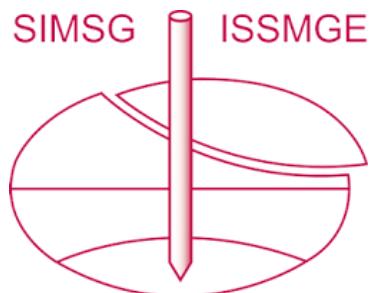


INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Soil Improvement

Amélioration des Sols

Chairman
Co-Chairman
General Reporter
Co-Reporter
Technical Secretary
Panelists

H. Cambefort (France)
J.A. Jimenez Salas (Spain)
J.K. Mitchell (USA)
R.K. Katti (India)
S. Knutsson (Sweden)
O.G. Ingles (Australia), W. Mackechnie (Zimbabwe), I. Stanculescu (Romania)
T. Yamanouchi (Japan)

H. Cambefort, Chairman

INTRODUCTION

Je suis avant tout un ingénieur qui a passé une grande partie de sa vie professionnelle à se battre contre diverses anomalies de tous les sols possibles et des écoulements souterrains, pour ne pas parler des accidents. Cette activité, associée à l'étude des coulisseaux d'injection m'a conduit à une certaine philosophie dont les aspects d'ont rien d'original mais méritent d'être rappelés:

- 1) les essais sur échantillons remaniés sont indispensables pour accroître nos connaissances. Mais leurs résultats ne peuvent pas être appliqués directement aux sols réels, ou encore intacts, dont le squelette est différent.
- 2) tous les processus d'essais sont standardisés afin de permettre la comparaison des résultats, parce que ceux-ci dépendent du mode opératoire. Et il est bien rare que celui-ci soit conforme à la réalité des cas étudiés. Les caractéristiques déduites des essais ne doivent donc être considérées que comme de bons ordres de grandeur.

3) dès qu'on est en présence de grains ultra-microscopiques, il faut raisonner en fonction de la physico-chimie des colloïdes. C'est très difficile et souvent oublié quand on est imprégné de physique et de chimie classiques.

4) enfin la mathématique aussi savante soit-elle, est analogue à la machine à faire la saucisse. On ne trouve à la sortie que ce que l'on a mis à l'entrée. Il est donc impératif d'avoir des hypothèses aussi exactes que possible. Il faut redouter leur simplification souvent nécessaires pour faciliter les calculs, ainsi que les hypothèses implicites évidentes pour le calculateur, comme par exemple, la structure uniquement granuleuse de toutes les argiles. Il est en effet bien difficile de prévoir si le passage dans la "machine" ne va pas conduire à des résultats peu conformes à la réalité.

Ces simples remarques sont certainement plus importantes pour l'amélioration des sols, que pour calculer une fondation. Dans ce dernier cas on a toujours le coefficient de sécurité qui pardonne bien des erreurs d'appréciation ou de calcul, tandis que dans le choix d'un procédé d'amélioration il ne faut pas faire d'erreur de principe. Ce sont les propriétés physiques des sols en place qui conditionnent la méthode et ses limites d'application.

Les procédés susceptibles d'être utilisés sont très nombreux. Un certain nombre d'entre eux n'a pas été retenu pour cette session, parce que peu utilisés comme par exemple le noyage des fouilles creusées dans du loess ou trop bien connus comme le compactage par couches minces.

Il semble que parmi tous ces procédés, un seul résulte des théories de la Mécanique des sols. C'est celui qui consiste à exécuter des drains verticaux afin d'accélérer la dissipation des surpressions d'eau interstitielles mises en évidence par Terzaghi. Tous les autres proviennent du sentiment, du bon sens ou encore de l'Art de l'Ingénieur. C'est ainsi qu'il est évident qu'en diminuant le volume des vides par compactage ou en remplissant ceux-ci par injection avec un corps rigide, on ne peut qu'améliorer le sol. On peut aussi lui donner la résistance à la traction qui lui manque en lui adjoignant des armatures et, enfin, essayer de reproduire un sol naturel à peu près acceptable comme la marne par exemple, en introduisant des produits chimiques adéquats.

Malheureusement, le bon sens ne suffit pas et l'absence de théorie se fait quelquefois nettement sentir. Par exemple:

- il est arrivé plusieurs fois que les drains verticaux ont bien diminué la pression interstitielle, mais sans accélérer la vitesse de tassement (J.R. Lake - 1960).
- à Linköping, le remplissage compacté de petites tranches creusées pour la pose de canalisations à environ 2 m de profondeur, a tassé de 5 à 8 cm en deux ou trois semaines, pour attendre par endroits 15 cm au bout de 18 mois (U. Bergdahl et al. 1979).
- au nouveau port de Nice, la digue en construction s'est brutalement effondrée en provoquant un important raz-de-marée local, et cela malgré le contrôle de l'évolution des pressions interstitielles.
- l'efficacité du noyau argileux de petites digues en terre a disparu au bout de quelques mois (H.Cambefort 1976). Un exemple est donné dans la communication de F. van M. Wagener et al. qui indiquent comment la remise en état a pu se faire en utilisant du gypse. L'origine de ce phénomène est aussi rappelé dans la session 6 par P. Stone et al.

Cette énumération sommaire montre bien que si une amélioration des sols ne donne pas toujours entière satisfaction, elle peut quelquefois provoquer une catastrophe, comme à Nice. Aussi est-il absolument impératif d'essayer de comprendre pourquoi il peut en être ainsi. Je vais donc aborder ce sujet important, quoique cela conduise à une introduction un peu longue. J'indique tout de suite que mon but est de mieux préciser la structure de sols cohérents en tenant compte du résultat de certains essais de laboratoire.

Double couche

Il faut commencer par rappeler qu'il existe à la surface de contact d'un solide, immergé dans de l'eau, une pellicule d'épaisseur microscopique qui n'a pas les mêmes propriétés que l'eau ambiante. C'est la double couche dont les Prof. Mitchell et Katti vous indiqueront les

caractéristiques. On l'appelle souvent eau liée, et Terzaghi avait pris pour elle l'image de la glace, ou plus exactement d'une eau solide.

Contrairement à une idée répandue cette eau liée ne se rencontre pas qu'avec les petites particules. Ainsi avec des sables et graviers de 1 à 10 mm de diamètre, dont la surface spécifique est voisine de $6 \text{ cm}^2/\text{g}$, le coefficient de perméabilité égal à 2.10^{-2} m/s avec un gradient de l'écoulement au moins égal à 5.10^{-3} , est divisé par 2 quand le gradient de l'est par 10 (H. Cambefort 1948 et 1971). Une telle réduction ne peut provenir que de l'épaissement de la couche d'eau liée, consécutif à la diminution de la vitesse d'écoulement. Autrement dit, suivant sa vitesse, le courant érode plus ou moins cette couche. Mais cette érosion ne semble pas pouvoir être totale si l'on juge par les essais de P. Habib (1953) qui a constaté qu'avec une argile remaniée au benzène, qui ne peut pas donner une double couche, l'écoulement de celui-ci était de 700 à 1000 fois plus rapide que celui de l'eau.

Avec des grains de petites dimensions l'effet de la double couche devient prépondérant devant celui de la pesanteur. On le constate en faisant sédentifier dans de l'eau des grains de quartz d'un diamètre quasi-uniforme de 4 microns. Le pourcentage des vides est de 55%, alors qu'il est de 26%, soit à peu près la moitié, avec un empilement de boulets (J. Duclaux - 1953).

Gelée colloïdale

En diminuant de plus en plus la dimension des particules, il arrive un moment où la physique classique ne s'applique plus. Il faut la remplacer par la physico-chimie colloïdale. Par exemple l'or, qui est en métal inoxydable, à peu près inerte, est utilisé finement pulvérisé par les verriers de Murano, près de Venise, pour colorer le verre en rouge. Cette suspension ne présente rien de particulier, mais avec des grains ultra-microscopiques, obtenus par un procédé convenable, on a un hydrosol rouge rubis qui par adjonction d'un sel minéral, se transforme en une gelée violette qui passe au bleu (J. Duclaux - 1953).

Il convient de remarquer que ce n'est pas la composition chimique des grains qui intervient, puisque l'or est inerte, mais la nature des ions de l'eau des pores. La flocculation à l'origine de la gelée est provoquée en Mécanique des Sols dans la détermination de l'Équivalent de sable et au contraire évitée lors d'un essai de Sédimentométrie.

Avec des gels de silicate de soude P. Luong et al. (1977) ont montré que l'on réduisait leur fluage, ou encore que l'on augmentait leur durée de vie, en augmentant le dosage du réactif, ce qui a pour effet de neutraliser de plus en plus la soude qui accompagne toujours le silicate. Les ions étant toujours de même nature, seul leur nombre conditionne le fluage.

Compressibilité

Divers essais effectués sur quelques argiles par J.A. Jimenez Salas (1953 et 1972) et par R. Genevois (1977) font apparaître, en particulier, des compressibilités différentes suivant la nature du liquide interstitiel qui conditionne les caractéristiques de la double couche, dont l'importance est ainsi confirmée.

Par ailleurs, d'après N.N. Maslow (cf. B. Félix - 1980), le même tassement relatif, ou encore le même degré de consolidation d'échantillons d'épaisseur H différente nécessite une durée proportionnelle à une certaine puissance de H. Cette puissance égale à deux

lorsque la consolidation est uniquement régie par la filtration de l'eau, comme dans le modèle de Terzaghi, serait nulle pour le fluage pur. Elle peut être quelconque entre ces limites. Qui plus est, elle ne serait pas constante pour un sol donné, mais uniquement fonction de son indice de consistance. On essayera, plus loin, de donner une explication à ces résultats qui montrent que les effets de l'expulsion d'eau et du fluage sont indissociables. On ne pourrait donc pas calculer le tassement en additionnant les tassements partiels provoqués par chacun des ces phénomènes.

Fluage

Dans leur communication sur le préchargement H. Aboshi et al. montrent qu'à la fin de la consolidation primaire, la pression des pores n'est pas annulée sur la totalité de la hauteur de l'échantillon, comme le voudrait la théorie classique. De plus les mesures faites lors du remplacement de la charge initiale par une charge plus petite ne semblent pas pouvoir être interprétées sans faire intervenir le fluage, qui se manifeste indépendamment de la valeur de la pression interstitielle, comme G. A. Leonards l'a observé en réduisant la valeur des charges appliquées à l'oeedomètre au lieu de la doubler chaque fois.

On ne peut plus alors faire la distinction entre consolidation primaire et secondaire. Le fluage apparaît bien avant que la pression des pores soit annulée. C'est ce qui peut expliquer pourquoi certains sols tassent à la même vitesse, avec ou sans réduction de la pression interstitielle, c'est-à-dire avec ou sans drains verticaux.

On peut caractériser les essais de fluage de S.R. Meschyan en rappelant que le tassement en fonction du temps est progressif avec des charges faibles, alors qu'avec des charges fortes sa plus grande partie se produit relativement vite et n'augmente que lentement par la suite. Cela proviendrait d'une destruction de la structure du sol suivie par une reconstitution due à des effets d'adsorption, de thixotropie ou autres, alors que les faibles charges déformerait uniquement le squelette. Quoiqu'il ne s'agisse pas de fluage, cette différence de comportement explique l'efficacité du piétonnage intensif qui brise toutes les liaisons sans leur laisser le temps de se reconstituer, et finit par liquéfier l'argile. Les liaisons ne se rétablissent que lorsque le squelette est plus compact. On voit ainsi apparaître l'importance de la structure dont la simple considération permet d'expliquer certains aspects du fluage et l'efficacité de la consolidation dynamique qui n'ont aucun point commun.

Modèles mécaniques

Afin de mieux préciser les idées, on peut prendre des modèles mécaniques. Ce ne sont que des caricatures, mais elles ont l'avantage d'être très parlantes.

Le modèle de Terzaghi est un corps de Kelvin-Voigt avec un ressort et un dash-pot en parallèle. Ce dernier ne figure que l'expulsion d'eau interstitielle. Il faut donc ajouter un autre dash-pot en série pour faire apparaître le fluage. Même ainsi complété ce modèle ne convient pas pour les essais de Meschyan. On peut alors considérer le corps de Schrödoff utilisé par plusieurs auteurs pour l'étude de la stabilité des versants. Ce modèle comporte un dash-pot et un patin en parallèle, système auquel est associé un ressort en série. Le patin a l'avantage de correspondre à la rupture du squelette sous les charges fortes. Mais il semble bien qu'il faille ajouter un dash-pot en parallèle avec le ressort pour retrouver tous les résultats des essais.

Comme on le voit, on est rapidement amené à compliquer tous ces modèles, ce qui réduit beaucoup leur intérêt. Nous ne retiendrons donc pour le moment qu'une conclusion à la fois simple et très importante, à savoir: on n'a pas le droit de schématiser toutes les argiles avec un seul modèle, surtout quand il est aussi simplifié que celui de Terzaghi.

Structures de base

Les diverses structures des sols cohérents ne peuvent provenir que de la dimension de leurs grains. On s'en rend compte en considérant les cas extrêmes.

Avec des grains relativement gros, les forces de liaison intergranulaires sont peu importantes devant la pesanteur et il se forme un squelette granuleux un peu semblable à celui d'un sable.

Au contraire lorsque les grains sont ultra-microscopiques les forces de liaison deviennent prépondérantes et les micelles étant fortement soudées entre elles la structure granuleuse disparaît pour être remplacée par un assemblage de flocons, qui constitue en réseau rigide un peu analogue à une mousse. La dessication d'un gel de silicate de soude montre cet aspect. On arrive ainsi aux gelées colloïdales dans lesquelles il suffit de quelques pour mille en volume, de matière solide pour qu'une rigidité soit mesurable (J. Duclaux).

Les argiles de Mexico sont un très bel exemple de gelée argileuse. Leur teneur en eau de 300 à 400 %, correspond à un volume de matière solide voisin de 10 % et non de quelques pour mille. Et c'est sans doute leur cohésion de l'ordre de 50 k Pa (0,5 bar) qui les empêche de se liquéfier lors des forts tremblements de terre, quoique leur teneur en eau nettement supérieure à la limite de liquidité permette de les pomper quand on les a remaniées (C.B. Crawford - 1963).

Les gelées argileuses pures étant rares, il nous a paru amusant de considérer quelques gelées extrêmement courantes, comme par exemple le blanc de l'oeuf dur, le yaourt et la gelée de groseilles.

On constate ainsi que la cohésion n'est pas uniquement due à la quantité de matière solide. Avec le blanc de l'oeuf et le yaourt qui contiennent, eux aussi, approximativement 10 % de solide, la cohésion est d'environ 1,5 k Pa (15 millibar) pour l'oeuf et 0,2 k Pa (2 millibar) pour le yaourt, soit 30 et 250 fois moins que les argiles de Mexico. Signalons au passage que la dessication du blanc d'oeuf donne un résidu d'aspect comparable à celui du gel de silicate, mais beaucoup moins beau.

La gelée de groseille a une cohésion comparable à celle du yaourt (0,25 k Pa), alors que son volume de matière solide est de l'ordre de 50% au lieu de 10%. Le sucre qui se trouve dans cette gelée, et qui d'ailleurs, se caramelise au moment de la dessication, doit se comporter comme une charge inerte.

La cohésion des gelées provenant de l'intensité des forces de liaisons et non de l'imbrication des grains, aucun frottement interne ne peut se manifester. Ainsi, avant la rupture des liaisons du squelette, les gelées sont uniquement cohérentes.

Il est très difficile de remanier le blanc d'oeuf, mais avec la gelée de groseilles on y arrive facilement avec une spatule. On obtient alors une agglomération de grumeaux dont la dimension diminue lorsque l'agitation augmente. Le produit ainsi transformé possède encore une certaine rigidité contrairement à ce qui se passe avec

le yaourt, qui pratiquement se liquéfie tout de suite.

On peut penser que cette différence de comportement, qui se retrouve probablement avec les gelées argileuses, provient au moins en partie de la valeur de la synergie de la gelée, c'est-à-dire de la facilité avec laquelle la phase liquide se sépare de la phase solide.

Argiles boulantes

On remarquera que le comportement du yaourt est tout à fait analogue à celui des argiles boulantes (ultra-sensibles ou quick-clays) qui d'après I. Th. Rosengqvist (1955) proviennent d'un lessivage de leur eau interstitielle par l'eau douce. Liquéfiées par remaniement, une adjonction de chlorure de sodium leur redonne de la cohésion, ce qui montre l'importance de la double-couche. Pour expliquer ce comportement il suffit de supposer qu'au moment de la formation de l'argile les ions présents ont permis l'édition du squelette en château de cartes dont la stabilité n'est assurée que grâce à eux. La diminution de leur nombre par lessivage réduit l'intensité des forces de liaison et par suite la stabilité du squelette. Une cause, ou une accumulation de causes, suffit alors pour provoquer l'écroulement de la structure, entraînant la liquéfaction du sol. La rupture est brutale et affecte un grand volume de terre. Une étude détaillée de ce mode de rupture est faite par G. Aas dans la Session 11 de ce Congrès.

On explique ainsi l'extraordinaire glissement de Surte, près de Göteborg, où malgré une très petite déclivité de la surface du sol, quatre millions de mètres cubes de terre environ se sont déplacés en trois minutes sur 400 m de large et 600 m de long. Certaines maisons se sont déplacées de 150 m (B. Jakobson - 1952). Les vibrations dues au démarrage d'un train semblent avoir été la dernière cause de ce gigantesque mouvement de terres. Le glissement de Rissa décrit par O. Gregersen dans la Session 11 est tout à fait analogue, et a été déclenché par la surcharge d'un petit remblai.

La grande analogie entre ces glissements et celui du nouveau port de Nice conduit à penser qu'à Nice on se trouvait aussi en présence d'argiles boulantes. Et ceci n'aurait rien d'étonnant car une nappe artésienne d'eau douce lessive actuellement ces argiles, comme l'on a été autrefois les argiles scandinaves. Cet exemple montre que les argiles boulantes ne sont pas le propre de quelques pays, mais peuvent se trouver n'importe où; quand les conditions s'y prêtent.

Il résulte de tout ceci une conséquence extrêmement importante. La consolidation des argiles boulantes ne peut pas se faire avec un préchargement qui déforme le squelette sans améliorer sa résistance, comme O.Q. Golder (1972) l'a constaté. Seuls: une adjonction d'ions convenables, un piéonnage intensif ou des explosions peuvent réorganiser la structure et ainsi supprimer son instabilité latente, à moins que l'on utilise des colonnes balastées pour réduire l'effet des surcharges.

Structure réelles

Ces deux structures, l'une granuleuse, l'autre en gelée constituent des cas extrêmes dont l'association en proportions diverses conditionne les propriétés des sols cohérents.

Un exemple simple est fourni par les loess et certains limons non saturés, dans lesquels les grains sont collés par un peu de gelée qui se ramollit en présence d'eau. L'intensité des forces de liaison diminue et sous l'effet de la pesanteur ou d'une surcharge le squelette se réorganise en provoquant un tassement souvent

important. On explique ainsi la consolidation obtenue en noyant des fouilles.

