relation to current practice. Measurements of interfacial bond and load transmission are presented.

INTRODUCTION
A world-wide survey of prestressed rock anchor practice by Littlejohn and Bruce (1975-1976) has highlighted a dearth of information concerning the fundamental behaviour of rock anchors with particular reference to the mechanism of load transfer and modes of failure.

SITE GEOLOGY

The anchors were installed in Upper Carboniferous sediments of the Middle Grit Group of the upper part of the Millstone Grit Series. The sequence fined downwards from gently dipping massive, coarse, gritty siliceous sandstones to finer grained flaggy and shaley sandstones. In addition, a total of eight soft, friable mudstone beds were exposed or inferred in the sequence. Different groups of anchors were installed from different stratigraphic levels due to the presence of various benches, but each intersected at least one argillaceous bed at the grouted fixed anchor level. The whole sequence was conspicuously vertically jointed, the major orientations being north, east-north-east, (most prominent), and south-east. Joint spacing varied greatly, being up to 1 metre in the coarser sandstones. The major geotechnical properties are provided in Table 1.

<table>
<thead>
<tr>
<th>Table 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Summary of geotechnical properties</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Fracture Index</td>
</tr>
<tr>
<td>RQD</td>
</tr>
<tr>
<td>Unit weight (Mg/m³)</td>
</tr>
<tr>
<td>Ultimate Pulse</td>
</tr>
<tr>
<td>Velocity (km/sec)</td>
</tr>
<tr>
<td>Diametral Point Load Strength (N/mm²)</td>
</tr>
<tr>
<td>Elastic Modulus - material (N/mm²)</td>
</tr>
</tbody>
</table>
ANCHOR CONSTRUCTION

All holes were drilled vertically, by rotary percussion, to provide a nominal diameter of 114 m. The tendons, which consisted of 10 No. 7-wire Dyform 15.2 mm diameter strands of 300 kN individual capacity were assembled in a straight, parallel formation with a 10 mm clear spacing (fig. 1) unless otherwise specified. In order to dissociate the free (elastic) length from the surrounding grout or ground, grease impregnated tape was wrapped around each strand over the required length.

Prior to tendon homing each hole was water tested, and sealed with neat cement grout, if the water loss exceeded 5 litres/minute/atmosphere. On completion of sealing, neat Rapid Hardening Portland Cement grout \((w/c = 0.45)\) was tremied into the hole and the tendon slowly homed.

The grout, prepared in a conventional paddle mixer, gave bleed readings of 1.3 to 2.1\% and stressing only took place once the grout had achieved a crushing strength of 28 N/mm\(^2\). At least two anchors of each type were installed.

Fig. 1. — Arrangement for standard ten strand tendon.

ANCHOR TESTING

A hydraulic stressing system was evolved to enable the anchors to be incrementally, and cyclically loaded to failure, or to a maximum of 260 kN per strand. The system comprised remote loading through a simply supported beam and accommodated both multistrand and monostrand stressing modes. Dial gauges yielded anchor extensions and rock surface displacements to 0.01 mm accuracy, and annular load cells gave a direct reading of the applied load to 1\% accuracy. A second, independent, direct measure of stress distribution was provided by strain gauges attached at strategic positions on a large proportion of the strands installed. At least one anchor of each type was instrumented in this way.

DISCUSSION OF RESULTS

Rock mass performance

Table 2 illustrates the overall performance of the shallow fully bonded anchors, where all the major modes of failure are reproduced.

<table>
<thead>
<tr>
<th>Anchor No</th>
<th>Embedment (m)</th>
<th>Maximum Test Load (kN)</th>
<th>Rock-grout bond (N/mm(^2))</th>
<th>Grout-tendon bond (N/mm(^2))</th>
<th>Grout Crushing strength (N/mm(^2))</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.75</td>
<td>440</td>
<td>1.64</td>
<td>1.23</td>
<td>45</td>
<td>Rock mass</td>
</tr>
<tr>
<td>2</td>
<td>0.75</td>
<td>500</td>
<td>1.86</td>
<td>1.40</td>
<td>45</td>
<td>Rock mass</td>
</tr>
<tr>
<td>3</td>
<td>0.75</td>
<td>450</td>
<td>1.68</td>
<td>1.26</td>
<td>46</td>
<td>Rock mass</td>
</tr>
<tr>
<td>4</td>
<td>1.50</td>
<td>1 495</td>
<td>2.72</td>
<td>2.09</td>
<td>60</td>
<td>Rock mass</td>
</tr>
<tr>
<td>5</td>
<td>1.50</td>
<td>1 355</td>
<td>2.52</td>
<td>1.89 F</td>
<td>62</td>
<td>Grout-tendon</td>
</tr>
<tr>
<td>6</td>
<td>1.50</td>
<td>1 206</td>
<td>2.24</td>
<td>1.68</td>
<td>50</td>
<td>Rock mass</td>
</tr>
<tr>
<td>23</td>
<td>1.50 (u)</td>
<td>1 834</td>
<td>3.41 *</td>
<td>2.56</td>
<td>50</td>
<td>Rock mass</td>
</tr>
<tr>
<td>24</td>
<td>1.50 (u)</td>
<td>1 594</td>
<td>2.97 *</td>
<td>2.23</td>
<td>51</td>
<td>Rock mass</td>
</tr>
<tr>
<td>51</td>
<td>2.25</td>
<td>2 411</td>
<td>2.99</td>
<td>2.24</td>
<td>35</td>
<td>None-rock mass imminent</td>
</tr>
<tr>
<td>52</td>
<td>2.25</td>
<td>1 978</td>
<td>2.45 F</td>
<td>1.84</td>
<td>36</td>
<td>Rock-grout</td>
</tr>
<tr>
<td>53</td>
<td>2.25</td>
<td>1 891</td>
<td>2.35 F</td>
<td>1.76</td>
<td>37</td>
<td>Rock-grout</td>
</tr>
<tr>
<td>7</td>
<td>3.00</td>
<td>2 355</td>
<td>2.19</td>
<td>1.64</td>
<td>49</td>
<td>Strand fracture</td>
</tr>
<tr>
<td>8</td>
<td>3.00</td>
<td>2 469</td>
<td>2.30</td>
<td>1.72 F</td>
<td>43</td>
<td>Grout-tendon</td>
</tr>
<tr>
<td>9</td>
<td>3.00</td>
<td>2 122</td>
<td>1.97</td>
<td>1.48 F</td>
<td>44</td>
<td>Grout-tendon</td>
</tr>
</tbody>
</table>

(*) Calculated as a straight shaft. F - failure value at interface.

TABLE 2

Average interfacial bond values at maximum test loads for shallow fully bonded anchors.
Up to embedment depths of 1.5 m, failure occurred mainly in the rock mass. For greater depths failure tended to be localised at one of the grout interfaces. For rock mass failure, the shape of the rock volume mobilised in each case was strongly controlled by the incipient rock mass structure (fig. 2).

For example, the major radial fractures developed along the trends of the major joint directions, whilst the projected shape of the rock volume mobilised below the surface was strongly influenced by the laminar nature of the mass. Under similar conditions, underreamed anchors sustained higher loads than their straight shaft counterparts. Underreaming was carried out with a UAC patented tool to form pairs of «bells» of 230 mm diameter and whilst a greater extent of rock was mobilised, the pattern of surface fracturing was not radically different.

For the shallow anchors installed in unweathered rock in this project, the ultimate resistance to rock mass failure is reasonably estimated from the empirical rule

\[ P = 600 d^2, \]

where \( d \) is the depth of embedment (m). For the traditional and conservative design concept pertaining to the weight of an inverted 90° cone, and using a unit weight of 2.5 Mg/m³ for the rock, factors of safety ranging from 14 to 45 are indicated, assuming the apex at the base of the anchor. Employing the observed extent of the fissuring to speculate on the size of cones mobilised, included angles of (117° - 144°) and (90° - 114°) can be calculated for the apex positioned at the mid-point and base of the anchor, respectively. Assessing the weights of these cones the factors of safety against pull-out are (14 - 36) and (8 - 29) for the apex at mid point and base, respectively. These figures highlight that other «rock strength» parameters constitute the major component of resistance to pull-out, and assuming that the actual failure volumes were more akin to cones with apices at the mid point of the anchor, then average «rock strength» values mobilised over the surface area varied from 0.076 - 0.185 N/mm². These values may be compared with the design recommendations of 0.034 N/mm² by Saliman & Schaefer (1968), and 0.024 N/mm² by Hilf (1973). Based on the rock surface displacements the tests show considerable surface disturbance for maximum loads of 900 kN for slenderness ratios (distance from rock surface to the proximal end of the grouted fixed anchor divided by the borehole diameter) up to 8, but at a value of 13 for loads of 1360 kN no surface movement was observed. Above a value of 13, failure was localised, invariably at the grout-tendon interface, and it is considered that this type of observation is invaluable when assessing the relevance of stressing through a bearing plate of a simply supported beam.

Grout-tendon interface

Bearing in mind the high grout strengths measured prior to the stressing of each anchor (> 35 N/mm²) no correspondence between ultimate average grout-tendon bond values and grout strength was detected. The presence of surcharge grout (up to 9 m) did not markedly affect grout-tendon bond values (table 3) or the phenomenon of debonding. A grout surcharge in excess of 3 m did however lead to a steady and quieter pull-out of strands compared with the sudden explosive type of failure which may be observed without surcharge.
It is also noteworthy that of the five anchors with less than 2 m surcharge, four had an initial failure followed by a higher maximum, or a maximum, followed by failure at a lower load on the subsequent cycle. Average ultimate bond values of 1.01 - 1.68 N/mm² were recorded compared with a design range of 0.25 - 1.35 N/mm² which are commonly observed in practice according to Littlejohn & Bruce (1975). In this respect it should be noted that for a single strand anchor PC 1 (1974) indicates a bond of about 3.1 N/mm², and the Australian Standard CA 35 (1973) suggests a working bond of up to 2.1 N/mm² in design for single or multi-strand tendons.

With regard to load resisting characteristics in relation to tendon configuration, individually noded strand tendons were more effective than generally noded tendons (table 4) but both showed distinct advantages over parallel, straight tendons. To effect general tendon nodding the strands were bound intermediate to the spacers in the fixed length. Individual strand nodes were produced by unravelling each strand and introducing a small metal collar onto the straight central wire at the appropriate point: the peripheral wires were then returned to their original lay around it, with a proturbance thereby created at that point.

<table>
<thead>
<tr>
<th>Anchor No</th>
<th>Spacing between strands (mm)</th>
<th>Max. Test Load (kN)</th>
<th>Grout Crushing Strength (N/mm²)</th>
<th>Max. Bond (N/mm²)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>10</td>
<td>1,535</td>
<td>60</td>
<td>1.79 F</td>
<td>Grout-tendon failure</td>
</tr>
<tr>
<td>36</td>
<td>10</td>
<td>1,555</td>
<td>62</td>
<td>1.81</td>
<td>Grout-tendon failure</td>
</tr>
<tr>
<td>41</td>
<td>5</td>
<td>1,555</td>
<td>42</td>
<td>1.81</td>
<td>Load held - large extensions</td>
</tr>
<tr>
<td>42</td>
<td>5</td>
<td>1,555</td>
<td>44</td>
<td>1.81</td>
<td></td>
</tr>
<tr>
<td>37</td>
<td>0</td>
<td>1,351</td>
<td>38</td>
<td>1.57 F</td>
<td>Initial yield at 1978 kN</td>
</tr>
<tr>
<td>38</td>
<td>0</td>
<td>1,455</td>
<td>40</td>
<td>1.69 F</td>
<td>Load held</td>
</tr>
</tbody>
</table>

Strand spacing was also varied but no reduction in bond was observed down to a clear spacing of 5 mm. Thereafter, only when strands were actually in contact was any significant reduction in bond observed (table 5). Nevertheless, the use of centraliser/spacer units in the grouted fixed anchor zone is strongly advised, and spacings lower than 5 mm are only recommended where noding is employed to increase mechanical interlock.