On peut aussi, à l'image de certaines gelées d'orange, avoir des grains relativement gros en suspension dans la gelée, et non en contact les uns avec les autres. Ce sont alors les propriétés de la gelée qui sont prépondérantes. Comme celles-ci dépendent de la teneur en eau, on comprend pourquoi Maslov a trouvé que l'indice de consistance pouvait intervenir dans l'évolution de la consolidation.

Courbe oedométrique

Afin d'illustrer la différence de comportement des deux structures de base, je prendrai la forme de la courbe oedométrique. Avec la structure granuleuse cette courbe est semblable à celle que Biarez a obtenu avec des graviers dont l'arrangement se modifie sous la charge, alors que la courbe des gelées de Mexico, qui doit correspondre à une rupture de flocons, a été retrouvée avec des billes de verre qui se brisent brutalement lorsque la charge appliquée dépasse une certaine valeur.

Cet oedomètre sur les graviers est très intéressant, car il fait apparaître, dès le premier chargement, une pression de consolidation qui évidemment n'existe pas. Cette charge limite provient de l'imbrication des grains qui s'amplifie avec l'augmentation de la charge, conduisant ainsi à un échantillon apparemment surconsolidé. Par ailleurs, les gelées colloïdales ont une pression de gonflement égale à leur pression osmotique. Par exemple celle du caoutchouc en présence de chlorure d'éthyle est supérieure à 0,7 MPa (7 bar), et elle n'a rien à voir avec une pression de consolidation. On peut alors demander si, pour un sol intact, dans lequel la gelée colloïdale n'est pas détruite, la considération de l'état normalement consolidé est une bonne référence. En l'abandonnant on est moins étonné par la sous consolidation des argiles de Tunis et par la surconsolidation qui se manifeste avec le temps, comme l'a montré Bjerrum. C'est la résistance du squelette qui compte et non le degré de surconsolidation qui ne résulte que d'essais sur échantillons remaniés, soit volontairement, soit par la trop forte intensité d'une consolidation préliminaire qui écrase les flocons.

Contraintes effectives et totales

Pour continuer la comparaison on peut revenir aux modèles mécaniques. Le squelette granuleux serait alors schématisé par le solide de Kelvin-Voigt et la gelée par celui de Schrödorff, par exemple, dont le glissement du patin simule la rupture des flocons.

On arrive ainsi tout naturellement au calcul des contraintes effectives. Celles-ci sont évaluées à l'heure actuelle en se basant sur le solide de Kelvin-Voigt, alors que celui de Schrödorff, plus valable dans certains cas, justifie le calcul en contraintes totales, avant la rupture des flocons. D'ailleurs si la Mécanique des sols, avait eu à s'occuper de gelée de grosses, d'oeufs durs ou de yaourt, elle aurait mieux étudié le fluage qui conditionne la résistance à long terme, et n'aurait jamais évalué la pression interstitielle sans tenir compte de la raideur du squelette. Les mesures de Bjerrum (1973) qui donnent une pression des pores voisine de la moitié de l'augmentation des contraintes verticales, montrent bien que le squelette n'est pas toujours infiniment souple. On peut alors penser que la prise en compte de la raideur du squelette aurait permis d'éviter la catastrophe de Nice.

Il ne faut donc pas mettre en compétition le calcul en contraintes effectives, et celui en contraintes totales. Ce sont les propriétés du sol qui commandent. On en a, depuis longtemps, la preuve avec la vérification de la stabilité de talus rompus, qui n'est à peu près satisfaisante qu'en considérant les contraintes totales dans certains cas, et les contraintes effectives dans d'autres. Et pour expliquer pourquoi quelquefois l'une et l'autre de ces méthodes ne conviennent pas je ferai appel au fluage ou même au critère de rupture adopté pour les sols. Il est, en effet, très possible qu'avec certains sols il faille considérer pour ce critère, non les contraintes, mais les déformations ou les distorsions qui, d'après Reiner conviennent pour le solide de Kelvin. Déjà Crawford a trouvé qu'avec une argile de Leda la pression des pores sous charge constante augmentait proportionnellement à la déformation axiale. De son côté LO a montré que la pression des pores était une fonction de la plus grande déformation principale, indépendante de la pression de consolidation, de l'anisotropie du sol, et de la durée d'application des charges. Enfin, le point de vue de l'ingénieur a été abordé par F.E. Barata et F. Danziger (Session 11) qui ébauchent une méthode de calcul faisant intervenir les déformations, afin de prévoir l'apparition des fissures, annonciatrices des glissements, dans certaines roches altérées.

Etude des structures

Tous les curieux comportements, connus actuellement ne doivent pas être considérés comme de rares exceptions et il ne faut plus se cantonner dans les simplifications des ancêtres célèbres, tels Coulomb ou Terzaghi, sous prétexte qu'elles ont permis à la Mécanique des sols de voir le jour. Ces anomalies ne sont qu'apparentes. Elles font partie des propriétés de certains sols, dont l'étude conduira à subdiviser les sols cohérents en plusieurs sous catégories. Les distinctions telles que sols saturés ou non, plus ou moins surconsolidés, ou diaclasés sont secondaires puisque l'essentiel se trouve dans le comportement de la structure qui dépend des propriétés et de la quantité de gelée qui en fait éventuellement partie.

Une telle étude n'est pas facile, car il faut trouver une méthode pour définir les mélanges de gelée et de grains les plus caractéristiques. En principe la granulométrie doit permettre de déceler la gelée, si l'on admet avec Cuclaux que les grains colloïdaux ne dépassent pas 0,4 microns. Malheureusement A. Demolon (1948) estime que "les méthodes par délaysage peuvent laisser jusqu'à 60% de matière colloïdale sous forme d'agrégat". Le volume des grains est donc augmenté, au détriment de celui de la gelée. Pour s'affranchir de cet inconvénient il faut utiliser les ultra-sons comme moyen de dispersion (Millot), et avec la pince d'Andreasen on peut déterminer la granulométrie jusqu'à 0,05 micron à condition d'y consacrer huit jours.

Si comme il le semble à l'heure actuelle l'essentiel se trouve dans la quantité de matière colloïdale et non dans sa granulométrie il suffit de mesurer sa surface spécifique, qui inversement proportionnelle au diamètre des grains, augmente lorsque celui-ci diminue. A titre d'exemple voici quelques ordres de grandeur:

| | |
|---------------------------------|----------------------|
| - grains sphériques de 1 micron | 2 m ² /g |
| - kaolinite | 20 m ² /g |
| - illite | 80 m ² /g |
| - montmorillonite | 80 m ² /g |

Ces valeurs correspondent à la surface des grains, alors

que la méthode utilisant l'adsorption du bleu de méthylène donne la surface totale, c'est-à-dire la surface externe plus celle des feuillets constitutifs des micelles, quand l'eau peut circuler entre eux. On constate alors que la surface spécifique de l'illite n'est pas modifiée, celle de la kaolinite est doublée ($40 \text{ m}^2/\text{g}$) et celle de la montmorillonite décuplée ($800 \text{ m}^2/\text{g}$). En outre suivant son mode d'interprétation cet essai au bleu de méthylène donne la capacité d'échange de cations. Enfin Tran Ngoc Lan (1980) a montré qu'il existait une excellente corrélation entre le résultat de l'essai et l'indice de plasticité, tous les deux augmentant ensemble. On peut alors penser que la mesure des surfaces spécifiques externe et totale, associées à quelques analyses chimiques des micelles et de l'eau interstitielle, permettront de mieux comprendre le comportement des gelées. Mais on sera probablement amené aussi à inventer d'autres essais, comme l'on fait les japonais pour les sables, ainsi que l'expliqué le Prof. T. Mogami dans sa conférence au Congrès de Tokyo.

En définitive, en cherchant à faciliter le choix des procédés d'amélioration, et à déterminer leurs limites par raison d'économie, ou ce qui est plus important pour éviter des catastrophes, on est conduit à mieux définir la structure des sols, ce qui d'ailleurs ne peut être qu'avantageux pour les calculateurs.

Vous voudrez bien excuser cette longue introduction due à l'importance du sujet, qui à mon avis, mérite que la discussion qui va suivre soit dans la mesure du possible axée sur lui.

Bibliographie

- 1 - BERGDAHL U. - FOGLSTROM R. - LARSSON K.G. - LILJEKVIST P. (1979) How to limit the settlements in fills above pipeline systems - A preliminary survey (en suédois) - Swedish Geotechnical Institute No 7
- 2 - BJERRUM L. - (1973) - Rapport général Session 4 - VIII ICOSOMEF - (Moscou) - Vol.3 - p. III
- 3 - CAMBEFORT H. (1948) L'écoulement des liquides à travers les milieux pulvérulents - Travaux Juin 1948 - Traduction No. 51-3 - Corps of Engineers U.S. Army - Research Center Waterways Experiment Station - Vicksburg MISSISSIPPI
- 4 - CAMBEFORT H. - (1979) Introduction à la Géotechnique - Eyrolles - Paris -
- 5 - CAMBEFORT H. (1976) Ecrans d'étanchéité des digues - Ann. I.T.B.T.P. - Dec. 1976 (Sols et fondations No 135)
- 6 - CAMBEFORT H. (1976-a) Elements de rhéologie en Méchanique des sols - Ann. I.T.B.T.P. mars 1976 Sols et fondations No. 129 (71 références bibliographiques)
- 7 - CRAWFORD C.B. (1963) Cohesion in an undisturbed sensitive clay - Geotechnique 1963 - p. 132
- 8 - CUISSET O. (1979) Le potentiel électrocinétique des argiles. Influence de la salinité - Bull. des Labo. des P et Ch. - Nov. déc. 1979
- 9 - DEMOLON A. (1948) Dynamique du sol - Dunod - Paris
- 10- DUCLAUX J. (1953) Colloïdes et gels - Gauthier Villars - Paris
- 11- FELIX B. (1980) Le fluage des sols argileux. Etude bibliographique. Rapport de recherche Labo. P et Ch. No 93 mai 1980 (41 références bibliographiques)
- 12- GENEVOIS R. (1977) Chemical interactions on the compressibility of remoulded kaolin - IX ICOSOMEF (Tokyo) - 1/23
- 13- GOLDER Q.H. (1972) Tassements sans dissipation de la surpression interstitielle - Bull. Labo. P et Ch. - Le comportement des sols avant la rupture. No. spécial Juin 1972.
- 14- HABIB P. (1953) La résistance au cisaillement des sols - Annales I.T.B.T.P. - Janv. 1953 (Sols et fondations XII)
- 15- JAKOBSON B. (1952) The landslide at Surte on the Göta River - Swedish Geotechnical Institute No. 5 - Stockholm
- 16- JIMENEZ-SALAS J.A. et SARRATOSA J.M. (1953) - Compressibility of clays - III ICOSOMEF (Zurich) 2/25
- 17- JIMENEZ-SALAS J.A. (1972) Quelques aspects fondamentaux de la déformabilité des sols - Bull. Labo P et Ch. - Le comportement des sols avant la rupture - No spécial juin 1972
- 18- LAKE J.R. (1960) Pore pressure and settlement measurements during small-scale and laboratory experiments to determine the effectiveness of vertical sand drains in peat - Pore pressure and suction in soils - Butterworths Londres - p. 103
- 19- LUONG P. - GANDAIS M. - ALLEMAND P. (1977) Comportement mécanique des sols injectés aux produits chimiques. Rev. Française de Géotechnique - No. 2 Oct. 1977
- 20- PHAN THI HANG - BRINDLEY G.W. (1970) Methylen blue absorption by clay minerals. Determination of surface areas and cation exchange capacities - Clays and clay minerals - Vol.18 p 203-212
- 21- PILOT G. (1972) Rupture d'un remblai sur sols compressibles - Bull. des Labo des P. et Ch. - Sept Oct. 1972
- 22- ROSENQVIST I. Th (1955) Investigation in the clay electrolyte-water system - Norg. Geot. Inst. Publ No 9
- 23- ROSENQVIST I.Th. (1959) Physico-chemical properties of soils: soil-water systems - ASCE Journ. Soil Mech. Found. S.M. 2- April 1959
- 24- SÖDERBLOM R. (1957) Some investigations concerning salt in clay - IV - ICOSOMEF (London) 1a/27
- 25- TRAN NGOC LAN (1980) L'essai au bleu de méthylène - Bull. des Labo. des P. et Ch. - Mai Juin 1980

H. Cambefort, Chairman

COMPLEMENT AU "STATE-OF-THE ART"

La distribution au cours du congrès des rapports sur l'Etat des Connaissances n'a pas laissé le temps aux Professeurs J.K. MITCHELL et R.K. KATTI d'apporter à leur remarquable texte quelques petites précisions souhaitables et d'ailleurs en partie exprimées par le Professeur MITCHELL dans son exposé.

Comme j'ai eu l'occasion au cours de la séance d'exprimer mon point de vue sur le cisaillement des colonnes balastées, je n'intervient ici que sur l'électro-osmose et sur l'injection.

Pour expliquer la baisse de débit en fonction du temps observée dans l'électro-osmose, il n'est pas nécessaire de supposer l'existence d'un gradient hydraulique susceptible de créer un écoulement allant de la cathode vers l'anode. En effet des essais, effectués avec des électrodes en graphite pour éviter les échanges d'ions, montrent que le gradient de potentiel électrique (V/cm) n'est pas constant entre l'anode et la cathode. Il se forme un pic qui se déplace en fonction du temps, et le gradient entre celui-ci et l'anode diminue progressivement. Il suffit de prendre la valeur de ce gradient amont pour que le débit d'eau mesurée corresponde à celui donné par la relation 18 du texte (CAMBEFORT - CARON - 1961). La diminution du débit ne provient donc que d'un phénomène électrique concernant les ions de la micelle argileuse et de sa double-couche.

En ce qui concerne l'injection, la figure 24 est trompeuse. Le bulbe en b ne se forme qu'avec des coulis pâteux et n'est recherché que pour le scellement des tirants d'ancrage. Ce n'est plus de l'injection proprement dite.

Le croquis d'encapsulation (fig. 24c) est à beaucoup trop petite échelle. Les fracturations hydrauliques, ou encore claquages, sont en général de très grande surface, et peu épais. On sait maîtriser leur formation et profiter de ce qu'ils finissent par devenir horizontaux pour, par exemple, soulever ou remettre de niveau une construction. Un exemple en est donné par la figure 27, à condition de remplacer les bulbes par des claquages horizontaux.

Si la règle du pouce, concernant la pression maximale d'injection des roches fissurées, et donnée d'ailleurs avec quelques réserves, satisfait pleinement l'esprit, elle ne permet absolument pas de faire des injections satisfaisantes. En effet l'expérience montre que si effectivement on peut quelquefois soulever le sol avec de très basses pressions, elle montre aussi que plus la pression finale est élevée,

meilleur est le résultat. Il ne faut donc pas généraliser des exceptions et limiter, a priori, les pressions à des valeurs insuffisantes, mais éventuellement contrôler les soulèvements de la surface du sol. En général il suffit de conduire les travaux avec précaution en augmentant progressivement cette pression maximale afin d'éviter des désordres en surface. Il est alors de pratique courante de terminer avec une pression de 5 à 10 M. Pa.

Avec des pressions fortes, les fissures du rocher s'ouvrent légèrement, ce qui facilite la pénétration du ciment. Ainsi les règles données, dans le paragraphe "particulate grouts" ne sont que théoriques et en pratique on ne s'en occupe pas. En contre partie on vérifie la qualité du travail avec des forages de contrôle dans lesquels on fait des essais d'eau.

Le ciment ne pénètre convenablement dans les sables et graviers que si leur perméabilité est supérieure à 10^{-2} m/s. En dessous de cette valeur le rayon d'action de l'injection devient très petit, et l'on obtient ainsi l'équivalent d'un pieu diforme.

Les coulis d'argile-ciment pénètrent dans les sables et graviers dont la perméabilité est supérieure à environ 3.10^{-4} m/s. Pour des perméabilités plus petites, il est pratiquement impossible d'éviter les claquages. Il faut alors prendre des coulis chimiques, par exemple : silicate ou résine.

Cet écoulement d'imprégnation obéit à une loi tout à fait analogue à celle de DUPUIT pour les puits filtrants. La pression augmente donc avec le débit qui par suite, ne doit pas être trop grand pour éviter les claquages, même avec un coulis bien choisi. Après le claquage de la manchette éventuelle qui peut nécessiter de très fortes pressions, celles fréquemment pratiquées sont de l'ordre de 1 à 2 M. Pa quelle que soit la profondeur. Mais alors que dans l'injection des roches fissurées on cherche à atteindre une pression maximale, dans celle des sables et graviers on limite les quantités injectées.

REFERENCES

Cambefort, H. (1977). Principles and Applications of Grouting - Quarterly Journal of Engineering Geology - The Geological Society London, England, Vol 10, 1977, pp. 57-95. (Texte français - Ann. I.T.B.T.P. sept. 1977 SF. 144).

Cambefort, H. - Caron, C. (1961) Electro-osmose et consolidation électro-chimique des argiles - Géotechnique, London, sept. 1961.

R.K. Katti, Co-Reporter

A LEADING PAPER FOR PANEL DISCUSSION ON IMPROVEMENT OF COHESIVE SOILS

INTRODUCTION

Irrespective of the method of soil improvement adopted in improving the strength characteristics of cohesive soils, it is of prime importance to get an insight into the development of cohesion in the clay-water system. This development of cohesion in the clay-water system, in general, is attributed to the interaction between the surfaces of the clay particles, ionic concentration and water. The clay particles because of their structure exhibit electrical charge concentration of varying degree, at the surface, edges and corners. The presence of different types of ions in the clay-water system influences development of cohesion. Thickness of water hull around different types of ions is not the same.

Magnitude of electrical charge on the surface of the particle depends upon the structure and type of the clay mineral present. The dipolar nature of water contributes significantly to the interaction between clay and water. Non-polar liquids such as carbon tetrachloride and benzene do not develop cohesive bonds in a clay non-polar liquid system. This fact is amply indicated by various research workers.

The development of cohesion in a clay-water system is influenced by the factors such as, (i) structure of the clay mineral, (ii) nature of cations in the clay-water system, (iii) amount of clay content (iv) void ratio, (v) past history under which the cohesive bonds are developed, (vi) the stress changes brought about in the system subsequently, (vii) nature, type and properties of the media through which, forces are transmitted to the clay-water system, (viii) internal changes brought about by the application of electrical potential to the clay-water system and (ix) thermal and time effects.

Prior to dealing with the soil improvement techniques concerning strength-deformation characteristics of cohesive soils, an attempt is made to examine as to what extent the above factors are interrelated to each other with respect to the development of cohesion. The clay-water system containing non-expanding to feably expanding type of clay minerals are considered. Certain points arising out of this analysis may help in directing our attention to them during panel discussion.