### TABLE 4
Average bond values at maximum test loads for different tendon configurations (10 strand tendon)

<table>
<thead>
<tr>
<th>Anchor No</th>
<th>Tendon Configuration</th>
<th>Max. (kN)</th>
<th>Test Load (% f.p.u.)</th>
<th>Grout Crushing Strength (N/mm²)</th>
<th>(Max. Bond) Rock-grout (N/mm²)</th>
<th>(N/mm²) Grout-tendon</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>straight, parallel</td>
<td>1,920</td>
<td>64</td>
<td>48</td>
<td>1.79</td>
<td>1.34</td>
<td>Grout-tendon failure</td>
</tr>
<tr>
<td>21</td>
<td>strands</td>
<td>2,093</td>
<td>70</td>
<td>50</td>
<td>1.95</td>
<td>1.46</td>
<td>Grout-tendon failure</td>
</tr>
<tr>
<td>31</td>
<td>general nodding of</td>
<td>2,481</td>
<td>83</td>
<td>48</td>
<td>2.31</td>
<td>1.73</td>
<td>load held - large extensions</td>
</tr>
<tr>
<td>32</td>
<td>tendon</td>
<td>2,248</td>
<td>75</td>
<td>49</td>
<td>2.09</td>
<td>1.38</td>
<td>Initial yield at 1978 kN</td>
</tr>
<tr>
<td>33</td>
<td>local nodding of</td>
<td>2,411</td>
<td>80</td>
<td>50</td>
<td>2.24</td>
<td>1.68</td>
<td>Load held</td>
</tr>
<tr>
<td>34</td>
<td>of strands</td>
<td>2,411</td>
<td>80</td>
<td>48</td>
<td>2.24</td>
<td>1.68</td>
<td>Load held</td>
</tr>
</tbody>
</table>

In relation to the extent of debonding table 6 illustrates the basic characteristics for a strand tendon, where the steel represents 10.7% of the hole area, and

### TABLE 6
Extent of effective debonding for 6 and 10 strand tendons, at various tendon stress levels

<table>
<thead>
<tr>
<th>Anchor No</th>
<th>No of Strands</th>
<th>Effective debonded length (m) at tendon stress of (% f.p.u)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40%</td>
<td>50%</td>
<td>62.5%</td>
</tr>
<tr>
<td>-----------</td>
<td>---------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>20</td>
<td>10</td>
<td>1.12</td>
</tr>
<tr>
<td>21</td>
<td>10</td>
<td>1.12</td>
</tr>
<tr>
<td>35</td>
<td>6</td>
<td>0.81</td>
</tr>
<tr>
<td>36</td>
<td>6</td>
<td>0.91</td>
</tr>
</tbody>
</table>

f.p.u.: characteristic strength of the tendon (0.1% proof stress = 83.5% f.p.u)
a 10 strand tendon (17.8% hole area). The inference is clear for the less congested tendons, namely that the rate of effective debonding is slower and the failure load per strand is greater. Whilst it is appreciated that 10 strand anchors would normally have a fixed anchor greater than 3 m, the homing of a high density tendon (17-18% hole area) can give problems due to damage or contamination of the strands, and fixing of the centraliser/spacer units can be difficult and time consuming. Therefore whilst the former installation is feasible, it is recommended that the tendon density be limited to 15% of the hole area wherever possible.

Debonding is a little understood phenomenon and although the analysis of strain gauge and load/extension data are by no means complete the major conclusions to date are:

1) effective debonding occurs at low loads (15 kN/strand) and progresses distally with increasing load. The effective debonded length comprises wholly debonded, and partially debonded sections. The latter, pertaining to adhesive bond failure, may be determined from strain gauge records, and the extent of this adhesion zone appears to be proportional to the applied load.

2) For grout/tendon failure the limit of effective debonding was 0.5 to 1.0 m from the distal end. Under working conditions (50-60% fpu), where fpu is the characteristic strength of the tendon, an effective debonded length of 1 to 2 m should be anticipated. The partially debonded zone extends some distance distally of the point of effective debonding, possibly about 0.8 m at 62.5% fpu. Based on this preliminary information it is clear that there should be no reduction in the current minimum fixed anchor length of 3 m, often recommended in practice.

In general, where localised failure of the complete tendon at the grout/tendon interface was observed subsequent restressing mobilised on average a total tendon load of about 85% of that recorded at first failure. When tested with a monojack individual strands commonly yielded pull-out resistances in excess of the initial average multijack value. «Failed» anchors may therefore have a useful role to play at a lower capacity for temporary works. In these circumstances it is strongly recommended that post failure cyclic loading tests be carried out in order to assess maximum safe working loads for anchors which might otherwise be discarded.

REFERENCES


CONTRIBUTION TO LOADING TEST PROCEDURE OF GROUND ANCHORS

La contribution à la méthode d'essais des tirants d'ancre sont par

by

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and
T. NAJDER
Gdansk Technical University, Gdansk - Poland

1. INTRODUCTION

During construction of a dry dock, ground anchors were used as permanent anchors of sheet walls of the dry dock wall and as temporary anchors of special foundations used for erection of a heavy gantry crane. Analyses were particularly made concerning the choice of a proper drilling method and its influence on the bearing capacity of the above anchors in sandy soils with clay interbeddings. All anchors were tested by means of loading tests made according to a modified program based on the result of an analysis of the (German standard DIN 4125) method. A comparison of results obtained using different methods known throughout the world has shown that the chosen method can be the most adequate one.

The performed tests, described in the paper, were made with an assumption that the results obtained would deliver information about the influence of the time period which elapsed between the construction of the ground anchor and the loading test, on the magnitude of deformations and bearing capacities of the anchors. Also the magnitude of friction of the free part of the anchor and the preservation of this length as dependent on the construction method were considered, whilst repeated loadings of the same anchors should bring results on the influence of these loadings on the bearing capacity.
The permanent ground anchors investigated were made using the rotary-percussive drill method called «Alvik-X». Casing tubes 3.5 and flushing water were introduced on the free length equal to 17.0 m. The crawler drill on wheels was of the Atlas Copco type BVB-33 whilst the tubes and extension rods were placed in the soil by power rotated drill (top hammer) of the BBE 57-01 type. Number of of blows = 1950/60 s, torque moment = max. 80 kpm, inclination = 1:3. The boring along the length bond to ground, equal to 10.0 m, was performed using the same method but without casing tube and with bentonite flushing. After the assumed depth has been reached the bentonite suspension was pressed out by cement grout (aqueous colloidal dispersion) with w/c = 0.4 to 0.5. Through raising the grout from the hole bottom level concreting of the borehole took place whilst at the same time through the upper end of the tube there was a discharging of the ground and bentonite. The pressing of grout was stopped about 5 min after a total removal of bentonite flushing occurred. Immediately after grouting a 7.5 m Bridon wire strand with centralisers placed in 2.0 m distance along the whole anchoring length was pressed in the grout. Grouting tubes were also installed. To avoid concreting of steel strands along the free length special packers were used.

Once the strands in the grout the anchor head was closed and the pressure was raised to about 15 at (1.5 N/mm²) to increase the bond of the anchor to the ground. One week after concreting of the anchor a prestressing tests was made by means of a hollow ram multi strand jack VSL for a force of 88 Mp on one anchor. The loading force was then lowered to working load of 59 Mp.

The temporary anchors were made using the overburden drilling method (OD) with a very rich water flushing. Reaching the assumed depth, the water flushing was replaced by cement grout of w/c = 0.4. The hole was filled with grout from the bottom level using flexible tubes. The reinforcement consists of 7.5 2 wire strands but without centralisers. The pulling out of casing tubes was connected with addition of grout to keep a constant grout level in the hole. Once the casing tube pulled out a 6.0 m long injection pipe with one way valve was placed. By means of this pipe the injection was performed under a pressure of 7 at (0.7 N/mm²). Seven days later the anchors were prestressed to 90 Mp. After prestressing the loading was lowered to 72 Mp.

Both methods used for construction of permanent and temporary anchors, belong to the rotary-percussive drill method. The action of the bit is a combination of torque and impact with a high frequency, and of axial thrust. It causes a very effective remoulding of the ground whilst the removal of the remoulded ground is assured by water flushing. To obtain an optimum interaction between the ground and the anchor, the remoulding of the ground around the casing tube should extend as low as possible whilst the water flushing should be limited as much as possible. These conditions are fulfilled using the Alvik-X system, depending on the action of an eccentric bit boring a hole which has a diameter slightly bigger than the diameter of the casing tube. The tube penetrates by following the bit but it does not rotate. This causes a lesser remoulding of the soil along the tube shaft. The bit, because of the impact and torque, cuts the soil simultaneously by the bit part and the reamer. The water flushing is discharged through extension rods and holes in the bit which has a special constructed guide. This guide protects the pipe from inflow of soil what means that a certain amount of grains is pressed back into the soil whilst through the tube only water flushing is discharged. In sandy soils it gives a big increase of density.

In the OD-method the bit and the tube are in the tip protected by a ring. They are under a torque and impact transmitted by a common part, namely the shank adapter. In the described cases, however, a simplified OD-method was used. The penetration took place using only casing tube whilst the soil from the tube was removed by water flushing.

Analyzing both methods and the bearing capacities of anchors obtained it can be stated that the OD-method which seems to be simpler and needs less work, does not give any significant increase of efficiency when compared with the Alvik-X method. The remoulding of the ground when using water flushing and casing pipes, as occurs in the OD-method, has very small influence on the bearing capacities obtained. However, in the existing conditions the soil conditions have had the most important influence existing around the grouted body. It was additionally stated that if boring in layered subsoil it is very important to observe the whole time what kind of soil is removed from the casing tube. Only such an observation gives the possibility to adjust the length of the anchor being installed. It should also be mentioned that it was found that the bearing capacity of an anchor is strictly dependent on the removal of bentonite.

3. LOADING TESTS

c) Moving back of the piston and further prestressing to 88 Mp. The piston, when moving, is pulling out the strand wires together with the stressing head. Measuring of piston displacement for the maximum load.

d) Measuring of prestressing losses one hour after application of maximum force.

e) Releasing of prestressing load. The strand wires together with the stressing head are moving back to the bearing plate.
The applied test loading method is a two-stage method in which reaching of a load equal to 150% of the working load is foreseen. Measurements of the magnitude of the creep displacement for this load should take place. The basic fault of this system is the small amount of measuring points which gives a very schematic loading diagram. Because no loading and unloading cycles are available it is not possible to determine the elastic and permanent displacements of an anchor. It is also impossible to find the conformity between the designed and actual free length of the ground anchor. Also the displacement measuring system of the anchor is simplified because it is assumed that this displacement is equal to the extended length of the piston. These values can be found to be different when analyzing for instance the backward movement of the anchored wall.

The necessity of a two stage prestressing method results also from the fact that the maximum outgoing of the piston depending on the structural solution of the jack is smaller than the total displacement of the free anchor length (17.0 m) for the applied load (88 MPa).

When testing the temporary anchors which were used for temporary foundations loaded by a vertical or inclined tensile force the following new test loading procedure was introduced (fig. 2):

a) Prestressing of the anchor in steps to reach 125% of the working load. The increase of load for each step is equal to 15 MPa starting from the initial load $F_o = 15$ MPa,

b) Unloading after each loading step to $F_o$. Unloading is starting from 30 MPa.

c) Keeping the observation time for loads of 45 MPa equal to 15 min, for loads of 60 and 75 MPa — 30 min, and for the maximum load $F_{max} = 90$ MPa equal to 60 min.

d) Measuring of anchor displacements by means of measuring of the outgoing of the piston and settlements of foundations.

e) Measuring of displacements immediately after loading at a certain step (about 1 min) and at the end of the loading steps to obtain the creep displacement. The resulting prestressing losses were equalized by increasing of oil pressure in the jack.

The results obtained made it possible to present diagrams of the load-displacement curves using a new elaborated method which is an extension of the method given in the German standard DIN 4125. These diagrams are made as follows:

a) Axis of a load-total displacement curve and a load-elastic and permanent displacement curve are drawn.

b) Measured results corresponding to all loading and unloading steps are plotted.

c) The real zero coordinate is introduced by shifting the loading axis to a distance equal to the measured outgoing of the piston on the beginning of test for zero increase of loading. It corresponds to the magnitude of the effect in the strand wires.

d) An auxiliary axis is placed through the measuring point which corresponds to the load $F_o = 15$ MPa.

e) Values are plotted on the permanent displacement curve which correspond to following unloading steps measured in relation to the auxiliary axis. It is assumed here that the permanent displacements for $F_o$ are equal to zero.

f) In relation to the load-permanent displacement curve the total displacement $s$ is plotted while it is measured from the axis going through the real zero. Points of the curve of elastic displacement $s_e$ are obtained.

g) A straight line is made through the above points which indicates the elastic displacements taking friction into consideration. This straight line intersects the loading axis in a point of a coordinate which is the sum of initial friction causing displacements lower than should be expected according to Hook's law. Distances between this line and a parallel line plotted through the beginning of the coordinate system correspond to the friction losses on the following steps of the prestressing load.

h) If during the tests the ultimate bearing capacity has not been reached, dependent on the pulling out of the anchor from the soil, an extrapolation is proposed according to two schemes. In the first the change of the curvature of the permanent displacement curve is used. The second depends on the increase of the load which corresponds to the permanent displacement of the anchor.
equal to 10 mm whilst this load is multiplied by 1.30. Such a value was reached during failure loading tests of ground anchors in similar soil conditions.