STRUCTURE OF CLAY MINERALS

The common clay minerals are hydrous aluminium phyllosilicates, composed of tetrahedral silica units and aluminium octahedral units. These units are bonded together in different combinations to form different clay minerals.

In case of Kaolinite the structure consists of alternate layers of alumina and silica

tetrahedron with stronger ionic and covalent bonds at the apex and less stronger hydrogen bonds at the base. The base exchange capacity is 3-15 meq/100 g of soil (Grim, 1953).

The montmorillonite type of clay minerals are characterised by the alumina layer sandwiched between two silica layers. A common oxygen is shared between octahedral and tetrahedral unit. In between two montmorillonite clay mineral layers there exists a very weak bond and a clear cleavage due to the presence of oxygen in the adjacent units. The isomorphic substitution of metallic ions of lower valency for Al and Si in the montmorillonite structure has resulted into a net negative charge of 0.66 per unit cell. The clay particles are formed due to the stacking of these sheet minerals in 'C' direction. This results into expanding lattice in the 'C' direction. See Fig. 1. Base exchange capacity of clay fraction containing montmorillonite clay minerals varies between 80-150 meq/100 g (Grim, 1953). The structure of Illite is same as that of montmorillonite except that one sixth of the 'Si' is always replaced by 'Al' and the residual charge deficiency is balanced by the potassium ions occur between unit layers, where they just sit into the perforations in the surface oxygen layers. The base exchange capacity of these minerals is in the range of 10-40 meq/100 g.

The structure of mica is similar to Illite except that the substitution of Al for Si is only one fourth.

The above observations clearly show that the clay minerals possess electrical charges of varying magnitudes at the surface due to the basic characteristics of the structure itself. The montmorillonite clay mineral exhibits distinct expanding characteristic in the 'C' direction.

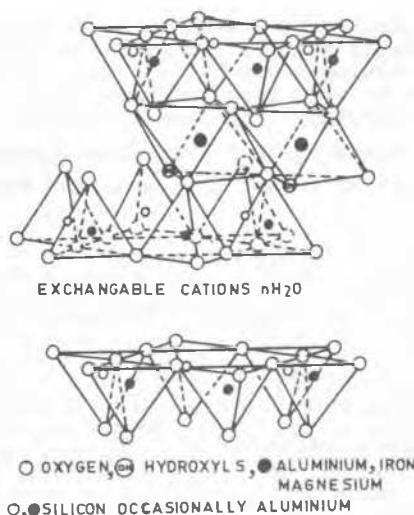


FIG.1. DIAGRAMATIC SKETCH OF THE STRUCTURE OF MONTMORILLONITE.

WATER

In water molecules the electrical centres of positive and negative valences do not coincide. This eccentricity results in electrical imbalance and hence the water molecule is considered as a dipole. The angle subtended at the centre is $104^{\circ}56'$. Refer Fig. 2. Under normal temperature conditions, the water molecules in a water media are supposed to have a structure designated as icosahedral symmetry and thus possess closest packing and extremely negligible cohesion tending towards zero. The molecules in the liquid water experience 10^{11} to 10^{12} reorientational and transitional movements per second (Fisenberg and Kauzmann, 1969). Raising the temperature of water, increases the rate of orientation and displacements and decreases the viscosity. In connection with this, limited studies relating to alteration in vane shear strength with temperature of saturated Bentonite clay system indicated that the vane shear strengths are 19, 18, 1 g/cm² corresponding to 4°C, 25°C and 85°C temperature.

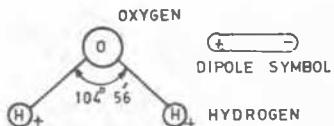


FIG.2. STRUCTURE OF WATER MOLECULE.

When the temperature of water is lowered the thermal movements of the molecule become less intensive, the electromagnetic properties of the water molecules prove to be stronger than these movements. The structure of ice then formed is similar to that of hexagonal form of silica. Each oxygen atom is surrounded tetrahedrally by four oxygen atoms with which it forms hydrogen bonds. See Fig. 3. Such a structure is very open and hence it has low specific gravity. The shear strength of ice at -5°C, based on half unconfined compressive strength is 15 kg/cm² (Leonards and Andersland 1960).

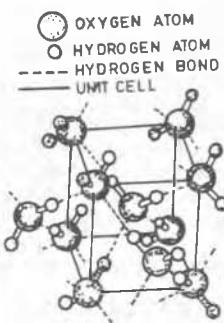


FIG.3. THE STRUCTURE OF ICE

These characteristics of water molecules clearly indicate that both the structure of water and bonding between molecules, are due

to the electromagnetic properties. From this, it appears that the bond between water molecules can be influenced by the presence of an external electromagnetic field.

Thus considerable alteration in bond strength in the water molecules adjacent to the electrically charged surfaces of clay particles and ions can be expected.

ADSORBED WATER

Clay fractions containing different types of clay minerals when treated with carbon tetrachloride or benzene do not exhibit any plasticity. Cohesion developed is negligible and is tending to zero value. However, the same types of clay fractions, exhibit significant magnitude of plastic limit, liquid limit and plasticity index properties in the presence of water as shown in Table 1.

This may be attributed to the alterations brought about in the structure of water and the bond between water molecules in the presence of electrical charges existing on the surfaces of clay minerals. It is also clear that higher the surface electrical charges the higher are the liquid limit and plasticity index values (Bauer et al, 1976).

TABLE 1.
Atterberg Limits for Different Clay Minerals
with Na and Ca Exchangeable Cations

| Clay | Ion | Atterberg limits | | | |
|-----------------|-----|------------------|------|------|------|
| | | L.L. | P.L. | S.L. | P.I. |
| Montmorillonite | Na | 710 | 54 | 10 | 650 |
| | Ca | 510 | 81 | 11 | 430 |
| Illite | Na | 120 | 53 | 15 | 63 |
| | Ca | 100 | 45 | 17 | 55 |
| Kaolinite | Na | 53 | 32 | 27 | 21 |
| | Ca | 38 | 27 | 25 | 11 |

This development of plasticity in a clay-water system is attributed to the formation of adsorbed water hull in and around the clay particles. This adsorbed water in turn may play an important role in the development of cohesion.

To understand the interaction in the clay-water system containing ions, attempts are made by various research workers by simplifying the surface of the clay particle having electrical charges on it and the liquid medium in which it is immersed.

First attempt in this direction was made by Helmholtz who considered an infinite plate, as shown in Fig. 4, immersed in a liquid media having counter ions. He showed that, the counter ions get attached to the surface of the solid and produce almost a fixed layer. It moves only when an appropriate potential is applied. (Maysels, 1967).

Later work by Gouy-Chapman shows that the compensating cations have a tendency to diffuse

away from the layered surface since their concentration will be lower in the bulk solutions. Refer Fig. 4. On the other hand, they are attracted electro-statically to the charged lattice. It is the thermal agitation, overcoming in part, the electrostatic attraction which causes the diffuseness of the double layer. Further work by Stern indicates that if the electro-static forces are too strong or if they are reinforced by Van der Waals forces, thermal agitation is not able to overcome them and a part of counter ions remain in such a compact layer. See Fig. 4. This compact part is called as Stern layer. This layer is in a dynamic equilibrium and in a sense is an integral part of the charged particle. Thus, stern layer takes into consideration the size of the ions. (Van Olphan, 1963).

The double layer theories indicate that a charged surface has a tendency to attract counter ions with varying degree of forces and even under normal temperature conditions, it is possible, that the force with which the cations are held by the solid surface may exceed thermal agitating force. On the basis of these theories, it is indicated that the water molecules closer to the charged solid surface are held with higher electro-static force than those away from it.

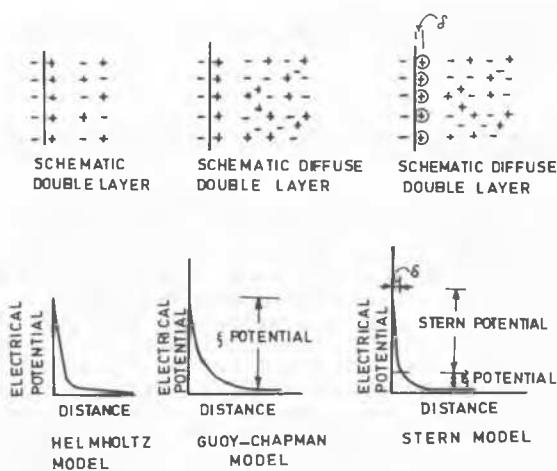


FIG.4. MODELS FOR ELECTRICAL DOUBLE LAYER AND POTENTIAL

A clay particle having charged surface is found to develop an adsorbed water layer around it when suspended in water. (Lambe, 1960). In the light of the above theories it may be expected that the influence of electrical charge on the clay particle would bring about varying degrees of bonding between the water molecules, depending upon the distance of the water molecules, from the clay surface, amount of charge on the clay surface, the presence of various types of ions in the water and temperature, etc. In the adsorptions of water on a natural clay surface both hydration and hydrogen bonding with the oxygen atoms on the surface may be involved (Van Olphan 1963).

The ions adsorbed on the surface of the clay minerals may affect the adsorbed water, in

several ways as mentioned below briefly.

- ii) Cations may serve as a bond to hold the clay mineral particles together to limit the distance through which they can be separated, and
- ii) Some of the adsorbed cations may become hydrated and influence the overall nature of arrangement of water molecules and the thickness. For the purpose of illustration, radius of hydrated cations and bond energies are given in Table II.

On the basis of the work done by various research workers, it can be stated that the structure of water is altered considerably adjoining the charged surface. This altered structure may play an important role in the development of cohesion. However, there exists considerable gap in knowledge with respect to the cohesion developed and the nature of bonding present between water molecules in diffused adsorbed water zone from the surface of the clay mineral. In case of a real situation, in addition to physical adsorption around clay particles chemisorption effects may have to be considered (Mysels, 1967). Chemisorption may result into development of diagenetic bonds.

TABLE II
Bond Energies of Various Cations and Anions with Charged Lattice Sites

| Cation | Anion | Cation Hydration | Hydrated Cation Radius (A°) | Bond energy ergs/bond $\times 10^{-12}$ |
|-----------------|--------------|------------------|---|---|
| Li^+ | O^- | 7 | 3.7 | 4.6 |
| Na^+ | O^- | 5 | 3.3 | 4.8 |
| K^+ | O^- | 4 | 3.1 | 5.1 |
| NH_4^+ | O^- | 4 | 3.0 | 5.3 |
| Mg_2^+ | O^- | 12 | 4.4 | 8.0 |
| Ca_2^+ | O^- | 10 | 4.2 | 8.3 |
| Al_3^+ | O^- | 6 | 1.85 | 21.6 |

ROLE OF EXCHANGEABLE IONS IN STABILIZATION OF COHESIVE SOILS

In case of a saturated clay-water system a large number of charged surfaces of clay particles are arranged in different forms at various distances, depending upon the void ratio. The water molecules and ions inside the clay-water system are subjected to electro-magnetic force fields not only from the surface of one particle but also from the surfaces of adjoining particles. Such force fields cause complex arrangement of water molecules and bond strength between water molecules would be the resultant effect. In addition, because of the presence of other types of ions, the bonding between the water

molecules adjoining the clay particles may be not only due to the physical adsorption of water and ions but also due to the chemisorption.

In majority of cases, the cohesive soil system normally consists of clay particle possessing negative charge at the surface due to the isomorphic substitution. In such a situation the addition of different types of cations may bring about alteration in the clay-water system due to:

I) aggregation.

II) pozzolonic reaction, etc.

Hydroxides containing cations such as K^+ , Ca^{++} and Mg^{++} are found to bring about aggregation in the system and alter the texture of the soil itself. This is very well exhibited through change in the index properties making clayey soils almost friable ones (Katti, Kulkarni, and Radhakrishna, 1966).

When higher amount of hydroxides, which produce basic environment are incorporated in a clay-water system, the pH of the water may change to beyond 10 resulting in the liberation of Fe^{++} , SiO_2 , H_2O_3 , etc. from the soil particles. These released ions may promptly combine with the available cations like Ca^{++} and result into complex cementation agents. Especially in case of soil-lime stabilization, the cementing material has to come from the liberation of Al_2O_3 , SiO_2 , etc. from the particle itself.

Amount of the complex cementing agent produced is a function of the specific surface area. As this surface area goes on increasing the amount of release of SiO_2 , Al_2O_3 , etc. under appropriate conditions and lime could be more which may help in producing the cementing material.

The strength of a stabilized soil-lime mix depends upon number of contact points to be bound together. Lesser the number of contact points, lesser would be the amount of cementing material needed for stabilization. However, as the specific surface area goes on increasing the contact points also go on increasing thus increasing the amount of cementing material needed, for bonding. It is found that the development of cementing material and requirement of cementing material for bonding the particles are not in the same proportion and which depends upon type of clay. In view of this, it is observed that there exists an optimum particle size upto which the lime stabilization or any other stabilization with similar chemicals may be more effective.

If the grains are bigger than the optimum size, say gravel, then although the contact points may be less, the cementing material produced may not be adequate to give the desired strength.

The type of clay mineral also affects the amount of stabilizing agent to be incorporated

as part of it may be used up for balancing the charge deficiency.

The diffusion of cations in the soil-water system also affects clay stabilization. This particular aspect may be of immense use with respect to lime piles and also hardening of soft clayey soils with hydroxides which are soluble and partially soluble (Katti et al, 1967).

General examination of literature on stabilization with lime, bring about that the following factors which play an important role :-

- i) Specific surface area of the particles,
- ii) No. of contact points,
- iii) pH of the soil-water system,
- iv) amount of ions,
- v) Temperature, and
- vi) Presence of organic matter, etc.

It appears by taking into consideration the mechanical analysis of the soil system and also the reaction rates between soil particles and the basic environment it may be possible to develop a reasonable physical and mathematical model to evaluate the probable strength of a stabilized soil system using ions. This needs certain amount of discussion by panelists.

ELECTRO-OSMOTIC DRAINAGE AND ELECTRO-CHEMICAL STABILIZATION

The concept of electro-osmosis has been used as a major soil improvement technique for soft clays and fine grained soils, etc., wherein, the flow of water towards the electrodes would take place under the applied electrical potential and thus improve the strength of the soil system. The electro-osmosis is based on the following principle :- At the solid-liquid interface there is concentration of ions due to the formation of 'double layer'. Under the application of electrical field parallel to surface, the ions in the inner layer of the liquid phase move towards electrode of opposite sign and drag along the free water enclosed by this moving boundary film.

Casagrande has developed an equation for the velocity of flow based on Helmholtz theory as :

$$V = \frac{\epsilon \cdot \gamma \cdot E}{4 \cdot \pi \cdot \eta \cdot L}$$

Where,

ϵ = dielectric constant,

γ = zeta potential,

E = Electrical potential,

η = fluid viscosity and

L = distance between the electrodes.

During electro-osmosis there is strength gain consequent upon the volume decrease (Mitchell, 1968).

Mitchell (1968) has indicated that for soil water system containing electrolytes the application of electrical energy causes the following changes :-

ion exchange, ion diffusion, generation of osmotic and pH gradients, dessication from the heat generated at the electrodes, mineral decomposition, precipitation of secondary minerals, electrolysis, hydrolysis, oxidation, reduction, physical and chemical adsorption and fabric change.

Boiko has reported hardening of argillaceous soil by electro-chemical method (Pavate and Katti, 1975). It has been indicated for soil water system containing electrolytes that the application of electrical energy causes alteration in chemical environment to a large extent. (Cuisset, 1979).

In case of Bombay Marine clay also under the magnitude of voltages used in the electro-osmosis generated high current. The studies subsequently conducted using low voltage and controlling current, by Pavate and Katti (1975), has shown that the hardening has mainly taken place in the zone where pH is say around 9 and above. Under the applied electrical potential dissociation of Cl_2^- takes place resulting in high pH near cathode zone due to formation of NaOH and low pH near the anodic zone. Release of hydrogen takes place near cathode. This type of condition brings about a new environment in the marine clay-water system. The basic environment results into liberation of SiO_2 and Al_2O_3 , etc. that too in nescent state.

With the incorporation of chemicals such as CaCl_2 in the form of piles, trenches; it is possible to bring about hardening of marine clay from a strength as low as 0.05 kg/cm^2 to as high as 18 kg/cm^2 , which is attributed to formation of complex calcium-alumino-silicate etc.

It is also observed that, the dissociation period is 36 hours. Any application of electrical potential beyond this period has very little use. In view of that, the electrical potential need to be applied only to change the environment. After which, the normal chemical stabilization method can be employed to improve the strength of marine clays. The laboratory studies indicate that it is possible to harden a large area by controlled process of interchanging the electrodes and neutralizing the acidic zone.

It appears that electrochemical hardening specially for marine or salt saturated system may be of immense practical applicability. This type of method would bring about increasing insitu strength and also the reduction in the settlement characteristics without application of heavy pre-load.

The panelists may consider giving their views on future approach to this type of soil improvement technique. They may also give a guidance regarding direction in which the research should be taken up to make this process a practically viable one.

SATURATED CLAY-WATER SYSTEM

As mentioned earlier, in case of a saturated clay-water system a large number of charged surfaces of clay particles are arranged in different forms at various distances, depending upon the void ratio. In addition, because of the presence of other types of ions, the bonding between the water molecules adjoining the clay particles may be not only due to the physical adsorption of water and ions but also due to the chemisorption.

Studies conducted using bentonite at different void ratios have clearly shown that under fully saturated conditions increase in ucs and or vane shear strength is associated with decrease in void ratio. Refer Fig. 5. If benzene is used it is clearly observed that there is hardly any cohesion. This clearly indicated that reduction in void ratio has brought about considerable change in the bond strength of the absorbed water in the clay-water system.

Similar effects are observed by engineering and research workers in case of clayey soils. This has resulted into developing pre-consolidation concept and methods to improve the strength characteristics of the clay deposits, for engineering construction (Johnson, 1968; Mitchell, 1968).

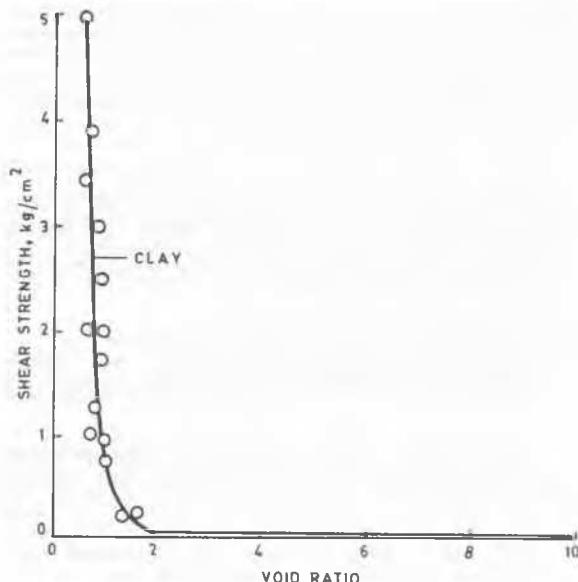


FIG.5. VARIATION OF SHEAR STRENGTH WITH VOID RATIO

In case of clayey deposits when the void ratio goes on decreasing the unconfined compressive strength increases significantly as shown in Fig. 5. These effects are more predominant when void ratio decreases below 0.7. The increase in ucs may be not only due to the physical adsorption but also due to chemisorption. Such effects are observed in case of over-consolidated clays and shales (Skempton, 1964). The high strength in over-consolidated clay and shales has been

attributed to the diagenetic bonds (Bjerrum, 1976).