Because of slight friction in the jack and on the free length of the anchor, mainly due to the straight arrangement of wires in the VSL wedge system, a simplified version of the proposed method was also used (fig. 3). It is based only on one unloading step after the maximum test load is reached and on presentation of the two curves in the way shown on fig. 3.

The conclusion that the anchors fulfill all requirements and may be used for permanent or temporary structures can be drawn analyzing:

- the designed free anchor length in relation to the stated real one, calculated from the inclination of the curve of elastic displacements;
- the difference between the ultimate bearing capacity and the working load;
- the allowable values of friction which are causing lower deformations of wires on the free length in the region of the anchor head an jack,
- the value of creep depending on time (for permanent anchors).

It should be stated that the free lengths obtained from these curves were much lower than designed. It was probably the result of the simplified way of reaching the free displacement of wires in the free length using the flushing out of grout by water. The grout mixed with water came back to the hole and made up an additional part of grouted body. The strand wires in the free length were not isolated or placed in plastic tubes which could allow them to move along this distance. Consequently a significant decrease of safety factor was obtained (fig. 4).

Repeated prestressings indicated a small increase of elastic displacements of anchors. It resulted probably by cracking due to cycle loadings and unloadings of the additional porous part of the grouted body, which caused the increase of the free length of the anchor. Loading tests of 10 anchors have been made twice, after 7 and 130 days from concreting date. The differences in displacements for the same prestressing load were not higher than + 6% to — 3%. This indicates that there is no influence of time on the results of loading tests.
During loading tests the magnitude of prestressing losses as a result of moving back of selfbraking wedges during blocking was also measured. It was stated that the back movement of wedges was from 3 to 8 mm, whilst its magnitude depended on the pressing of wedges by a hammer during preparation of the stressing head for blocking and on its cleanliness. The calculated lock-off losses were in the range of 5 Mp. They were confirmed during later measurements.
RESEARCH ON GROUND ANCHORS IN NON-COHESIVE SOILS

Etude de tirants scellés dans des sols pulvérulents

by

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SOMMAIRE

Des essais d'ancrages en vraie grandeur sur chantier ont été effectués dans des massifs de sables graveleux compactés à différentes densités. Des jauges de déformation collées sur des barreaux d'acier donnent la distribution des contraintes dans la zone fixe d'ancrage. Ces dernières augmentent progressivement avec la force appliquée en tête d'ancrage jusqu'à la rupture. Afin d’estimer par des essais standards in situ, l’influence du type et de la densité du sol sur la capacité portante du tirant d’ancrage, on a présenté le rapport entre la charge critique et le nombre de corps d’essais de pénétration (marteau de 50 kg).

SUMMARY

Full scale field tests on anchors have been performed in one and the same gravelly sand compacted to various densities. During the pull-out tests the distribution of skin friction along the fixed anchor length was determined from strain gauge measurements. The results of 5 series of tests comprising 30 anchors complement a previously published design chart and give an additional correlation between carrying capacity of anchors and number of blows of penetration tests. The variation of skin friction along fixed anchor length with increasing load or with load kept constant over a longer period of time, helps to explain the influence of soil density and fixed anchor length on the carrying capacity and longterm behaviour of anchors.

INTRODUCTION

The last Conference on Diaphragm Walls and Anchorages (London 1974) and the Seminar on the same topic (London 1976) showed that current practice in the field of ground anchors is ahead of theory and that there is urgent need for a proper understanding of the behaviour of the anchors and the surrounding ground under working conditions (C.P. Wroth, 1975). Design charts, which are based on field test results of about 300 anchors (H. Ostermayer, 1975) may help to estimate carrying capacity of anchors in relation to fixed anchor length in certain soil conditions. However, the main factor of soil density influencing the carrying behaviour of anchors in non-cohesive soil has not yet been tackled systematically in field tests.

In 1975-1976 a major research program was carried out in Munich to examine the influence of soil density on the carrying capacity of anchors. In order to have a better insight into the carrying behaviour of anchors, research aimed at investigating the distribution of stresses for different anchor lengths. The effect of time on the variation of stresses was also studied. The so sought information should then help to explain the various important factors that influence the carrying capacity and long-term behaviour of anchors.

TEST PROGRAM

On a test site five series of six anchors each were installed and pull-out tests were performed. A schematic arrangement of the test pit is shown in Fig. 1. The dimensions of the test pit were about 5 x 10 x 10 meters. A rigid concrete wall was used as abutment for the pulling jack.

The anchors of the first series were installed in the existing soil, which was sandy gravel of high density. For the following four test series the soil was replaced by gravelly sand. The grain size distribution curve of the test soil is given in Fig. 1, the coefficient of uniformity was U = 8 to 10. The density of the sand was varied for each series. After every test series the soil was removed and compacted again in layers of about 30 cm height to a desired uniform density with the help of vibrators. The compacted sand carried a surcharge of about 2 m gravel. For each series the soil density was checked by 8 standard penetration tests (SPT) and 4 dynamic penetration tests (50 kg hammer weight and 15 cm² cone area). In addition unit weight and density index were determined for at least 6 samples.

A temporary anchor construction (Type A) was used with a total length of 9 m and an inclination of
TABLE 1
Anchor and soil data for 5 series of tests with 6 anchors each

<table>
<thead>
<tr>
<th>Test series No</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressing steel tendon</td>
<td>4 × 16 mm dia.</td>
<td>32 mm dia.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Casing diameter (mm)</td>
<td></td>
<td>89</td>
<td>76.89</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grouting pressure (MN/m²)</td>
<td>0 to 5.0</td>
<td>0.5</td>
<td>0.5 and 1.0</td>
<td>0.5 and 2.0</td>
<td></td>
</tr>
<tr>
<td>Bond length of tendon (m)</td>
<td>3.0</td>
<td>2.0 and 4.5</td>
<td>3.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Soil type | sandy gravel | gravelly sand |
| Density index D | 1.1 | 1.14 | 0.76 | 0.28 | 0.82 |
| Dynamic penetr. test (N/10 cm) | > 80 | 76 | 20 | 2 | 30 |
| Standard penetr. test (N/30 cm) | > 130 | 120 | 43 | 11 | 60 |

A total of 9 anchors were specially prepared for the pull-out tests. In the laboratory temperature compensating strain gauges were attached to the steel bar at pre-selected points. A typical arrangement of the gauges along the bond length of tendon and the free tendon length is shown in Fig. 2. It is pointed out that the gauges were mounted on the profiled sides of the bar, which were carefully treated before the gauges were cemented to the steel-bar (see Detail A). To protect against the grouting, the strain gauges and wire connections were given several waterproof coatings. Each gauge was calibrated in the laboratory by stressing the steel bar.

The procedure of the installation of the anchors was as follows:
- the steel casing was driven to the depth of about 8 m;

Fig. 3. — One test series in sand dug out for careful examination.
- the anchor bars with fixed spacers (along the bond length) were inserted into the casing;
- the bore hole was grouted under constant pressure along the planned bond length while retracting the casing simultaneously;
- in the upper part of the bore hole the remaining cement suspension was flushed out with water;
- the pull-out tests were performed after a period of about 14 days.

The pulling force was applied in steps by a hollow ram jack until the failure load was reached. Between each step, loading and unloading cycles were applied according to the German Standards for Fundamental Tests (DIN 4125).

For each loading step, the displacements of the anchor head were measured manually through the readings of the dial gauge, while the measurements of the strain gauges were recorded automatically.

At certain loading steps, the force was kept constant with the help of an electric-hydraulic regulator for a period up to 2 months.

All the anchors were dug out after the pull-out tests (see Fig. 3) and the grouted bodies were carefully examined. The diameters and lengths of the grouted bodies were measured and the surrounding soil conditions were checked.

TEST RESULTS

Load Carrying Capacity

In Fig. 4 the failure loads (ultimate load carrying capacity $T_f$) of the five test series are presented in relation to the length of the fixed anchor (bond-to-ground length $L_a$). The results confirm the validity of the previously published design chart (H. Ostermayer, 1975) and supplement the chart with additional curves for loose gravelly sand and very dense sandy gravels. The diagram shows the smallest linear increase of carrying capacity with increasing bond length for loose sands. Contrary to this in case of dense sands, the greatest increase is encountered for smaller lengths which then tapers off steadily with increasing length. With lengths of more than 6 to 7 m the increase of carrying capacity per m length will probably be the same whether the anchors are installed in loose or in dense sands and gravels. The reason for this behaviour is a progressive failure mechanism which will be investigated in the following paragraphs.

In addition it should be noted that compared to the large influence of the soil density on the carrying capacity the influence of grouting pressure (minimum pressure 0.5 MN/m$^2$) as well as the influence of the diameter of the grouted body (diameter of 10 to 15 cm) seems to be negligible.

Distribution of Skin Friction

As a result of strain gauge measurements at anchors in dense sand a typical distribution of tensile forces in the steel tendon is represented in Fig. 5. The decrease of forces from the front part to the rear of the bond length corresponds with the load transmission from the tendon into the grouted body. As

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shown in Fig. 5 the forces in the tendon increase not only when the applied test load at the anchor head is increased (here 5 loading steps), but also when the load is kept constant for a period of time (in the example of Fig. 5 three loading steps were kept constant for about one day).

The difference in the values of forces measured at two adjoining points divided by the circumference area of the grouted body (grout-soil interface) gives the value of «skin friction». These calculated values are shown in Fig. 6. Obviously there are maximum skin friction values, the location of this maximum moves from the front part of bond length towards the anchor end when the test load is increased. The reason is that the elastic deformations of the steel tendon cause progressive displacements in the grout/soil interface. This progressive displacement causes the shear resistance of the dense sand to shift beyond the peak point into the region of lower residual shear values.

It is worth noting that at each loading step the same maximum friction value is reached for a short time (maximum «short-term skin friction») and that this value tapers off with time until a certain «long-term skin friction» value is not exceeded at any point along the bond length. As the applied test load is kept constant during one loading step, a decrease of skin friction in the front part of bond length will result in a corresponding increase of skin friction in the rear part. This kind of balancing could not be attained for the last loading step of 850 kN, so that failure occurred within 10 minutes.

For the load of 785 kN this balancing over a period of 300 minutes is illustrated in the three-dimensional plot of Fig. 7. The decrease and increase of skin friction along the bond length is shown for several points of time and demonstrated with the aid of shaded areas. It is pointed out that for any particular time the shaded area showing friction decrease in the front part is equal to the shaded area showing friction increase in the rear part.
When evaluating skin friction $\tau$ in the grout/soil interface from the bond stresses $\tau_b$ in the steel bar/grout interface (Fig. 8) it must be taken into consideration that the bond-to-ground length $L_v$ is longer than the bond length of tendon $L_w$. In the front part of the grouted body there is no steel/grout bond (due to the plastic tube), so that the forces resulting from skin friction $\tau_s$ in this area are transmitted further back and cause a concentration of bond stresses $\tau_b$ in the front part of bond length of tendon. These very high bond stresses which are obtained by strain gauge measurements are schematically shown in Fig. 8, where the existing $\tau_b$ is converted to an equivalent skin friction $\tau_s$ through the factor $d_b/d_o$ ($\tau_s = \tau_b \cdot d_b/d_o$).

The actual skin friction is achieved by equalizing the equivalent value over the entire length including the front part of the grouted body (see area $\Delta F$) in order to get the actual skin friction $\tau_s$.

In the case of dense sands the limit values of skin friction, max $\tau_s$, are effective along a relatively short length. For short anchors this length will correspond almost with the whole bond-to-ground length (Fig. 8a). For long anchors this length of max $\tau_s$ is only a part of the bond-to-ground length. The location of this part shifts towards the anchor rear when the test load is increased (distribution of skin friction near failure load is shown in Fig 8b). Assuming that the limit value max $\tau_s$ is identical for different bond-to-ground lengths, the mean values (mean $\tau_s$) for long anchors are smaller than for short anchors. This has already been presented in the chart of Fig. 4 in terms of carrying capacity versus bond length.

For the last loading step before failure load was reached Fig. 9 shows the «long-term skin friction» values of all test anchors in sand, which were equipped with strain gauges (diameter of grouted body being 9 to 12 cm).

For dense and very dense sand the skin friction values obtained experimentally fit very well with the qualitative distribution of skin friction of Fig. 8, thereby confirming the assumption made. The limit values, max $\tau_s$ of shorter anchors ($L_v = 2 \text{ m}$) however are likely to exceed the corresponding values of longer anchors ($L_v = 4 \text{ m}$). The difference may be partly traced back to the larger radial confining pressures in the front part of anchors.

In loose and medium dense sand the skin friction is found to be more or less constant along the whole bond-to-ground length. This corresponds with the stress-strain-behaviour of the sand for these densities.

The decisive influence of soil density is obvious in these tests when for example in the case of long anchors one compares the limit values of skin friction for loose and medium dense sand (about 150 and 300 kN/m$^2$ respectively) with the limit values for very dense sand (about 800 kN/m$^2$).

These high values of skin friction are mainly the result of an interlocking or wedging effect due to the dilatation of soil (E. Wernick, 1977). The peak values of up to 1 300 kN/m$^2$ do not represent the actual skin friction but the equivalent skin friction as already explained in Fig. 8.

Penetration Tests and Carrying Capacity of Anchors

As the density of non-cohesive soils is in current practice very often indirectly determined by penetrometer tests, it was decided to plot a diagram showing...
Fig. 10. — Relationship between carrying capacity, bond length of anchors and dynamic penetration resistance in two types of non-cohesive soils.

To investigate the important influence of the density of non-cohesive soils on the carrying behaviour of anchors it was for the first time that full scale field tests were performed in one and the same soil compacted to different densities. Under these controlled conditions the exact distribution of skin friction along the bond length could be calculated from strain gauge measurements.

On the basis of the results of the 5 series of tests comprising 30 anchors, the original design chart of 1975, showing carrying capacity versus bond length for different soil conditions, has now been complemented. Furthermore a diagram is presented from which it is possible to estimate carrying capacity of anchors with different bond lengths from the number of blows of standard penetration tests (SPT) and dynamic penetration tests. When using one of these charts it must be borne in mind that certain fluctuations in test results are possible due to the inhomogeneity existing in the soil at site, even when the anchors have been properly installed.

The different shapes of distribution of skin friction which were derived through measurements help to give an explanation for the influence of bond length of anchors and density of soils on the carrying capacity as shown in the design chart (Fig. 4).

The different shapes of distribution of skin friction of skin friction along the bond length with increasing load paved the way for the inclusion of valid assumptions in the calculation of carrying capacity in terms of soil constants.

In addition the variation of skin friction with respect to time was measured for several loading steps. The results (shown for only one anchor in Fig. 7) will provide a basis for anticipating the long-term behaviour of anchors.

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REFERENCES


NON LINEAR ANALYSIS OF THE ANCHOR-GROUND-WALL SYSTEM
Analyse non linéaire de l'ensemble ancrage-sol-paroi

by

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SOMMAIRE
Les dispositifs d'ancrages se sont montrés particulièrement économiques pour assurer le soutien des parois des excavations lorsqu'elles sont grandes. Le projet d'un mur flexible ancré est une opération complexe qui dépend de la connaissance de l'interaction entre l'ancrage, le sol et le mur. Dans la présente étude, on montre un procédé numérique qui prend en considération cette interaction.

Le premier but de l'analyse fait fut d'examiner au moyen de ce mode de calcul, la dépendance entre la force d'ancrage et un certain nombre de variables primaires comme la flexibilité du mur, la raideur de l'ancrage, la profondeur de la fiche et les conditions initiales des contraintes, variables qui régissent le comportement des murs ancrés à plusieurs niveaux. Les valeurs calculées de la réaction d'ancrages ont été comparées à celles prédites par les règles semi-empiriques usuelles.

Les solutions proposées dans cette communication sont basées sur l'analyse d'un grand nombre de situations. Elles sont utiles, à notre avis, pour l'évaluation par l'ingénieur de l'ensemble du problème du comportement d'une structure ancrée.

SUMMARY
For wide excavations the tied-back system has proved a particularly economical way of securing the walls of excavations. The design of a tied-back flexible wall is a complex operation which depends on a knowledge of the interaction between the anchors, the ground and the wall. In the present study, a practical numerical procedure taking into consideration this interaction, is introduced.

The primary purpose of the undertaken analysis was to examine, by means of this procedure, the dependence of the anchor force on some of the primary variables, such as the wall flexibility, anchor stiffness, embedment depth and initial stress conditions, controlling the performance of the multianchored walls. The computed anchor reaction values were compared with those predicted by means of the usual semi-empirical rules.

The paper findings, based upon a large number of analysed situations, are, in our opinion, useful for engineering evaluation of the over-all problem of the anchor systems behaviour.

INTRODUCTION

The tied-back wall is employed with increasing frequency as a form of support for deep temporary and permanent excavations, due to the improvement of grouted anchors which offer high bearing capacities at fairly low costs (Habib, 1969). Although the combined «anchor-ground-wall» system has been investigated extensively and the behaviour of anchors has been known for some time, the effect of various factors influencing the anchor force is not well documented.

The recent development of modern computers, which are able to handle the comprehensive programmes required to calculate the stresses and strains in a continuum with specified boundary conditions, has opened up a possibility of obtaining improved solutions to the problem of anchored flexible walls. These programmes which are based on the finite element method (Clough, Tsui, 1974) are still not developed far enough to be used for design purposes. Moreover their use is limited by the high computing cost.

In the present study, a practical numerical procedure to determine stresses and strains in an anchored flexible wall is introduced, taking into account the nonlinear soil and anchor behaviour. Comparison of the results obtained by this procedure with the finite element method results has shown good agreement.

The behaviour of single-anchored flexible wall has been studied both theoretically and experimentally by Rowe (1955). Rowe's design curves are accepted and used now, in general, by the profession.
As the use of multianchored walls developed, the empirical designs for multistrutted walls were applied, despite the fact that there were basic differences in behaviour between the two support systems.

For this reason it was decided to carry out a parametric analysis of the multianchored walls, using the proposed procedure (Popescu, 1977). This study is limited to the anchor force dependence on some of the variables which control performance of multianchored walls. The primary factors which have been included in the analysis are: (1) the wall flexibility; (2) the anchor stiffness, (3) the embedment depth; (4) the initial stress conditions.

**ANALYSIS PROCEDURE**

In structural mechanics, the flexure equation forms the basis of the theoretical analysis of the behaviour of a flexible elastic element, but the relationship between lateral pressure and deflection must be specified before such an approach can be used to analyse the deflections, moments and stresses in a flexible wall.

An efficient discrete-element solution for a flexible wall on elasto-plastic foundation has been presented by Haliburton (1968) and we will use an analogous model. The wall is divided into equal increments and effects are concentrated at the increment points along the structure. This procedure allows freely discontinuous variation of flexural stiffness, transverse and axial loads, and elasto-plastic foundation support along the wall.

However, the earth pressure variation law is a nonlinear function both in respect to deflection and depth. To represent nonlinear soil response, we consider that this response may be described at some point along the structure by a nonlinear lateral earth pressure-structure deflection curve, as seen in fig. 1.

Kondner (1963) has shown that the nonlinear stress-strain curves for a number of soils could be approximated reasonably accurately by hyperbolas. If we substitute the earth pressure deflection \( p-w \) relationship from eqn. 1 by two hyperbola branches, the relationship can be represented by two equations of the form:

\[
sp = sp_o - \frac{w}{E_{oa} + \frac{w}{p_o}} \quad (\text{for } w < 0) \quad (1a)
\]

\[
s_p = sp_o + \frac{w}{E_{op} + \frac{w}{p_p}} \quad (\text{for } w > 0) \quad (1b)
\]

in which \( sp_o \) = at rest pressure, \( sp_p \) = active pressure, \( p_p \) = passive pressure, \( E_{oa} \) = initial tangent modulus of soil in expansion, \( E_{op} \) = initial tangent modulus of soil in compression and \( \epsilon = 1 \). In the above equations the absolute values are considered for all parameters. The sign convention is given in fig. 1.

The relationships above apply to the soil mass behind the structure and above the dredge line. For the soil below dredge line, two different curves must be developed because passive pressure increase and active pressure decrease are occurring at the same time on different sides of the structure. The right-hand curve corresponds to that shown in fig. 1, whereas the left-hand curve is a symmetrical one with respect to the origin, but with different values. The corresponding equations for the left-hand curve are given by (1a) and (1b), in which \( \epsilon = -1 \) and \( p_o \) and \( E_{oa} \) values are interchanged with \( p_p \) and \( E_{op} \) values. The two curves are then automatically superimposed and a combined curve developed.

The deformation characteristics of the anchors can be introduced by a specified anchor yield value or, more realistically, by a specified anchor rod curve. As the force-deformation curve for anchor rods has a general form, it was considered more suitable to make provision that this curve be introduced by points.

---

**Fig. 1.** Nonlinear lateral earth pressure-structure deflection curve and sign convention.
To solve a flexible wall with nonlinear anchor-supports on nonlinear soil, repeated trial and adjustment solutions are made with a simple complete elastic wall solution in each trial. The convergence criterion consists of the comparison between two subsequent elastic lines of the wall. Consequently a digital computer solution was used.

To test the capabilities of applying this particular procedure to the anchored flexible wall problem, one of the walls included in Bjerrum’s State of the Art Report at the Madrid Conference (1972) was analysed and the comparative results are presented in fig. 2. The computed deformations, earth pressure distribution and bending moments are in fairly good agreement with those obtained from the finite element analysis. The results obtained from the «free earth support method» are also given in fig. 2.

The differences between the results of the method proposed in the paper and the results obtained from the finite element analysis are small and on the safe side. Taking into account its simplicity and the saving of the computation time, the numerical procedure used in this paper seems to be a good tool for obtaining a proper solution for anchored flexible walls.

CHARACTERISTICS OF THE ANALYSED MULTIANCHORED WALLS

It has been established that there are a large number of variables that influence the behaviour of an anchor-supported flexible wall. Single anchor walls has been investigated in detail by Rowe (1955). By introducing a coefficient of flexibility

$$\rho = \frac{H^2}{EI}$$

in which $H$ is the total height of the wall, $E$, its modulus of elasticity and $I$, its modulus of inertia, Rowe gave for the first time a quantitative relation between the anchor reaction and the flexibility of the wall. The important effect of flexure below the dredge level and flexure above the anchor level, was pointed out.

In designing the multianchored walls, the empirical rules for multistrutted walls are often applied, despite the fact that the experience underlying the recommended rules is meagre. By assuming a trapezoid (Tschebotarioff, 1951) or a rectangular (Peck, 1969) earth pressure distribution, the spacing of the tie-back is adjusted so that the same load is transmitted in each tie-back (fig. 3). The load-redistribution effects of wall continuity and the effects of yielding of each anchor-support are disregarded.

In order to supplement the small amount of empirical data available, it was decided to attempt to analyse the anchor reaction variation in respect to some of the primary influence factors, by means of the proposed procedure.

This study is based on the analysis of a 14 m length wall, having a 4 m embedment depth. The wall was considered supported by two, three and respectively four anchor rows. The layout of the systems is shown in fig. 4. The soil, with a nonlinear behaviour has the following characteristics: no cohesion, angle of internal friction $= 30^\circ$, unit weight $= 19 \text{ kN/m}^3$, initial tangent soil modulus in compression linearly increasing with depth, according to
Terzaghi (1955) expression:

\[ E_{op} = l_h (z/D) \]  

(3)

where \( l_h = 3000 \text{kN/m}^3 \), \( D \) is the effective embedment depth, \( z \), the equivalent soil depth taking into account the surcharge load. It was assumed that \( E_{oa} = 0.5 E_{op} \).

Initial stresses in the soil are taken as those corresponding to an at-rest condition with a lateral pressure coefficient \( K_o = 0.577 \).

In order to analyse the influence of wall flexibility on anchor reaction a large range of wall flexural stiffness values were incorporated in the study, namely from 28 850 kN/m² to 1 920 000 kN/m². Actual walls with similar stiffness values would range from a 2B type Larsen sheetpile to a 1.0 m thick concrete slurry wall. The corresponding range of the Rowe flexibility coefficient, \( \log \rho \), is \(-2.83 \text{ to } -4.65 \), \( \rho \) being introduced in Rowe’s original units (ft²/lb in² per ft).
Questions were raised concerning the wall stiffness influence on anchor reaction values. This influence can be analysed only in direct relation with the support stiffness.

The wall stiffness values considered in this study range from those corresponding to very flexible sheet pile walls to values corresponding to very stiff concrete cast-in-situ walls. It is probable that for the slurry trench wall cracking can occur, resulting in a substantial reduction in wall rigidity. In such cases the cracked moment of inertia is recommended for use in calculations.

In order to study the influence of anchor yield, three values of this parameter were considered in the present study, namely 0.0, 0.1 and 0.2% of the wall height. The results of the analyses are presented in fig. 5a, b and c, showing the calculated anchor reaction values versus wall flexibility for different anchor yields. The diagrams are self-explanatory and it may be observed that the anchor reaction values depend in a great degree on the wall flexibility and anchor yield.

For the zero anchor yield case, the lower tie-back is the most loaded and the reaction difference between this anchor and the upper anchors is very large. The more flexible the wall, the less low the anchor reaction.

As expected, the greater anchor yield results in a decrease of the lower anchor reaction. Also it results in a decrease of the upper anchor reaction, for the usual range of wall flexibility. For 0.1% and 0.2% anchor yield cases, the wall flexibility increase yields to an increase of the lower support reaction and a decrease of the upper support reaction.