From the above discussion it is clear that the changes on the surface of clay particles, in other words, the electrical environment influence significantly the bond strength between the water molecules in the clay water system under various void ratio conditions due to the phenomenon of adsorption. The cohesive force developed between the adjoining particles due to the electrical environment may be several times higher than the weight of the particles. The overall strength of a saturated clay-water element at a given depth is contributed both by the frictional characteristics of the soil particles and also by the magnitude of electrical environment existing in the clay-water system.

Thus, while dealing with the mechanics of clay-water system or of cohesive deposits it may be necessary to take into account, (i) weight of the clay particles, (ii) weight and properties of water and (iii) magnitude of electrical environment, etc.

STRENGTH DEVELOPMENT IN CLAY DEPOSITS CONTAINING MONTMORILLONITE TYPE OF CLAY MINERAL

The influence of the electrical environment on the strength and other aspects has been demonstrated in case of clayey soil deposits containing high amount of montmorillonite type of clay mineral, in India. In this work the following aspects are clearly brought about.

- I. The clayey soil deposit having swelling pressure of 3.0 to 5.0 kg/cm^2 is found to exhibit no volume change at a depth of 1.0 to 1.5 m of soil overburden which is equivalent to the pressure of 0.2 to 0.3 kg/cm^2 .
- II. The shear strength and cohesion increases rapidly with depth and remain constant as shown in Fig. 6 (Katti et al, 1979).
- III. The result based on the lateral pressure measurements have shown that the lateral pressure of the order of 3.0 to 5.0 kg/cm^2 is observed at depth of 1.0 to 1.5 m as shown in Fig. 6, although there exists cohesion of the order of 0.7 kg/cm^2 . With this cohesion as per the conventional equations of active pressure:
$$\sigma_a = \gamma h \tan^2 (45-\phi/2) - 2c \tan (45-\phi/2)$$

there should not have been any lateral pressure upto 5.0 m.

- IV. The studies with deadload surcharges have shown that even after the removal of surcharge considerable lateral pressure is retained and the cohesion is found to be considerably increased. Ref. Fig. 7.

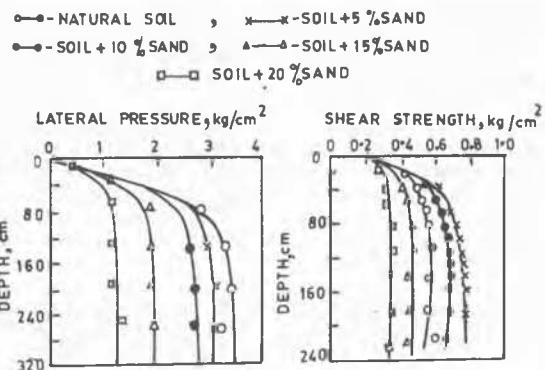


FIG.6. STUDIES ON BLACK COTTON SOIL FROM POONA

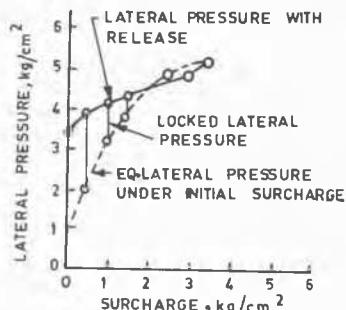


FIG.7. RESULTS OF LATERAL PRESSURE STUDIES WITH SURCHARGES

- V. A cohesive non-swelling soil of 1.0 to 1.5 m thickness having no swelling pressure but exhibiting cohesion is found to produce no volume change condition in black cotton soil below. This concept of CNS soil has been applied extensively in Malaprabha and Upper Krishna Project, Karnataka State, India, for construction of canals, dams cross drainage structures, etc. (Katti, 1978; Katti et al 1978).

The development of cohesion is found to be related to interaction of electrical charge on surface and the dipolar nature of water, as mentioned earlier. To understand the above effects and to see whether there exists a relation between the lateral pressure and forces balancing of it at shallow depths, the following physical and mathematical models have been put forth.

Micro Particle Model

The clay fraction in the expansive soil is idealized by assuming the particles to be consisting of size $2\mu \times 2\mu \times 2\mu$ with split as shown in Fig. 8, with a spring having expansion characteristics in C direction similar to that of montmorillonite type of clay mineral (Kulkarni and Katti, 1973). With the help of this model it is possible to explain equal magnitude of pressure in lateral and vertical directions, on the basis of the statistical distribution of the

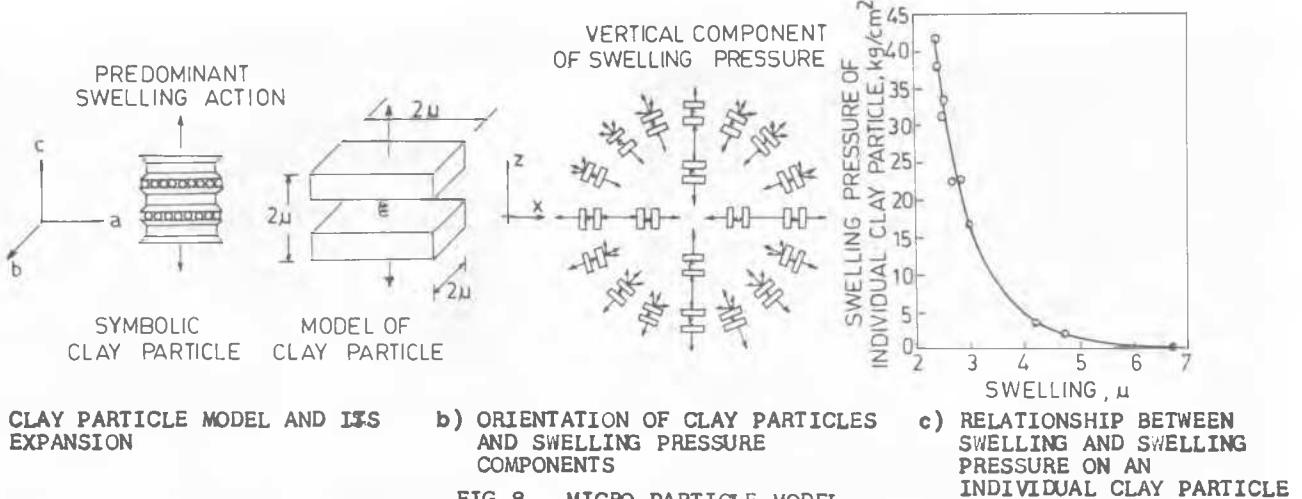


FIG. 8. MICRO-PARTICLE MODEL

randomly distributed particle as shown in Fig. 8.

By considering 1/3 particles distributed with their c axis oriented in each of the x, y and z axes, the apparent effect of the pressure becomes as though the pressure distribution is isotropic in nature. Further it is possible to relate the swelling pressure of the media to that of an individual particle using the equation.

$$q_{swi} = \frac{q_{sw} \times 1 \times 1}{\frac{1}{3} \left(\frac{P}{(2\mu)^3 (1+\epsilon_1)} \right)^{2/3} \times 2\mu \times 2\mu}$$

Wherein,

- q_{swi} = Swelling pressure of an equivalent individual particle, kg/cm^2
- P = Clay content %
- q_{sw} = measured swelling pressure, kg/cm^2 ; and
- ϵ_1 = initial void ratio

Swelling and swelling pressure of an individual particle can be correlated as shown in Fig. 8.

Cohesion Model

Work done with respect to the large scale test or in the field have shown that high magnitude of swelling pressure is balanced by the cohesion. To explain this, a cohesion model is proposed; based on similar lines to that of Kozeny's idealization for flow through pipes.

In this case swelling particles are moving with respect to the adjoining adsorbed water system which possesses varying degree of cohesion. The developed cohesion can be conceived as being able to bring about the resisting forces to counteract the swelling pressure. In case of clayey soils having clay

minerals with expanding lattice structure, the outward forces get developed due to the ingress of water into the interlayer and cohesive forces are developed around the clay particle due to formation of adsorbed water. Both these actions take place simultaneously. However, the adsorbed water film may get formed, around the particle at a faster rate than the ingress of water into the interlayer causing expansion. The cohesive forces thus developed may be effective in resisting the heave. In other words the resistance to heaving or expansion depends on how much and how fast the cohesive forces are developed around the particle.

In the media, however, incipient movement of certain clay particles with respect to the adjoining ones results in the mobilization of cohesion. Considering idealized particles as shown in Fig. 9, it can be shown that

$$Q_{sw} = 3.33 C_u \text{ for cylindrical particles,}$$

$$Q_{sw} = 5.0 C_u \text{ for cubical particles}$$

For general condition,

$$Q_{swv} = \alpha \cdot C_u$$

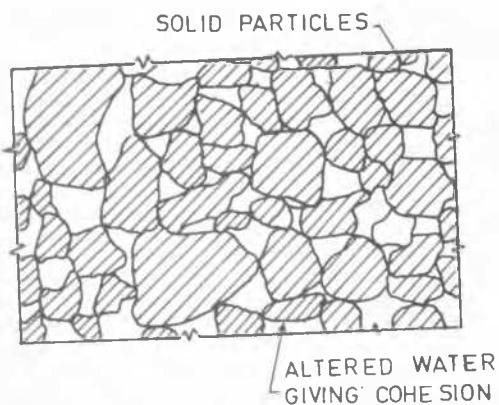
Wherein, α is shape factor (Joshi and Katti, 1980).

This model indicates that electrical environment rather than the gravitational force system alone controls the overall equilibrium.

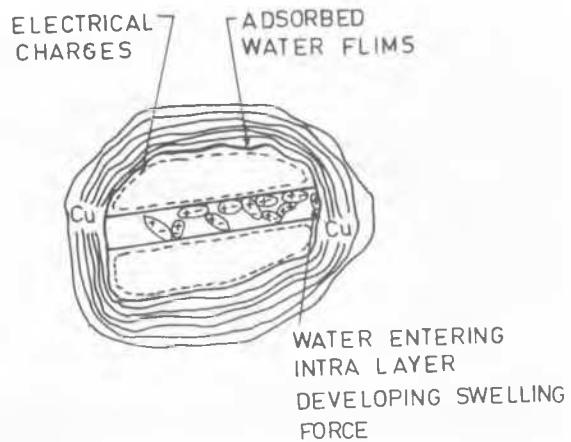
The above work shows that the drag forces, due to adsorbed water which is causing cohesion, are developed when a clay particle in soil displaces with respect to the continuous saturated adsorbed water.

SETTLEMENT CHARACTERISTICS OF CLAYEY DEPOSITS UNDER INFINITE FILL

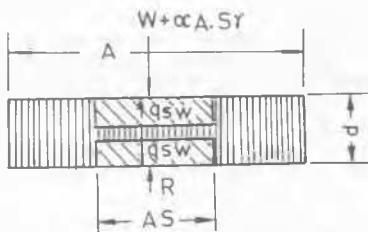
Many low lying areas in coastal and lake deposits are inundated during high tides on extreme rainy season. In large number of cases, 1.0 to 3.0 m of fill is normally adopted for reclamation. Many of these



a) SOIL PARTICLES EXPANDING IN RELATION TO ALTERED WATER SYSTEM
(Kozeny's reverse model)

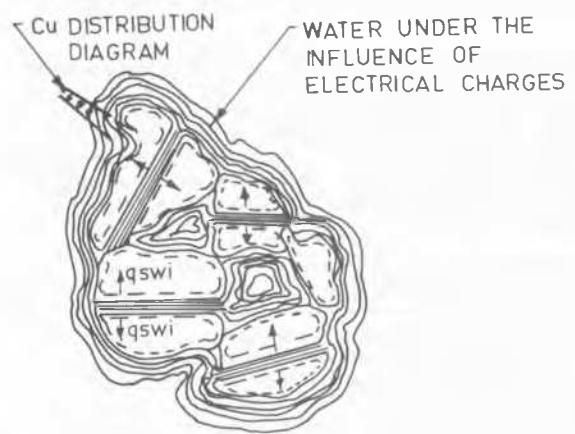


b) EXPANDING PARTICLE



$\alpha A.S\gamma$ = RESISTANCE FORCE
 $qswi$ = SWELLING PRESSURE DUE TO INDIVIDUAL PARTICLE
 qsw = SWELLING PRESSURE OF EQUIVALENT CUMULATIVE SOIL PARTICLE IN A VOLUME $A \times d$
 $P = As \cdot qswi = A \cdot qsw = \alpha A.S\gamma + W$

c) Forces on individual expanding particle



d) Expanding particle with adsorbed water layer resisting the expansive forces

FIG.9. CONCEPTUAL COHESION MODEL

deposits consist of a clayey soils alternating with silty clay, sand, etc. Sometimes very soft clays are underlying very stiff clays. An attempt is made to visualize the role played by the adsorbed water in respect of settlement characteristics of the clay-water system subjected to infinite fill loading. A saturated adsorbed water clay system may be picturised as clay particles connected to each other by spring at particulate level. Refer Fig. 10. When the solid particles try to move under a stress it can be expected that the cohesion due to the bonds in adsorbed water also need to be overcome. From this it can be stated that the stress transmitted to a clay and other soil particles at depths is not similar to that obtained by normal stress distribution theories. For this

purpose, it is necessary to take into consideration the stress transmitted to the adsorbed water also.

When the stress transmitted from a fill load is much less than the cohesive force due to adsorbed water bonds in the underlying clay-water system, the pore pressure may not develop. Secondly adsorbed water may produce drag on the soil particles and thus the stresses transmitted from grain to grain in the layer below, may not be of conventional type. In such a situation very little stress is transmitted to lower layer and the settlement may not be as per the calculation based on normal consolidation theory.

However, if the loads acting are higher than

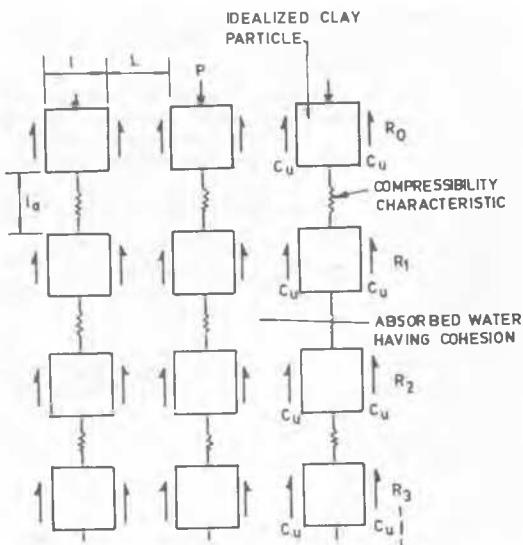


FIG.10. CONCEPTUAL SATURATED ADSORBED WATER-CLAY SYSTEM SUBJECTED TO FILL LOAD

the cohesive bonds, there is a possibility of breaking of adsorbed water bonds and changing into a different type of bond. Once the bonds break the material may appear as though it is conventional particulate system with development of pore pressure. In case, the cohesive forces are far higher than the load coming on them, the drag force has to be taken into account and the settlement equations of the form given below may be developed.

Total settlement

$$S_n = \alpha \cdot \sum_{R=1}^{n-1} \left[\frac{p - n\beta - p_0}{p - n\beta + p_0} \right]$$

Where, α = shape factor.

$$\beta = N_A 4 l^2 \cdot C_u$$

p = applied pressure

p_0 = initial pressure

C_u = drag force developed due to compression or consolidation

N_A = no. of clay particles per unit layer; and

l = width of clay particle.

The above mentioned effects are observed in 2 situations in India.

Case I

In case of Bombay marine clay having $C_u = 0.1$ to 0.15 kg/cm^2 a fill of even 1.0 to 2.0 m, which is equivalent to stress of 0.18 to 0.35 kg/cm^2 , produced 15 to 20 cm settlement almost as per the conventional theory. In this case the applied fill load is adequate to break the cohesive bonds. (Katti et al, 1974).

Case II

In Haldia similar soft clayey strata is overlain by 3 to 4 m of stiff clay, as shown in Fig. 11. When an infinite fill similar to that in Bombay Marine clay was placed on top of it, hardly any settlement was observed. In reality as per the normal method of analysis for settlement the ground should have settled considerably high, as in case of Bombay Marine clay because of the presence of very soft clay below stiff clay. This effect may be attributed to drag forces caused by the adsorbed water on the particles trying to cause displacement/settlement in the stiff layer. As the resisting forces due to cohesive bond are higher than fill loads, the type of settlement is different (Katti et al, 1976).

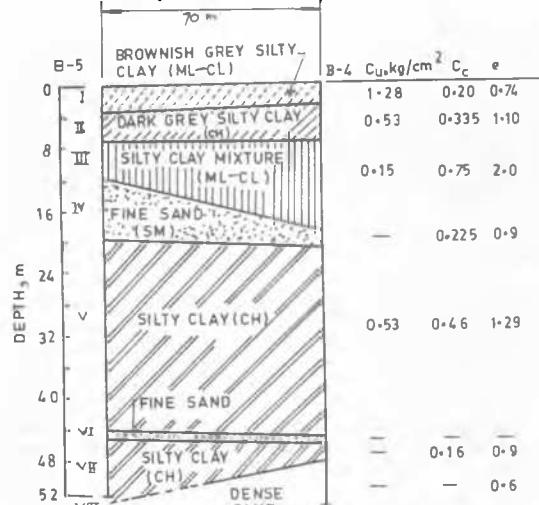


FIG.11. PROFILE ALONG BOREHOLES B-5 AND B-4 AT HALADIA.

In view of this it appears that the magnitude of cohesion in relation to nature of fill and its extent may have considerable effect on settlement aspects. This behaviour will affect considerably the improvement of soft clay underlain by stiff clay using preloading technique. This is very important in case of computation of negative skin friction. This aspect influences considerably the shear strength development in clayey soils.

It is appreciated that the panelists could enlighten on similar situation and suggest approach to be adopted for future research. This will have considerable bearing on improvement of clayey soils using fills and also by sand drains, especially when the upper crust is having very high cohesion.

From the above discussion it is clear that if we want to make scientific progress in the area of soil improvement of cohesive soils, it may be necessary to view the saturated clay-water system not merely two phase system consisting of soil particle and liquid water; but as a three phase system consisting of soil particles, water and weightless electrical environment which influences both clay and water system. Lack of measuring instruments to evaluate bond strength between water molecules and between water molecule and soil

particle has considerably hampered the development of mechanics governing improvement of saturated clay-water system.