If the double-anchored wall is analysed it may be observed that the two anchors are equally loaded for a particular value of wall flexibility. This particular value increases with anchor yield increase. For the walls with three and four anchor rows, there is no particular value of wall stiffness resulting in an equal load in each individual anchor.

As regards the anchor loads predicted by means of the semi-empirical rules treated by Tschebotarioff and Peck, they overestimate the upper anchor pull and underestimate the lower anchor pull.

The presence of tie-backs can be more realistically considered by introducing in the support points the corresponding nonlinear anchor deformation curves rather than an arbitrary anchor yield.

Tie-backs can vary in stiffness by a factor of 10 depending upon whether tie rods or multi-strand cables are used. In these analyses two values of anchor stiffness, EA, were considered namely 3.5.10^3 kN/m and 3.5.10^4 kN/m. The anchor nonlinear curve characteristics are based on the elasto-plastic behaviour of steel.

The effect of anchor stiffness on anchor reactions is shown in fig. 6 for all three analysed cases. The main outcome of the results presented in fig. 6 is that the stiff anchors are more unequally loaded than the flexible anchors.

When stiff tie-backs are used, the lower anchor is the most loaded one, whereas the upper anchor is the least loaded one, under all circumstances. The difference between anchor pull values may be as high as 80% for the flexible walls.
When flexible tie-backs are used, they are more uniformly loaded. For each case analysed, there is a particular wall stiffness value when the reactions of individual anchors are practically equal. This particular value corresponds to a log $p$ of $-3.75$, for the double-anchored wall and $-4.00$, for the walls supported by three and four anchor rows.

In general, as the wall flexibility increases, the upper anchor load decreases and the lower anchor load grows, for both anchor stiffness values. For the intermediate anchors, the anchor pull shows an increasing or a decreasing tendency, but these variations are not quite important. It may be also noted that the stiff anchor load is less influenced by the wall flexibility, when comparing with the flexible anchors.
The computed anchor yields are (0.0055 — 0.055) %H for the stiff tie-backs and (0.065 — 0.45) %H for the flexible tie-backs. It can easily be seen that a zero deflection (simple support) condition at each anchor location would have been too restrictive. The stiffer tie-backs reduce wall deflections, but the reduction is not in proportion to the stiffness change, since an increase in tie-back stiffness by a factor of 10 is required to cause a 30-60% reduction in movements.

Comparison of the computed anchor reactions with those estimated by means of the semi-empirical rules proposed by Tschebotarioff and Peck, again shows differences, which grow with wall stiffness decrease and anchor stiffness increase. As can be seen from fig. 6 these differences are not always on the safe side.
EFFECT OF THE INITIAL STRESS CONDITIONS AND EMBEDMENT DEPTH ON ANCHOR REACTION

In order to begin our computations it is necessary to define the initial horizontal stresses which are assumed to correspond to an at-rest condition.

In the determination of earth pressure at rest, there constantly occur discrepancies between experimental and theoretical results. There were several authors who undertook research on the determination of the coefficient of earth pressure at rest for granular soils, and presented different equations (Myslivec, 1972). Using these equations with our soil characteristics, the following values of the coefficient of at rest pressure were obtained: Jaky: \( K_0 = 1 - \sin \theta = 0.5 \); Brooker and Ireland: \( K_0 = 0.95 (1 - \sin \theta) = 0.475 \); de Wet: \( K_0 = (1 - \sin^2 \theta)/(1 + \sin^2 \theta) = 0.5 \); Siedek: \( K_0 = 0.75 (\pi/4 - \theta/2) + 0.25 = 0.499 \); Wierzbicki: \( K_0 = tg^2 (\pi/4 - \theta/3) = 0.491 \).

Pruška (1972) showed that the conditions for the so-called pressure at rest are fulfilled for different values of vertical to horizontal pressure ratio, which form a conical zone, I (fig. 7). The lower and the upper limits of this zone are defined by the equations:

\[
K_{01} = tg (\pi/4 - \theta/2) \quad (4 \text{a})
\]

\[
K_{02} = tg (\pi/4 + \theta/3) \quad (4 \text{b})
\]

The zones II are the local plastic area zones and the zones III are impossible strength zones. For our particular soil \( K_{01} = 0.577 \) and \( K_{02} = 1.732 \).

It may be observed that Jaky's equation as well as the other previous equations, refer to the lower limit of the coefficient of pressure at rest.

The influence of the coefficient of earth pressure at rest and the influence of embedment depth on the anchor reactions, for the double-anchored wall, are illustrated in fig. 8. The anchors stiffness is \( 3.5 \cdot 10^3 \text{ kN/m} \).

As embedment depth increases the lower anchor reaction decreases, tending to a constant value, whereas the upper anchor reaction increases. For the very stiff wall, the upper reaction reaches a maximum value and then slightly decreases when embedment depth increases. The most important variations (40 - 100\%) of the anchor reaction values occur in the embedment depth range 2 to 4 m. It is to be noted that below the 4 m embedment depth the values of the tie-back reactions, so far constant, quickly differentiate, thus making the upper tie-back completely useless. As far as the lower one is concerned, this reaches the over high values given in fig. 8. The corresponding soil restraint is of a sinking support type.

The effect of \( K_0 \) increase is a reduction of the lower anchor reaction. During the same time, the upper anchor load increases, for the very flexible wall and decreases, for the very stiff wall. But the anchor load changes induced by the \( K_0 \) variation between the two extreme limits, are not greater than 15\% and are relatively independent of embedment depth.

Fig. 7. - Limits of the earth pressure at rest zones.

Fig. 8. - Anchor reaction versus coefficient of earth pressure at rest and embedment depth.
CONCLUSIONS

The design of a tied-back flexible wall is a complex operation which depends on knowledge of the interaction between the anchors, the ground and the wall.

The numerical procedure for computer analysis of the anchored walls used in this paper takes this interaction into consideration. Nonlinear anchor behaviour and nonlinear soil response can be properly represented. Capabilities of the method have been illustrated by the good agreement between the method results and the finite element analysis results. It should be noted however that the accuracy of the method can obviously not be better than that of the input-data provided.

The primary purpose of this study was to examine, by a numerical method, how the anchor reaction is influenced by some of the primary factors which control the overall behaviour of multianchored flexible walls. This objective was achieved by considering the influence of the wall flexibility, anchor stiffness, embedment depth and initial stress conditions on anchor pull values.

In the design of tied-back wall systems the anchor pull evaluation is often based upon the semi-empirical earth pressure diagrams such as those suggested by Tschebotarioff and Peck. The comparison of the values obtained by these practical rules with the results of our analyses shows that the differences are not always on the safe side for the anchor forces evaluated by means of these rules.

It may be concluded that the computation method must be chosen on the basis of the extent to which it simulates the actual behaviour of real anchor-ground-wall system. Thereafter the value of the safety factor in respect to the ultimate pull-out capacity of the individual anchor and its application might well be left in the hands of the designer. This factor of safety allows for time effects, ground variability, repetitive loading and grouping action.

Despite the limitations related to the particular soil and structures considered in this analysis, it is believed that the findings of the paper are of general value for engineering evaluation of the overall problem of the anchor systems behaviour.

REFERENCES


INTRODUCTION

During the last few years the application of anchorages has become more and more widespread. The generalized use of these systems involves problems of various types. Among them the protection of anchorages against attack by corrosion and quality control on the job sites are of particular importance.

In the first part of this paper a new type of anchor called TPT, entirely protected against corrosion, is described. The results of two series of tests on prototypes of such anchor are reported. Prototypes in the first series have been installed in rock whilst those of the second series in normally consolidated clayey silts.

Anchors of the second series, in soft soils, were instrumented so as to measure distribution of stresses along the length of the grouted bulb. These measurements were also carried out on traditional anchor prototypes placed in the same soil thus allowing a comparison between the two anchoring systems.

1. DESCRIPTION OF TPT ANCHOR

As shown in figure 1 the characteristics of the TPT anchor may be summarized as follows:
- The TPT anchor consists of a number of cables placed all around a pipe equipped to perform repeated high pressure grouting.
- Each strand is protected by a plastic sheath down to the bottom plate, thus preventing any possible contact of the steel with waters present in the ground, even if fissuring of the grouted bulb occurs.
LONGITUDINAL SECTION

DETAIL OF THE BOTTOM ANCHOR DEVICE

LEGENDA

1. mortar of epoxy resins
2. cement grout
3. tendons protected with a plastic sheath
4. device for positioning the tendons
5. tube equipped with grouting device
6. reinforcement steel spiral
7. terminal steel plate
8. devices for fixing the steel tendons
9. tendons without plastic sheath protection

- The bottom plate equipped with strand fixing devices is embedded in epoxy resin mortar. In this way all the steel parts are thoroughly protected.
- The prestressing device may be any one of those employed for other traditional anchors.
- The TPT anchor belongs to the family of anchors with compressed grouted bulb in that the steel cables may run within the plastic sheaths and transfer the load to the bottom plate.

2. STRENGTH OF GROUTING MORTARS

A particularly important problem involved in grouted anchors, especially those with compressed bulb, is that related to strength of the injected mix.

This mix undergoes a state of triaxial stresses wherein the principal stresses are due to:

- The TPT anchor grouted bulb is carried out using the same construction method employed for the IRP anchor. (4)
- Hence it is possible to combine the advantages of repeated high pressure grouting with those relating to protection against corrosion both of cables and bottom plate.
distribution and magnitude of stress states in the grouted mix will also vary.

The author has carried out tests with a view to determining the strength range of a grouting mix of the type commonly employed, subject to triaxial stress states.

These tests were performed at ENEL (Government Electrical Agency) Laboratory at Niguarda, (Milan).

Tests were carried out on eight sets of specimens of a grouting mix with the following characteristics:
- size of specimens 10 × 10 × 10 cm;
- pozzolana cement 325;
- water/cement ratio 0.55;

Results of these tests are given in figure 2 where two curves are plotted, representing, in Rendulic's plane, the strength range limits of the grouting mix subject to triaxial stress states. The maximum stresses usually acting on the mix injected near the anchor bottom plate vary from 500 to 600 kg/sq.cm. By comparing these stresses with the stresses shown in figure 2 it will be seen that these states of stress do not cause breaking of the mix when the surrounding soil is able to exert such a confining pressure that the ratio between the principal stresses is equal to 0.3.

3. TEST ON PROTOTYPES

Tests on TPT anchor prototypes were carried out on two test sites representative of two limit conditions.

In the first case the soil consisted of rocks showing the best mechanical characteristics, while in the second case the soil was formed by clayey silts of very poor geotechnical properties. In both cases the aim of the tests was to provide information on the following points:
- verify that the TPT anchor actually worked as a compressed bulb anchor;
- verify that the strength of the injected mixes submitted to triaxial stress states was consistent with strength measured by the laboratory tests described in the preceding paragraph;
- compare the behaviour of compressed bulb TPT anchors with that of the traditional IRP anchors.

3.1. Tests of TPT anchors in rock on the Sorrento jobsite (Naples)

Six TPT anchors were built in soil consisting of calcareous rock.

The tests were carried out from July to September 1976. The characteristics of these anchors were the following:
- length: 25 m
- drilling diameter: 120 mm
- testing load: 117 t

The test load value is determined by the yield point of the steel. The tests were performed according to the procedure described by Portier (2), i.e. the cycle method. This system offers the possibility of interpolating the free length of the anchor taking into account the loading and unloading curves measured during the tests.

Measurements taken were highly satisfactory for they showed differences of 7% from the theoretical free lengths.

As an example, we show in figure 5 the stress/strain curve measured during the test on a TPT anchor.

The tests performed led to some conclusions which may be summed up as follows:
- in practice, the measured free lengths coincide with the theoretical lengths thus showing that the anchors actually behave in the same way as anchors with compressed bulb;
- stresses transferred by the bottom plate to the grouted mix were 800 kg/sq.cm. approx. and no failure phenomenon occurred.

This confirms the validity of values found through the experimental tests referred to in paragraph 2.

Following the successful outcome of the tests, TPT anchors have been employed in general in all jobsites involving anchors in rock which required a definitive protection against corrosion.

Fig. 2. — Variation of strength of grout under triaxial state of stress.

Fig. 3. — Rupture caused by uniaxial state of stress.
At present (March 1977) a jobsite where about 150 anchors of this type are being installed, is being set up. In this jobsite measurements of the free lengths are systematically carried out.

3.2. Tests on TPT anchor in soil of poor geotechnical characteristics

In February 1977 a series of tests were begun on TPT anchors, installed in soils of very poor geotechnical characteristics, that is alluvial clayey silt soils in the Po river delta.

The anchors were set out in the same site where traditional IRP anchors were under construction. This allowed comparison on the behaviour of the two types of anchors in the same soil.

A number of anchors were instrumented with removable strain gauges.