The panelists may kindly consider discussing various points brought about in this paper. The discussion would certainly help research workers in the future work in the area of improvement of cohesive soils using ion and adsorbed water layer concept.

ACKNOWLEDGEMENT

Thanks are due to Central Board of Irrigation and Power, New Delhi for sponsoring work on Various Aspects of Expansive Soils which has led to study the behaviour of clayey soils in depth.

Thanks are also due to Dr. A.K. De and Prof. R.E. Bedford, Directors, I.I.T. Bombay for their encouragement. Help rendered for the preparation of the report of Dr. U.V. Kulkarni, Ex Research Officer and Shri E.S. Bhangale, Research Officer, C.B.I.P. Project is acknowledged. Assistance of staff members of Geotech. Engg. Section is highly appreciated.

REFERENCES

- Bauer, L.D. (1976). Introduction to colloid Chemistry. 1st Edition, pp.475, Interscience Publishers, New York.
- Beijerrum, L. (1976). Progressive failure in slopes of overconsolidated plastic clay and clay shales the third Terzaghi Lecture. Proc. ASCE, J.SM and FE Div., Vol.93, SM5, pp.1-50.
- Cuisset, O. (1979). Le potential electrocinétique des argiles influence de la salinité, Bull. Liaison Lab, P.et.ch., pp. 1520, Ref. 2407.
- Eisenberg, D. and Kauzmann, W. (1969). The structure and properties of water. 1st Edition, 296 pp., Oxford at the Clarendon Press.
- Grim, R.E. (1953). Clay Mineralogy 1st Edition, 384 pp., McGraw - Hill book Company, Inc. New York.
- Johnson, S.J. (1968). Foundation precompression with vertical sand drains. Speciality Conf. on placement and improvement of soil to support structures, ASCE, Soil Mech. and Found. Div. pp 9-39, Cambridge Massachusetts.
- Joshi, R.P. and Katti R.K. (1980). Lateral Pressure Development with Surcharges. Fourth Internation Conf. on expansive soil, Vol.1, pp. 227-241, Denver, Colorado.
- Katti, R.K. (1978). Search for Solutions to problems in Black Cotton Soils, First IGS Annual lecture, Indian Geotech Journal, Vol.9 No.1, pp. 1-80, New Delhi.
- Katti, R.K., Bhangale, E.S. and Kulkarni, U.V. (1978). Role of CNS layer in Malaprabha canal system passing through Black Cotton Soil deposits, Proc. 47th Res. session of Central Board of Irrigation and Power, Vol. 1,pp. 153-179, Hubli-Dharwar.
- Katti, R.K., et al. (1967). Stabilization of Bombay Marine clay with various Inorganic additives, Conf. 2nd South Asian on Soil Engineering, pp.589-599, Singapore.
- Katti, et al. (1974). Report on Geotechnical investigation of the ocean bed at the Bombay High region for Sagar Samrat, Submitted to Project Manager, Oil and Natural gas Commission, Bombay Offshore Project, pp.1-57, Bombay.
- Katti, R.K. et al. (1976). Report on subsurface exploration and general foundation conditions for phosphate-plant at Haldia, Hindustan Level Ltd., Backbay reclamation, pp. 1-162, Bombay.
- Katti, R.K.; Kulkarni, U.V.; Bhangale, E.S. and Divishikar, D.G. (1979). Studies on shear strength development in expansive black cotton soil media with and without cohesive non-swelling soil surcharge, Application to cuts and embankments in canal areas, Final Report to Central Board of Irrigation and Power, PP. 1-392, New Delhi.
- Kulkarni, S.K. and Katti, R.K. (1973). A Microparticle and Micro-anchor approach to Mechanics of swelling soil Media, Proc. Third Inter. Conf. on Expansive Soils, Vol. 1, pp. 43-52, Haifa, Israel.
- Lambe, T.W. (1960). A Mechanistic picture of shear strength in clay, Research Conference on shear strength of cohesive soils, pp. 555-580, Boulder, Colarado.
- Leonards, G.A. and Andersland O.B. (1960). The clay water system and the shearing resistance of clays, Research conference on shear strength of cohesive soils, pp. 793-818, Boulder, Colarado.
- Mitchell, J.K. (1968). In place treatment of Foundation soils, speciality Conf. on Placement and improvement of soil to support structures, ASCE, Soil Mech. and Found. Div., pp. 93-130, Cambridge, Massachusetts.
- Mysels, K.J. (1967). Introduction to Colloid Chemistry, 1st Edition, pp. 475, Interscience Publishers, New York.
- Pavate, T.V. and Katti R.K. (1975). Electro-chemical hardening of marine clay, Technical note, Geotechnical Engineering, Vol. 6, No. 1, Bangkok.
- Skempton, A.W. (1964). Long term stability of clay slopes, The Fourth Rankine lecture, Milestones in Soil Mechanics, Institute of Civil Engineers, pp. 81-108, London.
- Van Olphen, H. (1963). Clay Colloid Chemistry 1st Edition, pp. 1-301, Interscience Publishers, New York.

W.R. Mackechnie, Panelist

I would like to compliment Prof Katti for his leading paper particularly that portion which refers to chemical stabilization by ionic exchange. There is a very real need for the Geotechnical engineer to grasp the nettle of the fundamentals of clay mineralogy if any advance is to be made in this subject area. For at least three decades now we have relied on empirical approaches to chemical stabilization and this will not serve our purpose well. Recall your own experience gentlemen; they probably parallel mine. These are the typical areas; chemical stabilization largely with cement or hydrated lime has been found to be a modifying agent in making heavy soil of high plasticity index manageable, it has been used to justify the use of borderline materials for road construction because we find that in laboratory conditions we can persuade ourselves that a small optimum percentage of additive will bring the P.I. within specification or produce a C.B.R. or unconfined strength that meets an empirical standard.

If we have persisted we may have found that for instance in using a base exchange process to convert a calcium rich montmorillonite to a sodium rich one to serve as a bentonite that only particular ranges of temperature, pH and solution concentrations of specific chemicals will achieve this. At this stage of experience we are probably beginning to be able to formulate the questions we must put to the colloid chemists and nuclear physicists. However the majority of us will find our background in chemistry too sparse or we may simply be frightened by molecules and ions which are both too small to see and which complicate analysis by constant movement.

My plea therefore is for a greater educational input in clay mineralogy. An engineering graduate must be able to talk in terms of cation exchange capacity, sodium

absorption ratio, isomorphous substitution, and similar related terms and further be able to attach numerical values to these terms with the same facility with which he thinks of plasticity index activity and shear strength. Only then will we be able to place chemical stabilization in a category more advanced than cookery which I regret to say is still too often the situation today.

My other concern is for the application of chemical stabilization to practice, the micro studies of the laboratory are applied on a macro scale in the field. I remain convinced that too often we fool ourselves into believing that chemical stabilization has achieved the improvements which in reality are due to a bonus of say negative pore water pressure and not to ionic bonding at all and that we have in fact a very variable stabilised product with significant zones of defect in which the stabilizer is probably totally absent.

I worry also a great deal about the increase in strength with age attributed to stabilizers such as lime and cement. Does our testing take into account the potential but small reductions in moisture content for instance? In terms of strength improvement this will probably be confined to the lower end of the spectrum of shear strength but I believe the real advantage of the method will be found to lie in modifications to soils such as the reduction in the tendency to disperse by calcium substitution for sodium for instance.

Finally Mr Chairman it is heartening to see papers to this conference coauthored by engineers and soil chemists. I found it very encouraging also to see a paper presented in South Africa recently by a soil scientist, Prof Harmse which quite independently of Prof Katti's paper sets out for the civil engineer the chemical stabilization and clay mineralogy situation as it is understood today in almost identical terms. At last we appear ready to take the next step forward in this subject area.

T. Yamanouchi, Panelist

SOFT COHESIVE SOIL EMBANKMENT USING QUICKLIME AND FILTER FABRIC

Mr. Chairman, ladies and gentlemen. I would like to speak about a method of soft clay embankment using quicklime and filter fabric which has been contrived by myself (Yamanouchi, et al., 1967, 1971, 1976). Before doing that, I would like to give a comment on the present techniques of soil improvement. There have quite many been invented on that. However, the responses to the motive forces in soil improvement are still restricted within a part applications merely adopting a monotype relation among various ones as shown in Table 1 (Nozaki, 1965) which shows these relationships, and we are given some suggestions on the new approaches to soil improvement from the Table. The method which is going to be spoken can be said to have adopted a few kinds of forces multiply. But, this method is very simple in practice unlike the conventional admixing method.

The principle of my method may be understood by Fig. 1. Speaking about the multiple applications of mechanical, physical and chemical forces, the following actions are itemized in success in the embankment work.

(1) The consolidative dewatering owing to the additional consolidation pressure arises besides that owing to the ordinary effective overburden pressure, resulting from the expansion of quicklime at the early stage of the hydration of quicklime. This seems irrational, but the additional vertical pressure actually arises especially when quicklime is spread providing an enough space of non-quicklime. This was proved by a laboratory experiment as shown by Fig. 2.

(2) Adsorption of water from the soil due to the hydration of quicklime. In this stage, a flow due to osmosis might arise toward quicklime from the soil through the sheet making it a kind of semi-permeable membrane.

(3) A considerable rise of temperature in the soil causing by the heat discharge of quicklime during its hydration. This brings about a reduction of coefficient of permeability of the soil to make the drain effect easier particularly in cold season.

(4) Carbon dioxide including in the soil changes slaked lime to calcium carbonate after a certain period and the resulted compound provides a kind of reinforcement to the whole em-

Table 1 Responses resulted from mechanical, physical and chemical forces (Nozaki, 1965)

| Motive force \ Response | <u>Mechanical force</u> | Temperature | Light | Electricity | Magnetism | Chemistry |
|-------------------------|---|---|---|--|--|---|
| | F | T | HR | E | M | μ |
| Mechanical d | <u>Viscoelasticity, thixotropy, pore water pressure</u> | <u>Expansion, freezing, change of viscosity</u> | Light pressure, radioactive ray, hardening | Peak inverse-voltage, impulse current, solidification | Magnetostriction | <u>Seepage force, chemomechanical phenomenon, chemical rheology</u> |
| Thermal s | Adiabatic expansion (Joule-Thomson effect) | Thermal capacity, thermal conductivity | Radiation heat | Inverse piezoelectricity, Peltier effect, Joule's heat | Adiabatic demagnetization | <u>Reaction heat, freezing mixture</u> |
| Rayey n | Tripoly-luminescence | Black-body radiation, thermoluminescence | Razer, mazer, luminescence | Electric luminescence | Magnetic field luminescence, Faraday effect | Chemical luminescence |
| Electric D | Piezoelectric effect, tribo-electricity | Pyroelectricity, thermoelectricity, thermo-electro-motive force | Light battery, photoelectric effect | Polarization, electric capacity, conduction | MHD generation of electricity | Battery, ferromagnetic ferroelectrics |
| Magnetic B | Piezoelectric effect, magnetostriction effect | Thermal magnetic effect | Photomagnetic resonance | Electromagnetic induction | Magnetization effect | Alloy, ferrite, ferromagnetic body |
| Chemical C | <u>Seepage pressure, dewatering phenomenon, impact alteration</u> | Diffusion separation, phase alteration | Photographical effect, photo-polymerization | Electric analysis, electrophoresis, <u>electro-osmosis</u> | Magnetic-chemical reaction, descale, water treatment | Catalysis, bio-reaction |

bankment body, as seen by Fig. 3. As these results, a small vehicle could travel on the embankment in a past in-situ experiment.

Effects as above mentioned are indicated with underlines in Table 1.

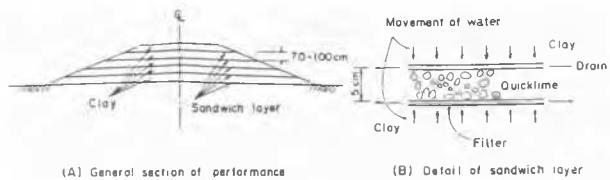


Fig. 1 Feature of the Embankment method

Well, I should introduce a case history in which the above method was satisfactorily successful recently in a big and high embankment with a soft cohesive soil. Specifications for the embankment work are shown by Fig. 4, and the formation level for the earthwork was decided to be the top of the crib retaining wall built at the part of slope toe. Each sandwich layer was made of granular quicklime of 5 cm thick being sandwiched between two sheets of polypropylene non-woven fabric belt, 30 cm wide and 0.3 cm thick, as shown by Fig. 5. Such sandwich layers laid with a space of 2.1 m between the centers in the horizontal direction and with a pitch of 1.8 m

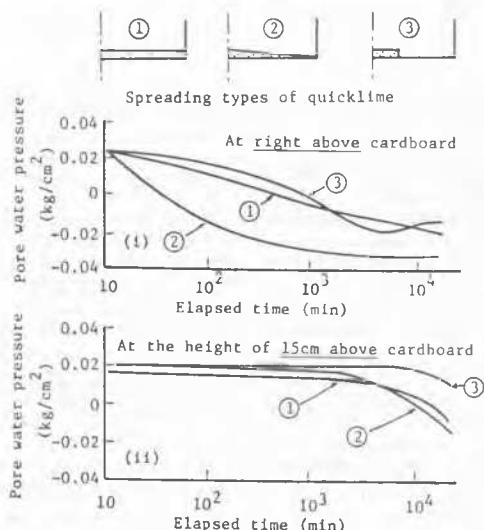


Fig. 2 Expansion effect of quicklime in laboratory

in the vertical direction, forming a triangle on the belt configuration in the cross section of the embankment. Consolidation calculations of the embankment were made after the manner of Barron (1948). Also, the safety factor for the slide failure was investigated in relation to the

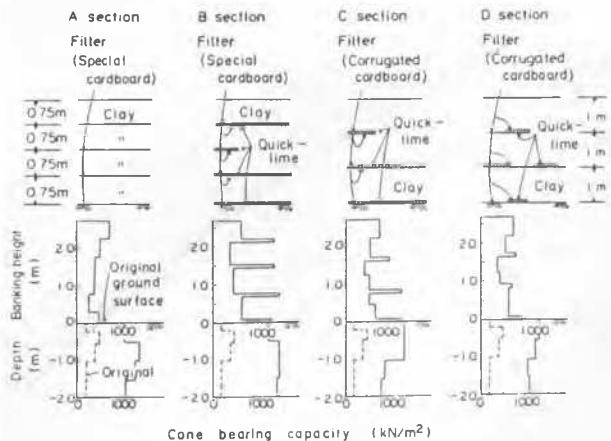


Fig. 3 Reinforcement of the embankment at a field experiment

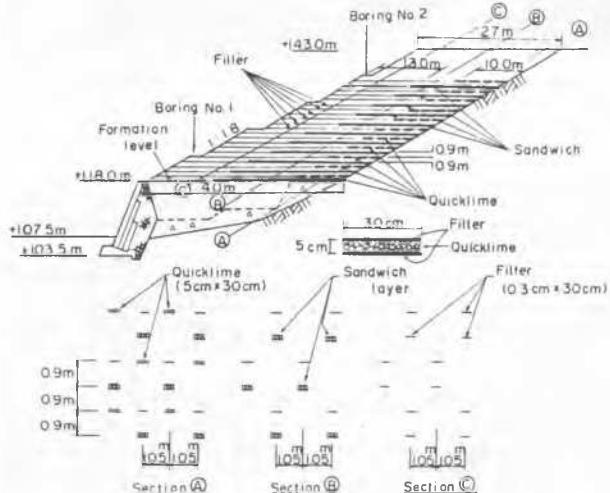


Fig. 4 Specifications of the high embankment

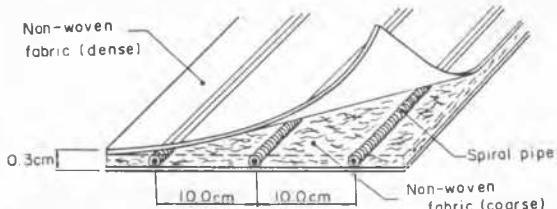


Fig. 5 Section of the filter sheet

embankment height and the embankment work speed, as shown by Figs. 6 and 7. A comparison of measured and calculated vertical pressures was made at a point on the bottom surface of the embankment, and the pressure ratio showed about 150% indicating a fact of arising the additional vertical pressure due to the three-dimensional expansion of quicklime accompanying about 50% increase over a term of 20 days, as seen by Fig. 8.

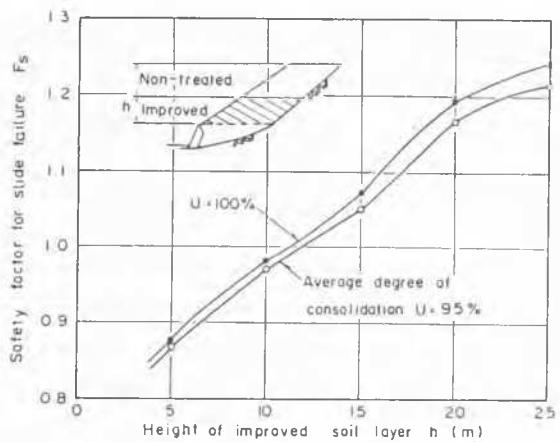


Fig. 6 Safety factor for slide failure on the embankment height

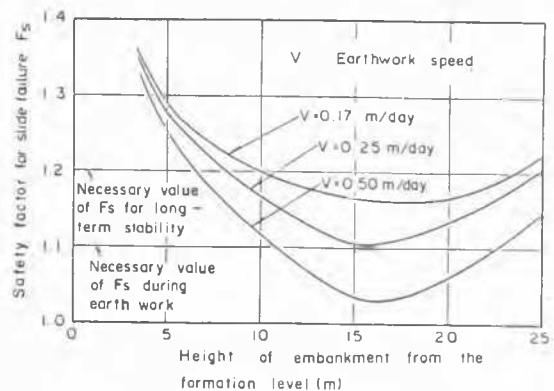


Fig. 7 Change of safety factor due to the earthwork speed

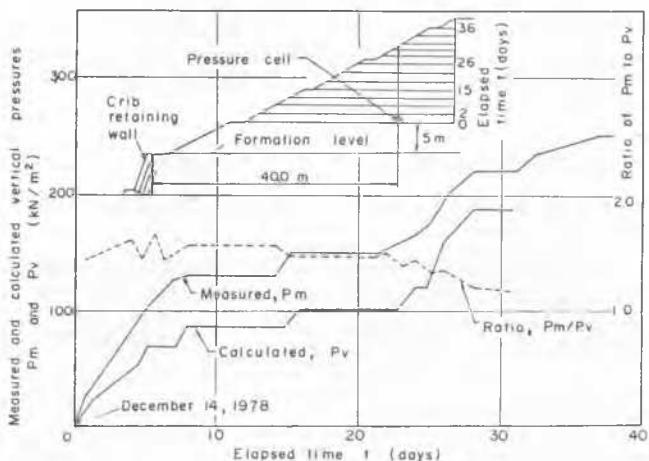


Fig. 8 Measured and calculated vertical pressures in the embankment

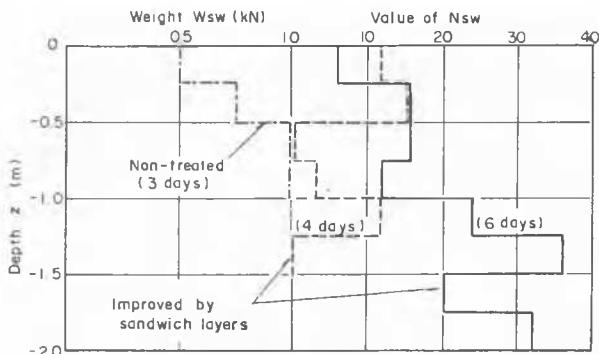


Fig. 9 Vertical strength of the improved embankment

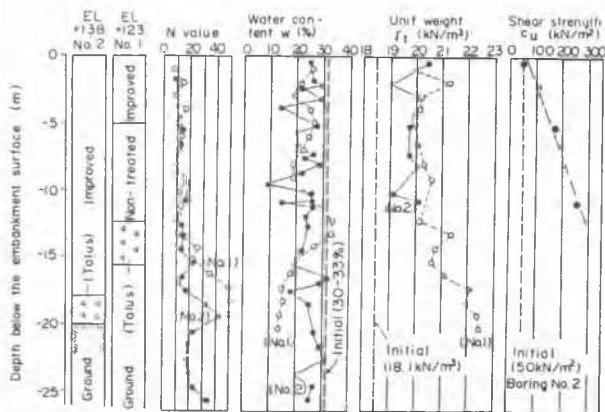


Fig. 10 Depthwise investigations of the embankment

The embankment soil was reasonably improved on various indexes, as shown by Figs. 9 and 10. Final figure, Fig. 11, shows an aerial view of the completed embankment which was achieved using the new method.

References

Barron, R. A. (1948). Consolidation of fine-grained soils by drain wells. Trans. ASCE, Vol. 113, pp. 718-754.

G.V. Rao and T. Ramamurthy (Written discussion)

"ALTERATION OF SOIL PARAMETERS BY STABILIZATION WITH LIME" (Paper by H. Brandl, Vol. 3, p. 583)

At the Indian Institute of Technology, Delhi, an extensive investigation is underway into the various aspects of the engineering behaviour of lime-fly ash stabilized soils. Two aspects of their behaviour are considered here.

Atterberg Limits:

Extensive literature review reveals that the addition of lime to soil or to soil-fly ash admixtures increases the plastic limit whereas the liquid limit increases in certain cases and decreases in other cases, giving an indication that mechanisms controlling these behaviours may be different from each other.



Fig. 11 An aerial view of the completed embankment

Nozaki, H. (1965). Mechanical and chemical forces in soil stabilization. Seminar textbook on Soil Chemistry, Japanese Soc. Electrochemistry. (in Japanese).

Yamanouchi, T. et al. (1967). Multiple-sandwich method of soft-clay banking using cardboard wicks and quicklime. Proc. 3rd Asian Reg. Conf. Soil Mech. and Found. Engrg., Haifa, Vol. 1, pp. 256-260.

Yamanouchi, T. et al. (1971). In-situ experiments on soft clay banking by means of multiple-sandwich method using cardboard wicks and quicklime. Proc. 4th Asian Reg. Conf. Soil Mech. and Found. Engrg., Bangkok, Vol. 1, pp. 342-345.

Yamanouchi, et al. (1976). Soft Clay banking using sandwich layers in situ made of wicked cardboard and quicklime. New Horizons in Construction Materials, ed. by H-Y Fang, Enviro Publishing Co., Inc., pp. 211-223.

Fig. 1 shows the immediate effect of addition of lime to a silt ($LL=27$, $PI=18$, Sand 33%, Silt 54%, Clay 13%) and a black cotton soil ($LL=56$, $PI=26$, Sand 14%, Silt 43%, Clay 38%). It is seen that the addition of lime (no curing) generally increases the liquid limit of silt whereas it decreases that of black cotton soil. This behaviour is very similar to that presented by the author for Soil II and I respectively. The addition of divalent Calcium has two effects: firstly the thickness of the diffuse double layer decreases because of divalency and this brings down the liquid limit; secondly attractive force is increased and hence flocculation occurs.

lation tends to take place, which leads to higher liquid limit. These two effects are opposite in trend. For a swelling soil like black cotton soil, the first effect dominates & for a non-swelling silt, the second effect is predominant.

If liquid limit is regarded as the water content at which sufficient free water is present to allow clay particles to slip one another under certain applied force and retain these new positions (Warkentin, 1960), then the liquid limit in all soils should be controlled by shearing resistance. But in the case of soils having expanding type of lattice minerals, the contribution due to diffuse double layer overrides and primarily governs the liquid limit. Thus soils containing expanding lattice type of minerals (illite and montmorillonite) are bound to behave in a different way compared to the other less active soils.

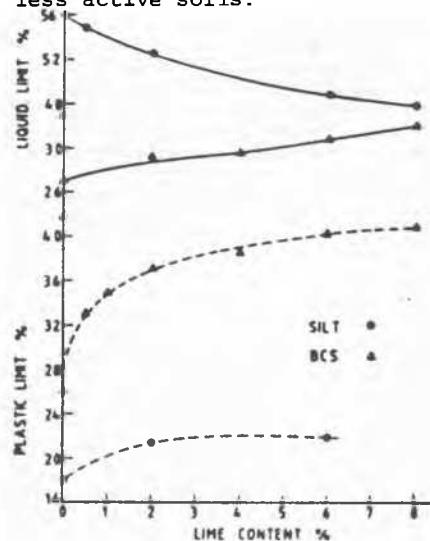


Fig. 1. Effect of addition of lime on plasticity limits

Fig. 1 further reveals that, the addition of lime decreases the plastic limit for both soils. This again is similar to the result obtained by the author and several other researchers. Two factors are responsible for controlling the plastic limit -- the shearing resistance at particle level and diffuse double layer.

M. Terashi and H. Tanaka (Written discussion)

ON THE PERMEABILITY OF CEMENT-AND LIME-TREATED SOILS

In the General Report of the Session 12, Mitchell and Katti stated that it was not clear whether lime columns act as sand drain, comparing the papers by Terashi and Tanaka vs Holm et al. To clarify the point of contradiction posed by the General Reporters, the writers of the discussion wish to show the ineffectiveness of the treated soils as drain material.

Permeability of Cement- and Lime-treated Soils

Permeability of treated soils by Deep Mixing Method has been studied by the constant head permeability tests (Terashi et al., 1980). Marine soil tested is Kawasaki Clay which is a

typical Japanese marine clay having the natural water content of 95%, liquid limit of 93%, plasticity index of 50 and clay content of 60%. Permeability of cement-treated soil is shown in Fig. 1 where ordinate shows the logarithm of the permeability, k (cm/s). Aw is the cement content respect to dry weight of soil. Wa is the water content of treated soil. Permeability of treated soil is known to be lowered with increasing Aw and decreasing Wa . As natural water content of the soil in-situ is much lower than those tested, permeability of the treated soil at site is lower than those in the figure. Therefore, it is far difficult to consider that treated soils act as effective drainage. The same tendency is also

Shear strength parameters:

The strength parameters in both total and effective stress conditions obtained through consolidated undrained triaxial compression testing, with back pressure, at low confining pressures, are shown in Fig. 2 for silt. It is clear that both ϕ and ϕ' increase with lime content, but are independent of curing period. The values of c as well as c' increase with lime content as well as curing period.

WARKENTIN, B.P. (1960), "Interpretation of the Upper Plastic Limit of Clays," Nature, Vol. 190, pp. 287-288.

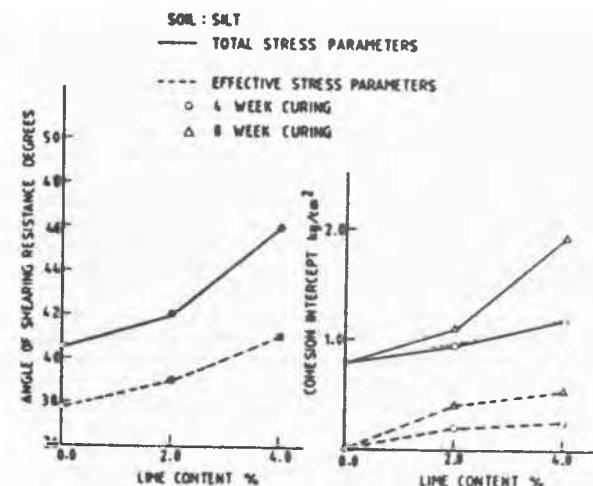


Fig. 2. Effect of addition of lime on strength parameters

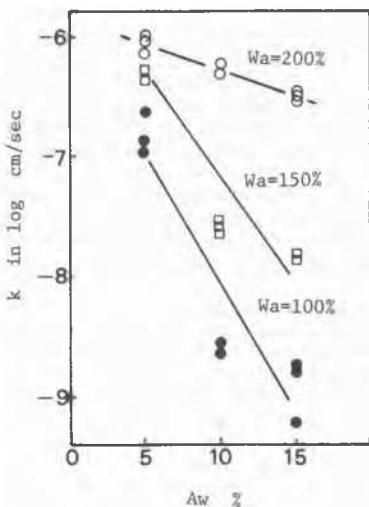


Fig. 1 Permeability of cement-treated soil found for lime-treated Japanese marine soils.

Consolidation of composite ground

A series of consolidation tests is executed on the model of improved ground by the columns of treated soil (composite ground). A part of the test results has been shown in the paper by Terashi and Tanaka (1981). Fig. 4 of the paper clearly shows the reduction of settlement by the improvement. Although the permeability of the treated soil is very low as shown in the preceding paragraph, it was known that the rate of consolidation was accelerated when the consolidation pressure, P_c was lower than the preconsolidation pressure, P_o of the composite ground. Fig. 2 shows the change of stress concentration ratio, $\Delta p/\bar{\Delta p}$ with elapsed time after loading. Where Δp is the stress increment carried either by treated soil or by untreated soil and $\bar{\Delta p}$ is the average stress increment applied to the composite ground. As shown in the figure where $P_c < P_o$, $\Delta p/\bar{\Delta p}$ is unity at the instant of loading, whereas applied stress gradually concentrates on the treated soil with time. This is due to the very low coefficient of volume compressibility of the treated soil. A key to the apparent acceleration of the consolidation lies in the fact that the treated soil behaves rather elastic under increasing load and untreated soil consolidate under decreasing load as long as $P_c < P_o$.

K. Kujala (Written discussion)

THE INFLUENCE OF ETTRINGITES IN THE STRENGTH OF GYPSUM-LIME STABILIZED CLAY (Comments on the topic "New Stabilizer Materials")

The influence of ettringites in the strengthening of humusrich clays is also remarkable when the stabilizing agent is a compound of lime and waste gypsum, the byproduct of the production of phosphoric acid. Here the mixture of gypsum and lime (gypsumlime) reacts with the aluminium oxide of clay (Al_2O_3). The reaction produces ettringites ($3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$) which can be recognized in the gypsumlime stabilized clay with

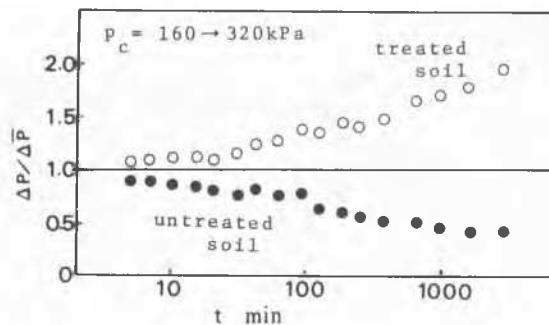


Fig. 2 Stress change with time

When $P_c > P_o$, $\Delta p/\bar{\Delta p}$ is always unity regardless of elapsed time and there is no acceleration of consolidation. These phenomena coincide with the knowledge on the consolidation characteristics of the treated soil itself. Okumura and Terashi (1975) stated that lime-treated soil showed high coefficient of consolidation and low coefficient of volume compressibility when consolidation pressure was lower than preconsolidation pressure of the treated soil. However, when the consolidation pressure exceeded the preconsolidation pressure, consolidation characteristics of treated soil were almost same as those of untreated soil.

Concluding remarks

Permeability of treated soil is lower than that of untreated soil and is lowered with increasing lime or cement content and with decreasing water content. Therefore treated soil is considered to be practically impermeable. Although the permeability of treated soil is very low, rate of consolidation under relatively small load is accelerated. This phenomenon is not so simple as to be judged by the analogy of sand drain. Sophisticated analysis must be further studied taking account of the stress concentration and consolidation characteristics of cement- and lime-treated soils.

References

- Okumura, T. and Terashi, M. (1975); Deep-Lime-Mixing Method of Stabilization for Marine Clays, Proc. 5th Asian Regional Conf. on SMFE
- Terashi et al. (1980); Permeability of Treated Soils, Proc. 15th Japan Conf. on SMFE
- Terashi, M. and Tanaka, H. (1981); Ground Improved by Deep Mixing Method, Proc. Xth ICSMFE

a scanning electron microscope (SEM) or an X-ray diffraction analysis.

The results of laboratory and field test (screw plate test) show 2...3 times greater strengths with gypsumlime than lime. In addition the strengthening is considerably faster, however, the stabilizing agent content is greater with gypsumlime.

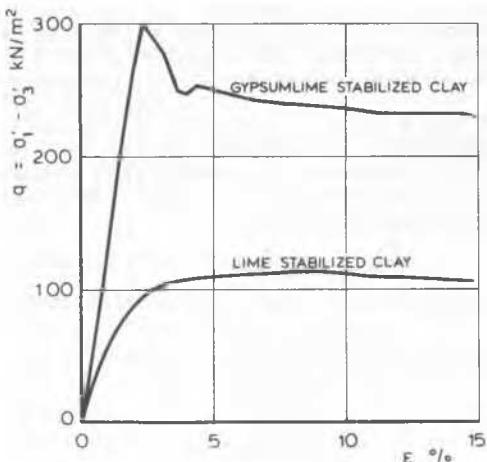


Figure 1. Stress-strain behaviour in gypsum-lime and lime stabilized clay.

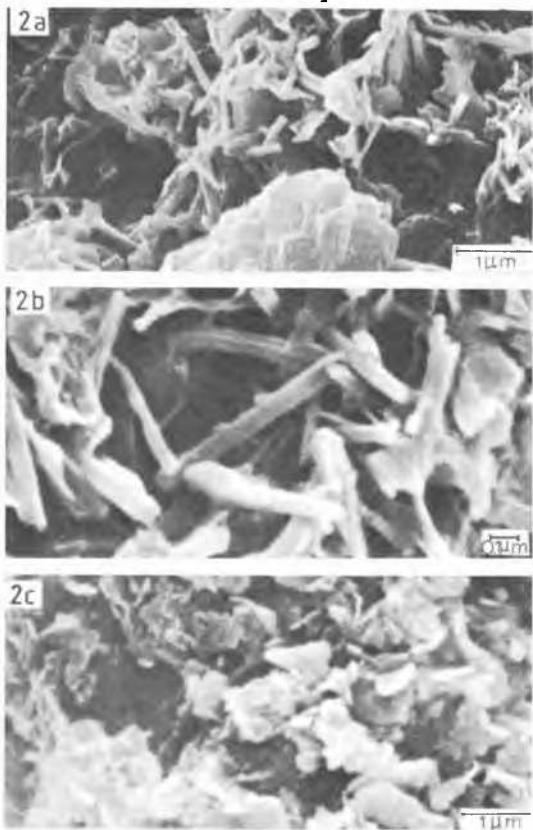


Figure 2. Scanning electron micrographs of stabilized clay. Stabilized with gypsumlime 2a and 2b, stabilized with lime 2c.

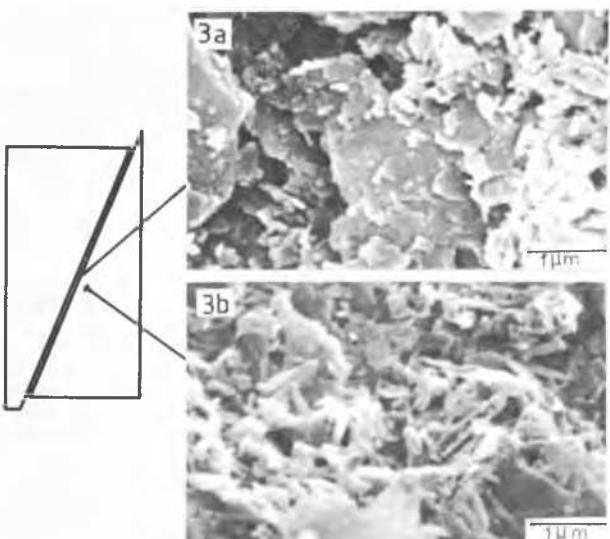


Figure 3. Scanning electron micrographs of gypsumlime stabilized clay. The failure surface 3a, 3b being a close up the failure surface.

The stress-strain behaviour of the stabilized clays differ according to stabilizing agent so that with gypsumlime the stress-strain proportion is almost linear up the peak value (Figure 1). Here the ettringite crystals break and the strength is somewhat lessened. In the lime stabilized clay no sudden changes occur in the stress-strain proportion.

Long, needlelike ettringites, with varied degrees of crystallization dominate the structure of the gypsumlime stabilized clay (Figure 2a and 2b). Compared with lime stabilized clay structural differences are great (Figure 2c).

When ettringites are formed they thrust strongly between particles of clay. Thus a dense network of ettringites is formed in the stabilized clay, which efficiently stops interparticular movements in the clay. This is the cause, among others, for the linear stress-strain proportion up to the peak value. Broken ettringite crystals and the parallel formation of clay particles can be seen in the failure surface of gypsumlime stabilized clay (Figure 3a). In the immediate neighbourhood of the failure surface the ettringite crystals are whole (Figure 3b).

In laboratory and field tests the mixture of gypsum and clay proved an effective stabilizing agent both for humusrich and humusless cohesive soils. Before a general adoption of gypsumlime in deep stabilization it must be clarified whether the ettringite compounds are permanent in different soil and loading conditions.