Measurement were taken by instruments and technicians of «Laboratoire Central des Ponts et Chaussées» of Saint-Brieuc (3). The tests made it possible to take measurements on stress distribution within the bulb on both TPT anchors (loaded by a bottom plate) and traditional IRP anchors where the load transfer to the bulb occurs by adhesion between steel and cement mix).

At present (March 1977) the testing programme is still under way, hence it is impossible to give general results on the behaviour of TPT anchors in soil of poor geotechnical characteristics.

Nevertheless we think it is of some interest to refer to the results of the strain gauge measurements detected along the bulb length, referred to in figure 6, 7 and 8.

In figure 6 a typical section of the work is given, showing both anchor position and points where strain gauge measurements were carried out. Measurements were taken only at the grouted bulb and strain gauges placed in the same position for all anchors.

In figures 7 and 8 strain measurements carried out both on a IRP anchor and a TPT anchor are reported.

A numerical evaluation of these results will form the subject of a future paper that will be published after completion of the present investigation programme. Here we are just making a few remarks of a qualitative character.

In figure 7 bulb deformations as a function of applied load for a TPT anchor are given. We have five load/deformation diagrams each referring to one of the points indicated in figure 6.

It should be noted how at the plate, the deformations are very high while they decrease towards the top of the bulb. This confirms the success of the anchor, from the technological point of view, in that working performance approaches theoretical performance, even in the range of the lowest loads.

In figure 8, curves relating to IRP anchor, similar to those in figure 7 are shown.

As already stated, in this type of anchor, load transfer occurs by adhesion of the strands to the grouted mix. Deformations decrease towards the end of the bulb.

It will be seen that the strain gauge placed near the inflatable packer on top of the bulb detects no deformations.

It is interesting to note that unit maximum per cent deformations, in anchors with bulb undergoing tensile stress, are much higher than those of anchors with compressed bulb. This proves the risk of bulb microfissuring, as already pointed out by various authors (Ostermayer, 1974, for instance) (5).
CROSS SECTION WITH ANCHORS
and distribution of deformations
as a function of loads

Fig. 6. — Cross section with anchors. Position of points of measurements.

Fig. 7. — Deformations in the ITP anchor. The loads are transferred to the bulb by adherence between tendons and grout.

Fig. 8. — Deformations in the IRP anchor. The loads are transferred to the bulb by a bottom steel plate. The measurement points are referred to those of fig. 6.
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REFERENCES


STRESSES AND STRAINS ON THE SURFACE OF ANCHORS
Contraintes et déformations à la surface d'ancrage

by

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SOMMAIRE

Des essais d'arrachement d'ancrages de sol de petit diamètre (5 à 15 cm, longueur 3 m) ont donné des résistances limites élevées que l'on ne peut pas expliquer à partir de la pression du poids des terres.

On a exécuté des essais à grande échelle, dans lesquels la résistance au frottement ($\tau_w = - \tau_s$) et la contrainte normale ($\sigma_s$) à la surface (rugueuse) de l'ancrage ont été mesurées à l'aide de cellules. Les rapports entre les efforts de cisaillement et les contraintes normales ont donné des angles de frottement très élevés, qu'il n'est pas possible de déduire d'essais triaxiaux ou d'essais de cisaillement. De plus, des contradictions apparaissent si l'on tient compte des conditions cinématiques tout en retenant les hypothèses courantes au sujet de la coaxialité des contraintes et des déformations. Par contre, si on abandonne l'hypothèse de coaxialité tout en prenant en compte la dilatance des matériaux dans les conditions cinématiques existantes, l'analyse des cercles de Mohr pour les contraintes et les déformations permet d'éliminer toute contradiction dans les résultats des essais. Ces analyses théoriques prouvent que les effets de dilatation sont le facteur prépondérant qui détermine la force limite d'ancrages cylindriques.

On explique les modifications que l'on a été conduit à apporter aux essais classiques en laboratoire et les résultats ainsi obtenus confirment les considérations énoncées ci-dessus.

On discute finalement les conséquences fondamentales qui en résultent pour le calcul de la force limite d'éléments d'ancrage cylindriques.

SUMMARY

Pull-out resistance tests on cylindrical ground anchors with small diameters (5 to 15 cm) show a high bearing capacity not derivable from the overburden pressure.

To investigate this fact full scale tests were performed measuring skin friction ($\tau_w = - \tau_s$) and normal stress ($\sigma_s$) on the anchor surface (rough) with the help of installed cells. Analysing the ratio of skin friction and normal stress, large angles of internal friction were stated not to be derived from triaxial tests or from direct shear tests. Beyond that contradictions appear if kinematic conditions and usual assumption of coaxiality of stresses and strain increments are taken into account. However, if coaxiality is disregarded and using Mohr's stress and strain circles with regard to the kinematic conditions concerned, the test results do not point out any contradictions. These theoretical considerations reveal that dilatancy effects are above all responsible for the bearing capacity of cylindrical anchors.

Modifications of conventional laboratory tests are explained and test results of a «true direct shear tests» are given confirming the considerations stated above.

The fundamental impact of these results on the calculation of the bearing capacity of cylindrical anchors is discussed.

INTRODUCTION

Pull-out tests of cylindrical anchors with small diameters (5 to 15 cm) reveal a bearing capacity (as a consequence of skin friction) much higher than what would be expected from the overburden pressure (H. Zweck (1970), H.-U. Werner (1972), H. Ostermayer (1975)). Further in many experiments it is observed (T.H. Hanna (1973), B.M. Das (1975)) that the skin friction does not exceed a limiting value beneath a certain depth.

It is difficult to analyse these phenomena because there are a lot of factors influencing the bearing capacity of ground anchors such as:

- overburden pressure;
- boring technique (ramming or boring);
- diameter and inclination of anchors;
- bond length;
- pressure of cement injection;
- rate of penetration of cement suspension into soil;
- soil density;
- grain size and grain-size distribution of soil.

This variety led to simplified tests to separate the influence of soil parameters and to state the author's opinion that the pulling resistance of ground anchors is decisively influenced by dilatancy effects.
PULL-OUT TESTS

Large scale tests were performed to determine the pull-out resistance of vertical cylindrical anchors. They consisted of pieces of steel pipe connected inside with threaded steel bolts. The bolts were provided with strain gages to measure the skin friction for each of the pipe sections. The anchors (diameter ≈ 5 to 10 cm with rough surface) are held vertically in a round test bin (diameter 2.5 m, height 3 m, see E. Wernick (1974)) and the bin is then filled with sand, varying density from test to test.

At the anchor surface cells were inserted (fig. 1) to measure the skin friction ($\tau_r = -\tau_{rz}$) and the normal stress ($\sigma_r$) at the same place. The details of the cells (with semiconductor strain gages) are given elsewhere (see B. Prange (1971)).

Fig. 1. — Arrangement of cells in the anchor.

CELL MEASUREMENTS

A representative result of cell measurements, obtained at a depth of $z = 2.12$ m below ground surface, i.e., surface of the sand body, is shown in fig. 2. Point «1» represents the anchor erected in the centre of the test bin; all stresses here are zero (reference stadium). During sand filling $\sigma_r$ increases to about 10 kN/m$^2$ (0.10 kp/cm$^2$, «2») which is in good agreement with the earth pressure at rest and can be calculated as $\sigma_r = K_o \cdot \gamma \cdot z = (1 - \sin \phi') \gamma \cdot z = (1 - \sin 40\degree) \cdot 17.14$ kN/m$^2 \cdot 2.12$ m = — 13.0 kN/m$^2$. But one may not be deceived by this agreement, because $\sigma_r$ is also influenced by «negative skin friction» caused by relative displacement between sand and anchor surface during filling, in this case $\tau_r = -5$ kN/m$^2$ (— 0.05 kp/cm$^2$) was measured.

During the following pull-out test — after overcoming the «negative skin friction» — $\sigma_r$ and $\tau_r$ increase

Fig. 2. — Typical stress path measured on the anchor surface during pull-out test.

Fig. 3. — Angles of shearing resistance obtained in different types of shear tests on the same sand.
rapidly until the maximum bearing capacity, $Z_{\text{max}}$, of the anchor is reached. During this period the angle of shearing resistance rises up to $\Phi'_p = 45^\circ (p = \text{peak})$. Continuing the test strain-controlled (<<17>> to <<23>>) $\Phi'$ dropped to a critical value $\Phi'_k = 34.5^\circ$. After relieving (<<27>>) skin friction vanished and $\sigma_{n} = 8 \text{kN/m}^2 (-0.08 \text{kp/cm}^2)$ corresponded to the earth pressure at rest again.

Two main points should now be noted:
- the increasing $|\sigma|_i$ clearly shows the wedging effect arising during pull-out test because only the pressure at rest existed before starting;
- very large angles of shearing resistance appeared.

The latter is not confirmed if these angles are compared with the results of the conventional shear box and triaxial tests conducted on the same sand (fig. 3). In fig. 2 the initial density index was $D = 1.1$

\[ D = (\gamma - \gamma_d)/\gamma_d, \quad \gamma_d \text{ unit weight in «loosest»}, \]

(\[ \gamma_d \text{ in «densest» state packing, see DIN 18 126).} \]

### THE «TRUE DIRECT SHEAR APPARATUS»

This discrepancy and the appearance of a shear band close to the anchor surface (pointed out in E. Wernick (1977 a)) led to the development of a «true direct shear apparatus» (fig. 4). The primary modification to the conventional box shear apparatus is that the loading platen is not permitted to turn, but can move in the vertical direction only. Three typical results of the «true direct shear test» are given in fig. 5.

During horizontal displacement $h$ —enforcing a horizontal shear band inside the sample—the angle of shearing resistance reaches a peak value $\Phi'_p$ and a residual one $\Phi'_k$ (fig. 5 a). $\Phi'_k$ corresponds to a critical

---

**Fig. 4. — Cross section and picture of the «true direct shear apparatus».

Fig. 5. — Results of «true direct shear tests».
density index inside the shear band not being influenced by the initial density index. Transfering these \( \Phi_p \) and \( \Phi_k \) values into fig. 3 and comparing especially \( \Phi_p \) with results of triaxial and conventional direct shear tests, a great discrepancy is observed again. But there is a good agreement between \( \Phi_p \) and \( \Phi_k \) measured by the cell (see rhombus in fig. 3).

The different values of \( \Phi_p \) and \( \Phi_k \) are to be explained by the varying kinematical conditions of the different shearing procedures: on the anchor surface and inside the shear band the kinematical conditions are more restricted than in triaxial tests (strains in two directions) and conventional shear box tests (turning of loading plate allows additional strains).

Only a few types of direct shear apparatuses avoid this additionally kinematical freedom as the direct shear machine with an automatic recorder, C.R.M. Maguin (1956) and likewise the improved direct shear apparatus of J.R.F. Arthur et al. (1977).

**STRESS AND STRAIN CONDITIONS**

It is not desirable to consider the stress and strain conditions on the anchor surface at first because strain increments are unknown except \( \delta s \), being zero (\( \delta = \) increment) assuming that the anchor is rigid and that the failure occurs along a shear band appearing at the anchor surface.

**«TRUE DIRECT SHEAR TEST»**

In contrast to this the strain conditions of the «true direct shear test» are known (and are comparable with those of the anchor tests as to be seen later):

- the failure occurs along a shear band following a zero extension line i.e. \( \delta s_z = 0 \) (fig. 6 a);
- assuming that all deformations are confined to the shear band the angle of dilatancy \( \upsilon \) can be calculated from fig. 5 b by \( \tan \upsilon = E_x/2E_y \) and\( \delta s_z = \delta/s, s = \) thickness of shear band;
- the third strain \( \delta s_z \), (marked with 2 to distinguish it from \( z \) in fig. 1) is zero (plane strain).

Using this information Mohr strain circles can be drawn. Test 1 from fig. 5 was chosen to draw the Mohr circles in fig. 6 b and 6 c. Fig. 6 b has been drawn for the stadium of peak friction, \( \upsilon \) being 13° at this stage (fig. 5 b). Fig. 6 c represents the strain circle for residual stadium, \( \upsilon \) being zero here.

To draw the Mohr stress circles a further assumption has to be made, because only \( \sigma \) (vertical stress) and \( \epsilon_p \) (shear resistance) are known. The usual assumption in soil mechanics is coaxiality of stresses and strain increments —yielding dotted circles in fig. 6 d (peak state) and fig. 6 (residual state). According to the Mohr-Coulomb criterion the plane of maximum stress obliquity gives an angle of peak shearing resistance \( \Phi^* = 33.1° \) and a residual one of \( \Phi^* = 41.7° \).

Comparing \( \tan \Phi_p = 1.53 \) (D = 1.1) and \( \tan \Phi_k = 0.89 \) with the results of shear tests in fig. 3, it is obvious that the assumption of coaxiality cannot hold in this case.

Instead of this we now make the assumption, that the failure follows a static characteristic i.e. a plane of maximum stress obliquity. In this way one can get the solid circles in fig. 6 d and 6 f. The angles of shearing resistance measured directly in the «true direct shear test» now correspond to the inclination of the tangents to these Mohr circles and the above contradictions vanish. The deviation of the principal axes of strain increments and stresses is (45° - \( \upsilon/2 \)) - (45° - \( \Phi_p/2 \) - \( \Phi_k/2 \) - \( \upsilon/2 \)) (the deviation would be zero, if coaxiality were valid).

It would be desirable to compare these somewhat unusual results with results of other investigations particularly with respect to

- large angles of shearing resistance;
- the assumption of non-coaxiality.

Peak angles of shearing resistance of the same magnitude were found by J. Vardoulakis (1977), who performed plane strain tests with the same sand. This agreement is at first surprising since the stress strain conditions in these tests run coaxial up to the peak differing from those in «true direct shear».

Hence the conclusion may be drawn that the Mohr-Coulomb criterion is valid in both cases with the same angle of shearing resistance irrespective of the deviation of the principal axes of strain increments from those of the stresses.

The investigations of J. Vardoulakis (1977) also confirm the assumption of non-coaxiality in shear bands under certain kinematic conditions. They prove theoretically (conceiving the formation of shear bands as a bifurcation problem) and experimentally that the non-coaxiality holds inside a shear band under kinematic conditions as is the case here.

J.R.F. Arthur et al. (1977) also found developing an empirical model and discussing data from different shear apparatuses that principal axes of stress and strain increment do not always coincide. They prove that the deviation of principal axes of stress and element forming thin rupture layers. For kinematical conditions as discussed here they also found a deviation as given in the last chapter.

**ANCHOR**

We can now consider the stress and strain conditions near the anchor surface again. Starting from the appearance of a shear band near the anchor surface we have to estimate the deviation of the actual strain state from plane strain. In doing this we assume that the first invariant of strain increments (volume change) in «true direct shear» (t.d.s.)

\[
\begin{align*}
1_{\text{st. idx.}} &= \delta \varepsilon_1 + \delta \varepsilon_2 + \delta \varepsilon_3 = \delta/s, \quad (\text{fig. 6 a})
\end{align*}
\]
is equal to the volume change inside the shear band near the anchor surface (a)
\[ I_{\delta \varepsilon x} = \delta \varepsilon_x + \delta \varepsilon_y + \delta \varepsilon_z = \frac{d}{dr} \frac{\delta u}{r} + \frac{\delta u}{r}, \] (fig. 7) (2)

Assuming that the first invariant of stresses \( I_\sigma \) has no influence on \( I_{\delta \varepsilon} \) or is of the same magnitude (sign convention of continuum mechanics used: for traction and extension positive signs) and setting
\[ I_{\delta \varepsilon x}^{\text{const}} = I_{\delta \varepsilon x}, \] (3)
we obtain the differential equation
\[ \frac{d}{dr} \left( \frac{\delta u}{r} \right) + \frac{\delta u}{r} = \frac{\delta u}{r}, \] (4)

Fig. 6. — Stress and strain analysis in the shear band.
whose solution with the boundary condition
\( \delta u(r_a) = 0 \)
\((r_a = \text{radius of anchor, fig. 7})\) is
\[ \delta u = \delta \varepsilon_i \left( r - \frac{r_a^2}{r} \right) \frac{2}{s} \]
\[(6)\]
Taking now the ratio \( \delta \varepsilon_i / \text{Dev}_{\delta \varepsilon \tau \text{ds}} \), \( \text{Dev}_{\delta \varepsilon \tau \text{ds}} \) being the deviator of strain increments of the «true direct shear test», as a measure for the deviation from plane strain we obtain.
\[ \frac{\delta \varepsilon_i}{\text{Dev}_{\delta \varepsilon \tau \text{ds}}} = \frac{\delta \varepsilon_i}{\frac{2}{s}} \frac{1 - r_a^2}{r^2} \frac{2}{\sqrt{2} s^2 + \frac{3}{2} \delta \varepsilon_i^2} \frac{\sqrt{3}}{s} \tan \nu \left(1 - r_a^2 \right) \frac{2}{2 \sqrt{2} \tan^2 \nu + 1.5} \]
\[(7a)\]
where
\[ \frac{\delta \varepsilon_i}{\delta h} = \tan \nu, (\text{fig. 6 a}). \]
\[(8)\]
In order to make a quantitative estimate of the ratio in (7) it is necessary to set a limiting value \( r_a / s \) and thus for \( r_a / r \). For reasons explained elsewhere (see E. Wernick (1977 a)) the ratio of anchor diameter to thickness of the shear band is limited to
\[ 2 r_a / s \geq 10. \]
\[(9)\]
Considering middle of the shear band \( r = r_a + s / 2 \) and inserting \( \tan \nu = 0.25 \) as the most unfavourable value, we obtain.
\[ \frac{\delta \varepsilon_i}{\text{Dev}_{\delta \varepsilon \tau \text{ds}}} = 3\% . \]
\[(10)\]
Even for these unfavourable conditions the deviation of the strain state inside the shear band from plane strain is very small and therefore negligible. Further it is interesting to consider three limiting cases in equation (7b):
- dilatation or \( \tan \nu \) is zero (for residual case for example);
- \( r \) approaches \( r_a \) and
- \( r_a \) is very large (with \( r = r_a + s \) the ratio \( r_a / r \to 1 \)).

For all these cases the plane strain condition is achieved.

These considerations reveal that results of the «true direct shear test» can be used to calculate bearing capacity of cylindrical anchors. Neglecting the above considered deviation of plane strain they permit determination of:
- the correct angle of shearing resistance;
- the correct angle of dilatancy and
- the magnitude of dilatation inside the shear band from (6), which for \( r = r_1 \) (fig. 7) is
\[ u_1 = u_1 = \frac{r_a + s}{r_a + s} \]
\[(11)\]
or at the end of pull-out test (in total)
\[ u_1 = u_1 = \frac{r_a + s}{r_a + s} \]
\[(12)\]
For $i$, in relation to initial density see fig. 5. The thickness of shear band for the sand used here is $s \approx 15 \cdot d_{50}$, $d_{50}$ being the average grain size, E. Wernick (1977 b).

Equation (12) quantifies the wedging effect in a very simple manner and it is now obvious that dilatancy causes skin friction not being a function of the overburden pressure. This is stated also by results of pull-out tests given in fig. 8.

We remark that the depth of initial constant skin friction is a function of its magnitude or of soil density index, which is not discussed in more detail here. Further more, from the distances of the average lines in fig. 8 it can be concluded with respect to written density index $D$— that skin friction increases more than linear with increasing density index. But considering the influence of density index it has to be remembered that the test having $D = 1.20$ was compacted by vibration whereas the others were built up by sand rain.

All these considerations reveal that a theoretical model to calculate the skin friction has to take into account the appearance of a shear band near the anchor surface and wedging effect caused by dilatancy. Such a model using results of «true simple shear tests» is under preparation and will be published elsewhere.

CONCLUSION

Analysing the stress and strain conditions on the surface of cylindrical anchors it was possible to prove that the high bearing capacity is caused by a wedging effect produced by dilatation. Furthermore large angles of shearing resistance were measured with the help of cells installed on the surface of anchors, which could not be confirmed by conventional shear tests. These values could, however, be confirmed by the test results of a «true direct shear apparatus». The apparatus also permits the determination of the angle of dilatancy inside a shear band.

Analysing stress and strain conditions of the «true direct shear test» it was pointed out that the usual assumption of coaxiality does not hold. Furthermore, the results of «true direct shear tests» are transferred to the anchor with help of invariants proving that the deviation of plane strain is negligible in the shear band. Wedging effect could thus be quantified and it is now obvious that skin friction is not a linear function of the overburden pressure but is mainly a consequence of wedging effect, being a geometrical effect. This led to the judgement that a reasonable ratio of grain size or thickness of shearband to diameter of anchor or corresponding measure has to be observed for model tests in granular materials producing stresses as consequence of hindered deformations. Above that the compressibility of the surrounding soil has to be imitated correctly. To avoid such difficulties the problem under discussion here was investigated in full scale.

The «true direct shear test» is of much more importance far beyond the anchor problem. Correct soil parameters can be ascertained for all problems where shear bands of corresponding kinematics appear or can be assumed. The higher shear resistance actually available could be used to obtain economic design in soil and foundation engineering. For this reason the introduction of the «true direct shear test» as a routine test is recommended. It is shown that this can be carried out by a simple reconstruction of conventional shear box apparatuses.

REFERENCES


LONG TERM ANCHOR HOLDING CAPACITY IN SATURATED CLAYS
Résistance à long terme d'un ancrage dans de l'argile saturée

by
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SOMMAIRE
Cette communication présente les résultats d'une étude en laboratoire sur le comportement du système ancrage-sol soumis à un chargement de longue durée dans de l'argile saturée, normalement consolidée.

Des sondes piézométriques miniaturisées enregistrent la pression interstitielle autour de l'ancrage et la dépression interstitielle (aspiration) sous l'ancrage. Les résultats expérimentaux ont mis en évidence des relations fonctionnelles entre le déplacement de l'ancrage, la charge de l'ancrage et la pression interstitielle associée au processus de consolidation.

SUMMARY
The results of a laboratory model study investigating the behavior of deeply embedded anchors are presented. The primary purpose of the tests is to study the behavior of the anchor-soil system under long-term loading in saturated, normally consolidated clay.

The tests were conducted in soil of two different shearing strengths. For each soil strength, three different long-term load tests were performed. Long-term loads applied were at stress levels equivalent to 25%, 50% and 75% of the short-term (undrained) anchor breakout capacity. Pore pressure responses around the anchor, including the negative pore pressure (suction) beneath the anchor, were monitored using miniature piezometric probes. The test results provide a functional relationship between anchor displacement, anchor load and pore pressure responses associated with consolidation processes.

INTRODUCTION
Although the use of anchors to moor ships reportedly began in the Bronze Age, according to Frost (1963), significant research in an organized manner on model anchor breakout resistance did not begin until the 1930's in the United States (Howard and James, 1933; Leachy and Farrin, 1935; Lucking, 1936). Increased ocean engineering applications have, in the last two decades, increased the amount of research devoted to anchor holding capacity considerably. The research results can be broadly classified into two categories. The first category treats the anchor problem mathematically. The transformation of classical cavity expansion solutions into anchor problems represents one form of this approach, e.g., Gibson (1950) for clays; Skempton, et al (1953) for sands; Ladanyi (1959) for sands, for clay (1967), for sensitive clays (1967) and in permafrost (1974); and Vesic (1971, 1972) among others. The finite element numerical simulation represents another mathematical approach, e.g., Sandhu and Wilson (1969); Christian and Boehmer (1970); Ghaboussi and Wilson (1971); and on-going research in the Civil Engineering Laboratory of the U.S. Navy.

Although it is possible to solve some important, characteristic anchor problems with a rigorous theoretical and/or numerical solution, the time and cost required usually makes this approach prohibitive for engineering applications. In most cases, the solutions depend upon many soil factors which are only known approximately or which have to be postulated. The anchor holding capacity problem appears to be a non-conservative mechanics problem. There are theoretical difficulties in solving non-conservative mechanics problems at this time. The second category of previous research is primarily related to model tests and/or field observations. Numerous articles are available particularly for sandy soils and for on-shore projects. Only limited data are available for model tests in cohesive soils and even less for off-shore projects, Maritopskii (1965); Adams and Hayes (1967); Meyerhof and Adams (1968); Bhatangar (1969); Bemben (1973, 1975); Colp and Herbich (1972); Meyerhof (1975) and Beard (1974). For anchors in clay, only a few had pore pressure measurements for the clay soil within which the anchors were embedded (Adams and Hayes and Beard). The authors are aware of only one, Beard (1974), that included the pore pressure measurements around the model anchor in a simulated condition of saturated submarine clayey soil.

From a practical viewpoint, a deep ocean embedded anchor in service will be under load for a long period of time. The long-term anchor-soil behavior is related to the consolidation and flow of water into or out of the soil surrounding the anchor. Consequently, the pore pressure/pore suction and the migration of pore
water may have either a strengthening and/or weak­
ening effect on the surrounding soil and thus influence
the holding capacity of the anchor. Most deeply
embedded ocean anchorages will occur in saturated nor­
mally consolidated clays. Since little is known about
the pore pressure/suction response around the anchor
in such soils, this aspect of the soil-anchor behavior is
considered in this paper, i.e., the long-term behavior of
depth embedded anchors in normally consolidated,
saturated clays. Deep embedment is a condition
in which the ratio of the depth of embedment to the
diameter of the anchor is greater than 5.