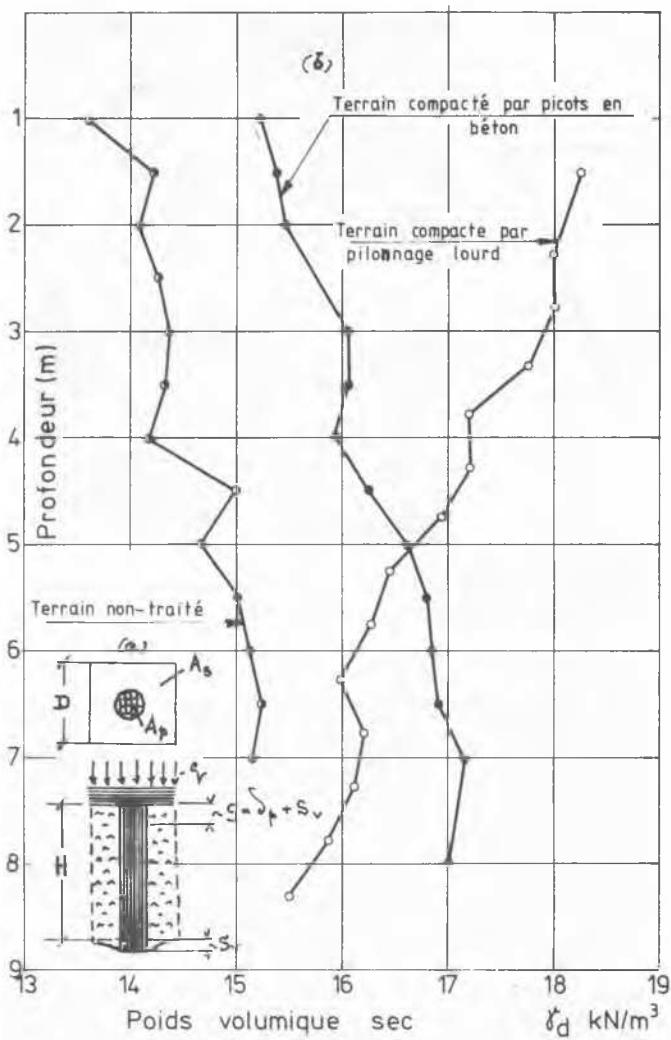
I.T.A. Stanculescu, Panelist

COMPACTAGE DU LOESS EN PROFONDEUR AVEC DES PICOTS

Le système, qui peut être considéré pour la traitement du loess, comme un procédé mixte de terre armée et de compactage en profondeur, nous a été suggéré par le résultat de quelques essais concernant le comportement des fonds -

tions sur pieux classiques, préfabriqués, introduits par battage.

Les picots sont coulés dans des trous cylindriques réalisés par mandrinage, en introduisant un tube métallique récupérable, par



vibration ou vibropercussion.

Le comportement des picots et leur sollicitation axiale peuvent être estimés en utilisant le calcul suivant, Fig. 1a.

$$\begin{aligned} A_p + A_s &= A \\ \frac{A_p}{A} = \alpha & \quad \frac{A_s}{A} = \beta \\ q = \alpha \cdot q_p + \beta \cdot q_s & \end{aligned} \quad (1)$$

Déformation du picot :

$$s_p = (q_p/E_p) H \quad (2)$$

Tassement de l'extrémité inférieure du picot :

$$s_p = \frac{\omega \cdot d_p \cdot \eta \cdot q_p}{E_{S.H.}^{*}} , \text{ avec ,}$$

$$E_{S.H.}^{*} = \frac{E_{S.H.}}{1 - \sqrt{2}} \quad (3)$$

Tenant compte de (1) :

$$q_s = (q - \alpha q_p) / \beta \quad (4)$$

Déplacement de la tête du picot :

$$s = \frac{q_p}{E_p} H + \frac{\omega \cdot d_p \cdot \eta \cdot q_p}{E_{S.H.}^{*}} \quad (5)$$

Tassement de la formation compressible de hauteur H, en tenant compte de (4) :

$$s = \frac{q - \alpha q_p}{\beta \cdot E_s} H \quad (6)$$

Le déplacement (5) étant égal au tassement (4) on obtient finalement :

$$q_p = \frac{(1/\beta) H}{\frac{\alpha}{\beta} + \frac{s}{E_p} + \frac{s}{E_s} \omega \eta d_p} q \quad (7)$$

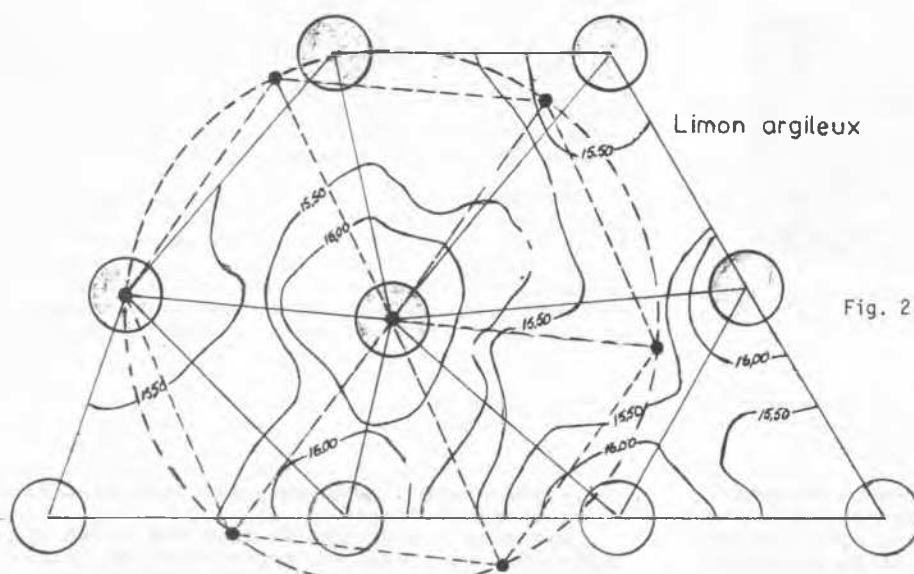


Fig. 2. Distorsion du maillage des picots; tracé des lignes d'égale valeur γ_d - kN/m³

Le degré de compactage pour obtenir l'insensibilité à l'humidité dépend de la densité du maillage. L'effet du compactage se ressent uniformément sur toute la profondeur du terrain traité (Fig.1b).

Des essais en vraie grandeur ont permis de fixer la distance nécessaire pour réduire le tassement supplémentaire par humidification, à un taux admissible. On a vérifié la distribu-

tion du poids sec du loess compacté obtenu en profondeur, pour diverses distorsions du maillage des picots (Fig.2).

Le terrain homogène comme porosité devient non homogène et pour les surfaces de fondation de petite dimension, les distorsions du maillage peuvent conduire à des tassements différentielles et à des rotations inadmissibles.

A.. Van Wambeke (Oral discussion)

After the informative addresses of the session president, Mr. CAMBEFORT, general reporter Mr. MITCHELL and co-reporter, Mr. KATTI, I would like to add a few personal observations.

Soil improvement could be compared to various Forms of medical therapy, such as :

- Surgery
- Allopathy
- Homeopathy

In particular, compaction by heavy tamping (dynamic consolidation) and using explosives may be catalogued under homeopathic methods even though at first sight, it appears a Ludicrous description for a system of ramming the soil with 20 tonne weights from a height of 30 m or using several kilograms of explosive at a depth of 20 m.

However, nothing is added to the soil (as opposed to grouting) and no other material is substituted in place of the soil (as opposed to sand columns and piles). The only modification is an improvement of the internal structural characteristics in itself and by itself through physical or physico-chemical process.

Finally in the case of clayey soils, the most controversial and interesting form of improvement expected by Louis Menard, it is in fact a brutal mechanical stressing required to modify the natural electrochemical bonds and restructure the soil.

The depth of influence of heavy tamping is a function of the impact energy according to a law of stress absorption with depth, a second degree parabolic curve.

But of course, other factors may have an influence which is difficult to quantify, either favourable or unfavourable.

- the presence of air in the soil which is a function of the soil type (very high for compressible organic soils and partially saturated fine grained soils but questionable in the case of clayey soils).

- the parabolic curve for pressure absorption with depth is certainly optimistic for unsaturated and heterogeneous soils (refuse depots, rubble) the presence of non negligible air pockets or small caverns reduce the wave

propagation considerably.

- for saturated soils a certain percentage of the impact energy is transmitted by the liquid phase and acts on the soil structure without absorption which greatly improves the efficiency of the method.

Finally there is one point which has been insufficiently stressed to date. The condition of constant stress to a certain depth for efficient compaction is in practice not the only criterion to be adhered to. The forces are acting in a material where the natural stresses increase with depth and thus the surcharge pressures produced by the impact must be greater than the proportional stresses at depth.

From this we find that the efficient depth limit for the compacted soil is inversely proportional to the cube of the depth and not the square.

These factors intervene in a complex pattern and it appears to be impossible to give a theoretical formula linking the impact energy with the efficient depth. In assuming the second degree parabola we may use:

$$z_{\text{lim}} = k \sqrt{W \times H}$$

k being less than one and decreasing with increasing heterogeneity and clay content or increasing with moisture content.

REFERENCES :

- MENARD, L. and BROISE , Y. (1975). Theoretical and practical aspects of dynamic consolidation. Geotechnique, March.
- BOURDON, G. (1979). La consolidation dynamique et le problème de l'évacuation de l'eau. Sols - Soils, nos 30-31.
- MENARD, L. (1972). La consolidation dynamique des remblais récents et sols compressibles. Application aux ouvrages maritimes. TRAVAUX, Novembre.
- DE BEER, E. VAN WAMBEKE, A. (1973). Consolidation dynamique par pilonnage intensif - aire d'essai d'Embourg. Annales des Travaux Publics de Belgique, n°5, Octobre.

J.A. Charles (Oral discussion)

DYNAMIC CONSOLIDATION

During the eight years that have elapsed since the Menard technique of "dynamic consolidation" was introduced into the United Kingdom, this method of ground improvement is known to have been applied at some 25 sites in Britain. It has been used almost exclusively to compact loose fills (domestic, industrial and mining wastes, demolition rubble and opencast mining backfills) and in only one known instance has it been used to treat a soft alluvial soil. Much of the theoretical background that is usually advanced to explain the action of dynamic consolidation, relating to such matters as the compressibility of saturated soils due to the presence of micro-bubbles, liquefaction, hydraulic fracture, thixotropic recovery etc (Menard, 1972; Menard and Broise, 1975; Schlosser and Juran, 1979) is irrelevant when the technique is used to compact loose fills. Indeed, it diverts attention from the two major uncertainties associated with the use of the method on fills, namely the depth to which the method effectively compacts the fill and the magnitude of movements that will occur in the fill subsequent to dynamic consolidation, respectively.

(a) Depth of effectiveness

It is necessary to have some definition of depth of effectiveness. In investigations carried out by the Building Research Station settlements produced during dynamic consolidation have been measured at different depths within the fill and the depth of effectiveness has been defined as the depth to which significant settlement, and consequently significant increase in density, has occurred.

Menard and Broise (1975) stated that

$$z < \sqrt{W.H.}$$

where z is the depth of effectiveness in metres when a weight W tonnes has been dropped from H metres.

Investigation of dynamic consolidation on a clay fill suggested that

$$z = 0.35 \sqrt{W.H.} \text{ (Charles et al, 1978)}$$

S. Varaksin (Oral discussion)

AMELIORATION DES SOLS PAR COMBINAISON DE LA CONSOLIDATION DYNAMIQUE ET DES DRAINS VERTICAUX

Je voudrais illustrer deux commentaires faits par le rapporteur général. Je cite : " Il n'y a pas de solution à tous les problèmes et l'expérience précède toujours la théorie."

La capacité de prévision du comportement des sols traités par les diverses techniques d'amélioration de sols est à la base de l'exemple pratique cité.

En effet, peu de techniques mécaniques isolées permettent de résoudre un problème géotechnique complexe.

and on an old domestic refuse fill

$$z = 0.4 \sqrt{W.H.} \text{ (Charles, 1979).}$$

At these sites there was a total energy input of 2800 k.N.m/m² and 2600 kN.m/m² respectively. It would seem that with the typical weights (15 tonnes) and heights of fall (20 metres) used in the United Kingdom, loose fills are likely to be effectively compacted to a 5 or 6 metre depth.

(b) Movements in the fill subsequent to treatment

Charles et al (1981) have presented data from several sites showing that significant settlements can occur subsequent to dynamic consolidation of loose fills.

References

CHARLES, J A (1979): Field observations of a trial of dynamic consolidation on an old refuse tip in the east end of London. Proc Symp on Engineering Behaviour of Industrial and Urban Fill, Birmingham, pp E1-E13, Midland Geotech Soc.

CHARLES, J A, EARLE, E W and BURFORD, D (1978): Treatment and subsequent performance of cohesive fill left by opencast ironstone mining at Snatchill experimental housing site, Corby. Proc Conf on Clay Fills, London, pp 63-72, Instn of Civ Engrs.

CHARLES, J A, BURFORD, D and WATTS, K S (1981): Field studies of the effectiveness of "dynamic consolidation". Proc 10th Int Conf Soil Mech & Fndn Engg, Stockholm, vol 3, pp 617-622.

MENARD, L (1972): La consolidation dynamique des remblais récents et sols compressibles. Travaux, Nov, pp 56-60.

MENARD, L and BROISE, Y (1975): Theoretical and practical aspects of dynamic consolidation. Geotechnique, vol 25, no 1, pp 3-18.

SCHLOSSER, F and JURAN, I (1979): Design parameters for artificially improved soils. General report. Proc 7th European Conf on Soil Mech & Fndn Engg, Brighton, vol 5, pp 197-225.

La construction d'un hall d'usine en République Fédérale d'Allemagne posait les problèmes de fondation suivants :

- des descentes de charges importantes au droit des appuis
- la fondation d'un dallage avec des charges réparties importantes,
- les déformations sous l'influence de remblais d'épaisseur variable pour amener la plate-forme à sa cote de fondation.

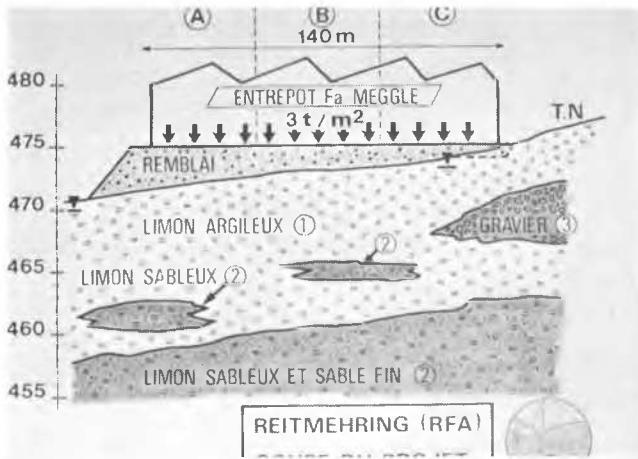


Fig. 1 Coupe du projet de construction de l'entrepôt REITMEHRING / R.F.A.

Trois techniques d'amélioration de sol se complétant mutuellement ont été utilisées :

- A - La consolidation dynamique
- B - Les drains verticaux cylindriques, à grand débit et présentant une résistance élevée aux chocs,
- C - la méthode traditionnelle d'excavation et remplacement.

et surtout l'interaction de ces techniques.

A - La consolidation dynamique

La Consolidation dynamique permettait de résoudre les problèmes de portance et déformation dans les remblais et les problèmes de déformation des sables limoneux sous-jacents, si un exutoire était créé pour évacuer rapidement l'eau expulsée vers les argiles imperméables.

B - Les drains verticaux

Les drains permettent d'accélérer la consolidation sous le poids des remblais, et évacuer rapidement l'eau du terrain mis en surpression par la consolidation dynamique.

C - L'excavation et remplacement

L'excavation et le remplacement **sous les semelles** assurent la portance à un taux élevé dans les zones de faible épaisseur de remblais.

La photo n° 2 illustre les courbes granulométriques des diverses couches :

- 1 - 3 zones traitées par la consolidation dynamique

Analyses

Granulométriques

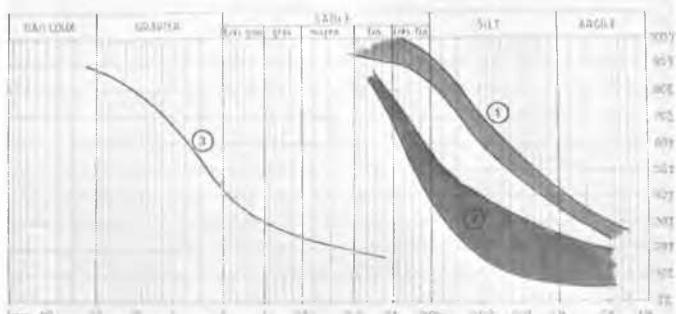


Fig. 2 Courbes granulométriques

- 2 terrains où les propriétés de drainage doivent être modifiées pour permettre une consolidation statique et dynamique rapide.

La figure 3 représente une vue générale de la mise en place des drains et le remblaiement de la plate-forme.



Fig. 3 Mise en place des drains

Voici l'aspect d'un drain cylindrique après "le jeu de massacre" (Fig. 4)

La courbe de tassement et de chargement (fig. 5) illustre que des drains cylindriques installés à des mailles supérieures à 3 mètres et l'utilisation de la consolidation dynamique, maintenant une pression interstitielle élevée, permettent d'atteindre 100 % de consolidation primaire dans des délais extrêmement courts.

La courbe de Tassement précédente a montré que les problèmes de tassement dans les argiles étaient résolus et ce profil pressiométrique montre les caractéristiques de cisaillement et de déformation dans les couches supérieures et inférieures, après consolidation dynamique.



Fig. 4 Aspect du drain

CONCLUSION

Le choix d'une succession judicieuse de plusieurs techniques d'amélioration de sol, le choix de techniques de contrôle appropriées et l'expérience du concepteur permettent

H.Q. Golder (Oral discussion)

ON GROUTING

INTRODUCTION

Cement grouting is probably the oldest process used to improve the mechanical properties of soils.

Grouting means 'to fill the voids of a granular material with a substance which increases weight and reduces permeability, and cements the soil grains together as the grout sets'.

Thirty years ago it was common to try to grout sands with cement grout. Today everyone knows that it doesn't work. The particles of cement are silt size, i.e. less than 0.06 mm in diameter. These are too large to enter the voids of a medium sand or any finer material.

Cement grout cannot permeate a medium sand. This is a negative property. But something happens if you try to grout a sand with cement. A 'pear-shaped' body of neat cement is formed around the grout-pipe. The size of the 'pear-drop' depends on the grouting pressure used, and the original degree of compaction of the sand. In a loose sand the volume of the pear-drop can be quite large. Using a 5 cm diameter pipe perforated over the bottom 60 cm the bottom of the drop can have a diameter of about 30 cm and the top 10 cm.