TESTING PROGRAM

Soil conditions and equipment used

The soil used is a low plastic clay (CL/ML, according
to the Unified Soil Classification System) which has a
liquid limit of 23% and plastic index of 6. To insure
complete saturation, special mixing equipment is neces­
sary. A vacuum apparatus and the general test set-up
are schematically shown in figure 1, in which soil is
mixed with water to provide a deaired slurry. The
slurry is then consolidated under its own weight or
surcharged in a consolidation bin. Another apparatus
used is a modified concrete mixer in which soil is
mixed at 29% ± moisture content under vacuum then
consolidated under its own weight. The soil strength
profiles in which the anchor tests were performed are
shown in figure 2.

A disk-like stainless steel anchor with built-in piezo­
metric probe(s), figure 3, was embedded in the consoli­
dation bin. The consolidation bin has an elastic lining
to decrease side friction. «Wicks» were installed to
accelerate consolidation by radial drainage. Pore pres­
sure response was monitored by stainless steel piezo­
metric probes which have a 0.035 inch (0.089 cm) I.D.
and 0.065 (0.1588 cm) O.D. Two side ports were cut
in the closed end probe and the probes were inserted

Fig. 1. — Schematic diagram showing sample preparation
system.

Fig. 2. — Vane shear strength vs. depth
(1": 2.54 cm; 1 psf: 47.9 N/m²).

121
equal to the hydrostatic head of the soil in the consolidation bin, a state of normal consolidation is reached. An average of 2 to 3.5 weeks was usually required for complete consolidation, i.e. excess pore pressure less than 0.005 psi which is the limit of the pore pressure monitoring system.

Test results

Table 1 shows the types of model tests performed.

Table 1

<table>
<thead>
<tr>
<th>Testing Condition</th>
<th>Short-Term Capacity</th>
<th>Long-Term Tests (*)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N-Series</td>
<td>NQ</td>
<td>NL-1/4  NL-1/2  NL-3/4 (*)</td>
</tr>
<tr>
<td>Consolidated Under Own Wt. from Slurry</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-Series</td>
<td>SQ</td>
<td>SL-1/4  SL-1/2  SL-3/4</td>
</tr>
<tr>
<td>Consolidated Under Surcharge from Slurry</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D-Series</td>
<td>DQ</td>
<td>------  ------  DL-3/4</td>
</tr>
<tr>
<td>Consolidated Under Own Wt. at ave. W% = 29%</td>
<td></td>
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</table>

(*) Long-term tests under 1/4, 1/2 and 3/4 of Short-Term holding capacity; e.g. NL-3/4 indicates a long-term test under a load equal to 3/4 of short-term anchor capacity in a soil which was consolidated from slurry under its own weight.

A short term capacity test is defined as one in which virtually undrained soil conditions exist during incremental loading until the anchor is pulled out. Once the short term capacity for each testing series is determined, long-term tests with loads equal, 1/4, 1/2 and 3/4 are performed and they are designated as shown in table 1.

«N» and «S» series tests were performed with a 3" (7.62 cm) diameter anchor (fig. 3) while the «D» series was performed with a 1.75" (4.45 cm) diameter anchor.

Large pore pressure responses and significant displacements were recorded during the NL-3/4, SL-3/4, and DL-3/4 tests. However, as would be expected, when the load applied to the anchor decreases, both the displacement of the anchor and pore pressure response in the surrounding soil decrease as well. In terms of displacement, NL-1/2 is approximately 1/10 that of NL-3/4 while NL-1/4 displacement is only 1/25 that of NL-3/4. The total displacement of NL-3/4 is 1/2 inch (1.27 cm). For the surcharge (S-Series) model tests, the displacement for SL-1/2 is only 1/250 of that for SL-3/4 and SL-1/4 displacement data is too small and erratic to be consistent with the other two. The total displacement of SL-3/4 is about 2.5 inches (6.35 cm). Generally speaking, the quality of test data varies according to the percent of the short-term failure load applied. The best quality data is at 75% of the short-term failure load, for which significant changes in pore pressures and displacements were recorded. The data at 25% of short term failure loads are generally too small and are not very consistent. Consequently, only NL-3/4, SL-3/4 and DL-3/4 have been selected for a thorough analysis. Testing results from NL-1/2 are included in the analysis where appropriate.

Pore pressure versus displacement

The test results are analysed in terms of dimensionless parameter so that preliminary generalized conclusions can be made regarding the soil-anchor behavior during loading.
Fig. 4. — Normalized pore pressure responses vs. normalized anchor displacement for 75% short term holding capacity.

area. The corresponding anchor displacement is normalized by the total displacement (δmax) for the given test. For example, figure 4 is a representative plot of normalized pore pressure (U/P) vs. normalized displacement (δ/δmax) for the probe located on the bottom surface on the anchor. This probe consistently measured the largest negative pore pressure (or pore suction) during all short and long term tests. The pore pressure responses in the surrounding soil corresponding to different anchor displacements, is shown in figure 5 with the use of normalized pore pressure contours. The continuous shifting, expanding and contracting of the pore pressure responses shown in figure 5 demonstrates the complexities in the soil response for an anchor under long term loading. It will be a formidable task to develop a complete solution which describes this phenomenon. Instead, a simplified mathematical model which is needed for practical engineering applications will be presented in the Analysis section.

Fig. 5. — Normalized pore pressure contours is anchor displacement ratio for NL - 3/4.
1969. The conical surface varies. If the conical surface is too small the product of the frictional resistance \((P_{N'}/\tan \Phi')\) and \(A_u\) is less than the anchor load, a greater conical surface has to be mobilized until the product of frictional resistance and the enlarged \(A_u\) is equal to the anchor load. This mobilizing process exhibits itself qualitatively in figure 5. During this process, the soil consolidates and shears under large strain due to the ascending anchor. Consequently, the conical surface of figure 8 advances against the resistance, \(P_{N'}\) due to soil overburden, \(P_o'.\) \(P_{N'}\) may be approximated by the passive resistance \(P_{N'} = K_p \cdot P_o' \cdot \cos \theta\) \((2)\)
in which \(K_p\) is the coefficient of passive pressure.

Combining equations (1) and (2) and noting that \(A_u = \frac{A}{\sin \theta}\) in which \(A\) is the circular area of the anchor, the long term anchor load can be expressed as follows:

\[
L = K_p \cdot P_o' \cdot \tan \Phi' \cdot (A_u^2 - A^2)^{1/2} \tag{3}
\]

Or, in terms of average anchor contact pressure

\[
P = \frac{L}{A},
\]

\[
P = K_p \cdot P_o' \cdot \tan \Phi' \cdot \left(\frac{A_u^2}{A^2} - 1\right)^{1/2} \tag{4}
\]

For the tests performed, all terms of equation (4) are known except \(A_u\).

Figure 9 shows the relationship of \(A_u\) as a function of soil liquidity index. It can be seen that \(A_u\) increases with the stiffness of the soil and increases with the magnitude of the load applied. Figure 9 also shows data from prototype tests performed by the Civil Engineering Laboratory of the U.S. Navy that check satisfactorily with this study. Although additional studies are needed to further define \(A_u/A\), equation (4) appears to provide an engineering estimate on anchor holding capacity under long term load.

CONCLUSIONS

1) There are two distinct stages of soil-anchor interaction under long term loading. The parameters in the relationship of \(U/P\) versus \(S/S_{\text{max}}\) appear to be significant in describing this phenomena.

2) For the soil tested at liquidity index greater than 1, there is no evidence indicating that the long term capacity is smaller than short term capacity.

3) A simplified mathematical model has been developed to provide an engineering estimation for the long term anchor load in normally consolidated clay. This model involves the effective angle of internal friction and depends on liquidity index. The model compares well with anchor performance in field studies.

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REFERENCES


# TABLE DES MATIERES

<table>
<thead>
<tr>
<th>Auteur(s)</th>
<th>Titre</th>
<th>Pages</th>
</tr>
</thead>
<tbody>
<tr>
<td>R.H. BASSETT (Great-Britain)</td>
<td>Underreamed ground anchors</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>Ancregés dans le sol avec élargissement</td>
<td></td>
</tr>
<tr>
<td>M.P. BOON - W.H. CRAIG (Great-Britain)</td>
<td>Model ground anchors under gravitational and centrifugal accelerations</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>Modèles réduits d'ancrages au sol en gravité normale ou en centrifugeuse</td>
<td></td>
</tr>
<tr>
<td>M. BUSTAMANTE - F. DELMAS - J. LACOUR (France)</td>
<td>Comportement des tirants précontraints dans une argile plastique</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>Behaviour of prestressed anchors in plastic clay</td>
<td></td>
</tr>
<tr>
<td>A.J. da COSTA NUNES - P.H.V. DIAS (Brazil)</td>
<td>Experimental verification of an anchored curtain wall</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>Vérification expérimentale d'un rideau ancré</td>
<td></td>
</tr>
<tr>
<td>A. EVANGELISTA - G. SAPIO (Italia)</td>
<td>Behaviour of ground anchors in stiff clays</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>Comportement de tirants d'ancrage en argiles raides</td>
<td></td>
</tr>
<tr>
<td>Y. FENOUX (France)</td>
<td>Enceinte étanche de la centrale électrique de Blaye : essais et mesures sur les tirants d'ancrage</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>Ground anchors: tests and measurements</td>
<td></td>
</tr>
<tr>
<td>K. FUJITA - K. UEDA - M. KUSABUKA (Japan)</td>
<td>A method to predict the load-displacement relationship of ground anchors</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td>Modèle pour calculer la relation charge-déplacement des ancrages dans les sols</td>
<td></td>
</tr>
<tr>
<td>P. HABIB - M.P. LUONG - D. AUGER - Y. TCHENG (France)</td>
<td>Détermination de la longueur libre optimale d'un tirant d'ancrage soutenant une paroi</td>
<td>63</td>
</tr>
<tr>
<td></td>
<td>Determination of the optimal free length of the line of an anchor supporting a wall</td>
<td></td>
</tr>
<tr>
<td>A.S. KANANYAN - M.I. NIKITENKO - Y.A. SOBOLEVSKY - V.N. SUKHODOEV (URSS)</td>
<td>Base calculation of anchor foundations using approximate model testing</td>
<td>69</td>
</tr>
<tr>
<td></td>
<td>Calcul des fondations d'ancrages à l'aide des modèles approximatifs</td>
<td></td>
</tr>
<tr>
<td>H. KRAMER (Germany)</td>
<td>Determination of the carrying capacity of ground anchors with the correlation and regression analysis</td>
<td>76</td>
</tr>
<tr>
<td></td>
<td>Calcul de la force portante des tirants d'ancrage à l'aide de l'analyse de la corrélation et de la régression</td>
<td></td>
</tr>
<tr>
<td>G.S. LITTLEJOHN - D.A. BRUCE (Scotland) - W. DEPPNER (Great-Britain)</td>
<td>Anchor field tests in carboniferous strata</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td>Essais en place d'ancrage dans un sédiment carbonifère</td>
<td></td>
</tr>
<tr>
<td>B.K. MAZURKIEWICZ - T. NAJDER (Poland)</td>
<td>Contribution to loading test procedure of ground anchors</td>
<td>87</td>
</tr>
<tr>
<td></td>
<td>Contribution à la méthode d'essais des tirants d'ancrage</td>
<td></td>
</tr>
<tr>
<td>H. OSTERMAYER - F. SCHEELE (Germany)</td>
<td>Research on ground anchors in non-cohesive soils</td>
<td>92</td>
</tr>
<tr>
<td></td>
<td>Etude de tirants scellés dans des sols pulvérulents</td>
<td></td>
</tr>
<tr>
<td>M. POPESCU - C. IONESCU (Romania)</td>
<td>Non linear analysis of the anchor-ground-wall-system</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>Analyse non linéaire de l'ensemble « ancrage-sol-paroi »</td>
<td></td>
</tr>
<tr>
<td>C. MASTRANTUONO - A. TOMOLIO (Italia)</td>
<td>First application of a totally protected anchorage</td>
<td>107</td>
</tr>
<tr>
<td></td>
<td>Première application d'un ancrage à protection totale</td>
<td></td>
</tr>
<tr>
<td>E. WERNICK (Germany)</td>
<td>Stresses and strains on the surface of anchors</td>
<td>113</td>
</tr>
<tr>
<td></td>
<td>Contraintes et déformations à la surface d'ancrage</td>
<td></td>
</tr>
<tr>
<td>B.C. YEN - S.J. YOUNG (USA)</td>
<td>Long term anchor holding capacity in saturated clays</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>Résistance à long terme d'un ancrage dans de l'argile saturée</td>
<td></td>
</tr>
</tbody>
</table>

128
numéro spécial

ANCRAGES DANS LES SOLS
GROUND ANCHORS

Avec la participation des Comités français de:
- Mécanique des Sols.
- Mécanique des Roches.
- Géologie de l’Ingénieur.