This 'positive' aspect of the above negative property means that although you cannot 'permeate' a loose sand with a cement grout, the grout can push the sand away from the pipe if injected under pressure and so it will compact the sand. This is a positive property of great value, which was successfully employed

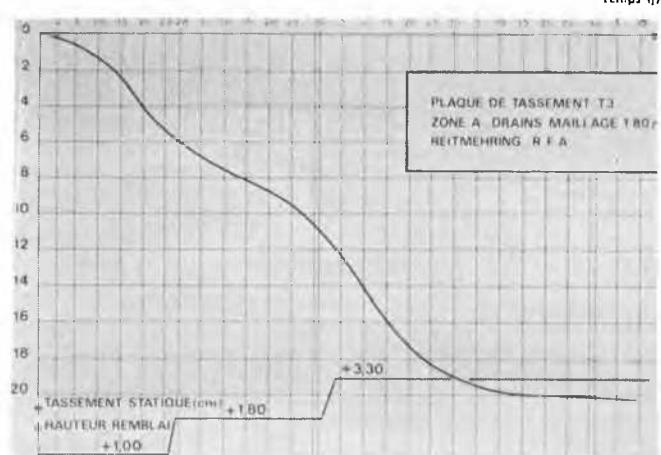


Fig. 5 Courbe de tassement

d'élargir le domaine d'application de l'amélioration des sols et d'éliminer le risque, que représentait l'utilisation d'une seule technique pour résoudre des conditions de sol hétérogènes et souvent imprévues.

in the case described below.

DESCRIPTION OF GROUTING IN KARSTIC LIMESTONE

A Nuclear Power Station was to be built in an area of Karstic Limestone.

Soil Conditions

The site was flat and low-lying. A surface fill of sand and gravel had been placed for access, but all foundations were to be in a soft limestone known as 'Biopel' which occurred close to the surface. The Biopel contained vertical joints and where these intersected each other solution cavities had been formed. The cavities were filled with fine sand, which in some places was in a very loose condition. Many 'N' values were below 10, and in some places the boring rods dropped under their own weight.

At a depth of some 20 m Dolomite, a much harder Limestone, was encountered.

Problem

A reactor building 45 m in diameter and with an applied foundation pressure of 380 kPa was in course of construction when an area of badly decomposed Biopel was discovered under part of the site.

Borings showed the area to be about 400 sq m of which a 'lozenge' 25 m long by 6 m wide was under the rim of the reactor foundation.

Of the possible solutions grouting was chosen.

Grouting

The grouting programme consisted of the following steps:-

1. Gravity grouting with cement grout to fill the large voids - primary holes at 2.5 m centres.
2. Secondary and tertiary holes at 1.2 m centres. Low pressure cement grouting at 140 kPa to compact the loose sand. The grout did NOT penetrate the sand.
3. Silicate grouting of the compacted sand to give strength to the sand. This was considered necessary in case of further solution of limestone in the future. This was done from a fresh series of holes, again in three stages.

Records

Records during grouting are most important. The 'take' of each hole at each stage should be recorded and marked up on a wall chart immediately - NOT left in some engineer's field book. Colour charts can be very useful here, using green for 'safe' areas, and red for 'dangerous'. But make sure that the engineers

C.H. Beyrer (Oral discussion)

CONTROLE DES RESULTATS DE L'AMELIORATION EN UTILISANT LA METHODE PRESSIONOMETRIQUE

Je voudrais parler du contrôle des résultats de l'amélioration de sols en utilisant la méthode pressiométrique pour la prédiction du degré d'amélioration.

Le système de contrôle de qualité par essais dans tous les domaines technologiques se passe en trois étapes :

- essai préalable (développement de la solution)
- contrôle spécifique (d'identité) de qualité des échantillons
- contrôle global de la qualité de la solution.

En géotechnique, ce système est limité normalement aux travaux de terrassement (construction de routes ou de barrages) ou d'améliorations relativement superficielles. Dans ces cas, on utilise les méthodes conventionnelles d'essais de contrôle, qui sont encore suffisantes et économiques. Mais ces méthodes deviennent inexactes dans leur valeur de prédiction et peu économiques s'il s'agit de l'amélioration de sols de mauvaise qualité en profondeur, soit pour obtenir une force portante nécessaire avec des tassements limités ou même pour améliorer la stabilité des pentes.

Si on peut encore utiliser des mesures continues de tassement et de pression interstitielle pour le contrôle de travaux de consolidation, aussi bien que des sondages simples - dynamiques ou statiques - pour démontrer les résultats obtenus, ces méthodes sont insuffisantes s'il s'agit de stabilisation en particulier par injections chimiques, surtout en raison de la

concerned are not colour blind (a handicap which is more common than usually realized).

Checking

The success of the treatment was assessed by regarding the treated area as a new site to be examined from scratch by all the usual methods.

These were:

- a) Boring and undisturbed sampling with 'N' values.
- b) Core-drilling.
- c) Continuous soundings.

We considered that if the 'N' values were in general greater than 30 the site was satisfactory. Similar criteria were laid down for the properties measured by the other methods.

The treated area was considered to be satisfactory and construction went ahead.

Settlement

Very careful measurements of settlement were taken over the period of construction. The final settlement was less than 1 cm and the tilt was 2 mm.

profondeur et de l'hétérogénéité du sol.

Ainsi au cours de la construction d'une autoroute traversant les Alpes Autrichiennes, le tracé demandait des tranchées et remblais importants dans les flancs d'une vallée étroite.

Les pentes sont constituées de phyllite-noire, assez peu compacte et mouillée de 20 à 30 mètres d'épaisseur avec un facteur de sécurité pour la stabilité égal à 1.

Grâce aux techniques modernes de terrassement, déblais et remblais sont exécutés très vite et déjà pendant la construction, on observait des glissements superficiels et une accélération du fluage de la masse totale de la pente. En plus, il était aussi très difficile de juger de la perméabilité et des différents passages d'eau qui changeaient continuellement à cause de la teneur en limon, résultant de la décomposition des phyllites.

Une étude exacte de méthodes efficaces mais économiques à utiliser pour la stabilisation, de ce glissement potentiel était nécessaire.

On a choisi une nouvelle méthode d'injection par infiltration chimique sans pression avec des lances perdues. Par cristallisation chimique, la liaison entre grains est améliorée.

Pour pouvoir contrôler l'amélioration, on a établi un programme d'essais de laboratoire et sur chantier, permettant une connaissance des facteurs les plus importants :

- hétérogénéité du matériau de glissement superficiel,
- possibilité de stabilisation des masses instables profondes,
- changement probable du bilan hydrologique dans les pentes de la vallée,
- changement de la déformation par cisaillement.

Le contrôle global a été précédé d'une part par des essais de cisaillement au laboratoire en boite assez grande ($50 \times 50 \text{ cm}$) dans le matériau non amélioré (mise en boite en compacité naturelle) et d'autre part "in situ" (surface de cisaillement = $90 \times 90 \text{ cm}$) dans le matériau amélioré par injection sur place (figs 1 et 2).

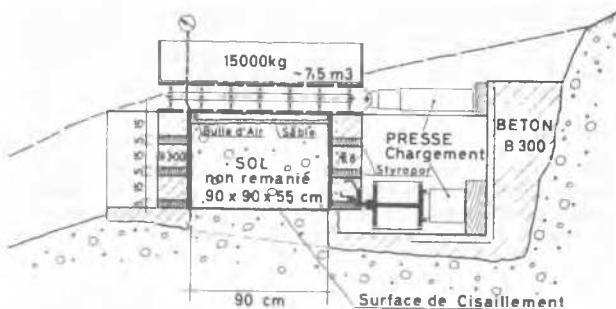


Fig. 1 Essais de cisaillement IN SITU

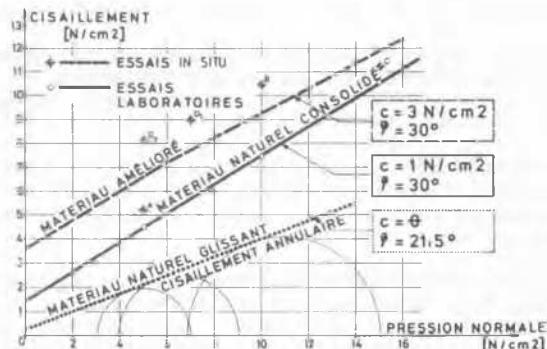


Fig. 2 Résultats des essais de cisaillement.

On a ainsi pu définir la possibilité d'augmentation des valeurs de cisaillement dans ce matériau.

On a utilisé ensuite des essais pressiométriques Ménard pour des zones d'essais à grande échelle sur une partie de la pente à stabiliser. Un grand nombre d'essais (tous les 1,5 - 2,0 m) jusqu'à 20 m de profondeur totale avant et après injection dans des délais de 4, 6 et 20 semaines) ont été exécutés et permettaient d'étalonner la méthode en fonction de la résistance au cisaillement.

Toutes les valeurs habituelles E_M , $\frac{p}{E_M}$, E_p , p_p étaient interprétées statistiquement (valeur moyenne et écart type).

Une relation empirique (fig. 3), basée sur la pression limite, était établie pour l'amélioration réalisable (domaine de rupture en cas de dépassement de la résistance au cisaillement).

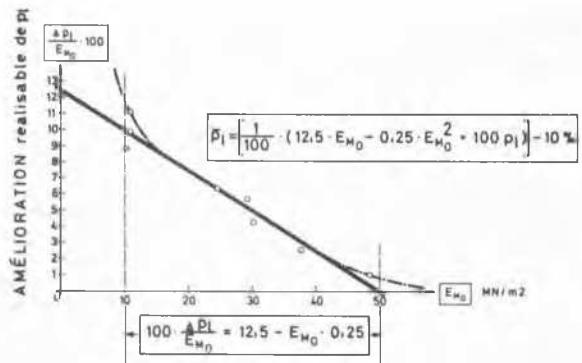


Fig. 3 RELATION : E_{M0} / AMÉLIORATION PI / E_{M0}

Cette relation était donc fixée comme base de comparaison pour le contrôle d'exécution des injections. (fig. 4)

AMÉLIORATION RÉALISABLE ET AMÉLIORATION RÉALISÉ

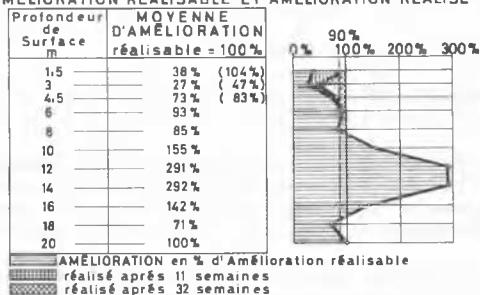


Fig. 4 Comparaison avec les résultats obtenus.

En plus on a établi des courbes d'amélioration pour 4 différentes zones de profondeur dépendant du temps (fig. 5)

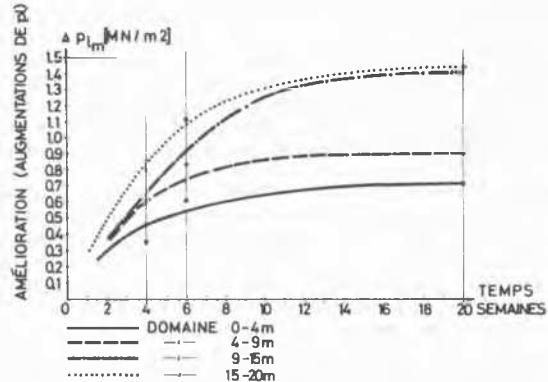


Fig. 5 Amélioration en fonction du temps.

De même manière, on a contrôlé les améliorations possibles par injection de fissures dans les flancs rocheux d'un barrage en béton. Ici la définition se basait sur les variations des mo-

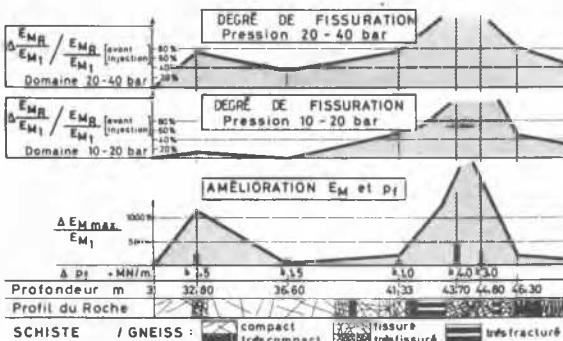


Fig. 6 Amélioration du rocher.

A. Partos (Oral discussion)

CONTROL OF COMPACTION GROUTING BY NUCLEAR DEPTH DENSITY TESTS

Essais de Densité en Profondeur pour Vérifier la Densification de Sol

SYNOPSIS Soil improvement by compaction grouting is monitored by nucleonic depth density and depth moisture gauges. Dry unit weights, void ratios and relative densities are computed from field and laboratory data.

Table 1 - Void Ratios (e) at Test Locations

| Depth Below Floor m | Test Location No. | | | | | |
|---------------------|-------------------|-------|-----------------|-------|-------|-------|
| | 2 | | 5 | | 10 | |
| | e_o | e_f | e_o | e_f | e_o | e_f |
| 2.7 | 1.00 | .75 | .92 | .69 | 1.00 | .92 |
| 3.5 | 1.04 | .85 | 1.04 | .96 | .89 | .80 |
| 4.1 | 1.00 | .75 | .88 | .82 | 1.00 | .88 |
| 4.7 | .96 | .69 | 1.00 | .85 | 1.10 | .97 |
| Average 0-4.7 | $e_{oa} = 1.00$ | | $e_{fa} = 0.82$ | | | |

e_o = initial; e_f = after compaction grouting

INTRODUCTION

Chemical grouting of loose to very loose sands could provide required stiffness for proposed forging equipment foundations (1) but sand particles could be separated through hydraulic fracturing and the costs of chemical grouting are high. Therefore compaction grouting was also employed to reduce void ratio of loose sand.

MONITORING

Nine m long and 47.3 mm I.D. aluminum pipes were installed at 12 locations in an area of 12 by 27 m in size. Troxler model 1351 density and model 1251 moisture gauges and a model 2651 Scaler-Ratemeter (2) were used. Each gauge consists of a probe, a fitted cable and a shield and a standard. The density probe contains a 3 millicurie source of Radium-226 and measures the total unit weight by backscatter and absorption of nuclear radiation of a spherically shaped volume approximately 13 cm in radius. The moisture probe contains a 3 millicurie source of Radium-Beryllium and detects neutrons which are thermalized by hydrogen. Several hundreds of readings were

obtained before and during and after completion of compaction grouting.

Typical void ratios at 3 test locations and project average void ratios are shown in Table 1. Compaction grouting increased an

Table 2 - Relative Densities (D_r)

| Laboratory | Void Ratio (e) | | D_r (%) | | | |
|------------|--------------------|-----------|-----------|----------|----------|----------|
| | e_{max} | e_{min} | e_{oa} | e_{fa} | D_{ri} | D_{rf} |
| | 1.27 | 0.64 | 1.00 | 0.64 | 40 | 71 |

D_{ri} = initial; D_{rf} = after compaction grouting

Table 3 - Volumetric Stability of Forge Foundation Soils

| Phase | Initial | After Grouting | |
|------------|---------|----------------|----------|
| | | Compaction | Chemical |
| Solids (%) | 50 | 55 | 78% |
| Voids (%) | 50 | 45 | 22% |
| Void Ratio | 1.00 | 0.82 | |

average relative density from 40% to 71% (Table 2). The volume of solids and voids was 78% and 22%, respectively, after completion of compaction and chemical grouting (Table 3).

CONCLUSIONS

Nucleonic depth gauges provide repetitive testing of the subsoil to monitor at the same location an increase of relative density due to successful compaction grouting operations.

REFERENCES

- Woods, R.D. and Partos, A. (1981) "Control of Soil Improvement by Crosshole Testing". Proceedings X. ICSMFE Vol. 3 pp. 793-796.
- Partos, A. and Koerner, R.M. (1975) "Monitoring Consolidation Settlements of Soft Soils Induced by Dewatering" ASTM, STP 584, pp. 111-126.

USE OF GEOTEXTILES IN SOIL IMPROVEMENT IN JAPAN

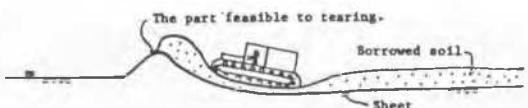
In Japan, soil improvement techniques are in most cases necessary to adopt in earthworks of alluvial soft clays, volcanic ash cohesive soils and peaty soils which are distributed there very commonly. Main problems are concerned with (a) the placement of transported good earth by some thickness on the original soft ground as the first stage of construction, (b) the deep improvement of the soft ground itself, and (c) the building of high embankment using such difficult soils. Of these works, the first item work is often called "the primary stage ground improvement" and the secondary one "the secondary stage ground improvement." The first stage ground improvement is more important in Japan than in other many countries.

The polyvinyl or polyester sheet has for the first time begun to use for the primary ground improvement in the end of 1960s (Fukuzumi et al., 1970). Following that, a much more resisting material, the polyethylene net (mesh) of high density, has been applying to use in the same means as the above sheet method since the first of 1970s (Yamanouchi, 1970) as shown by Fig. 1 (b). This resinous net was proved to be useful also for the separation of granular material when the material is spread on the soft clay ground (Yamanouchi, 1967). Both materials, the sheet and the resinous net, have then become to use together with a rope net laying on the upper side of the material in order to reinforce against the rupturing failure feasible to arise due to a too large curvature which is inevitable at the front part of a working bulldozer as seen in Fig. 1 (a). Such modified methods are called "the roped-sheet method" and "the roped-net method."

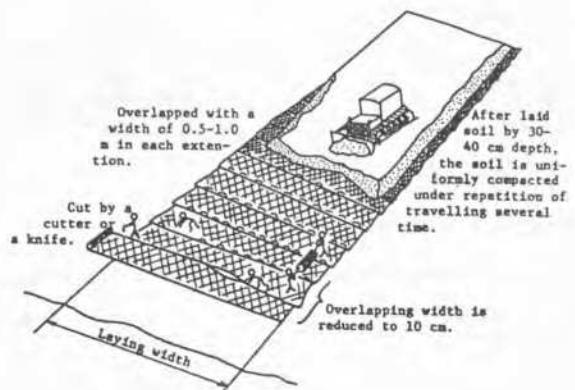
A steel net (mesh) has once been tested in an experimental road of low embankment by Japanese Road Public Corporation (JS SMFE, 1969). But, the steel net has never been used since then for a worry about a possible trouble in the future subsurface earthwork. The bamboo net fabricated at the field was for the first time applied at a soft clay ground by a company in 1960s, but the material has not been many used owing to be out of Japanese soil engineers' taste.

Such various methods are summarized in relation to the water content of alluvial soft clay in reclaimed onshore land in Fig. 2 (Kobori, 1977). Where, the highest water content is learned to be around 300% that was overcome by means of the roped-net method. When the bulldozer is used in the spread work of earth on the soft clay ground, it is not easy to build in a form of uniform thickness for a remarkable flow movement of the ground clay arisen beneath the working bulldozer. For solving this problem, a company (Shimizu et al., 1977) has contrived a method of spreading sand using a special pump as shown by Fig. 3.

Analytical studies have also been published on the sheet and resinous net methods (Shimizu et al., 1977, Yamanouchi et al., 1979). However, the results of these studies seem to be ambiguous in anyway because the rigorous settlement



(a) Usual state at the front part of a bulldozer in sheet method



(b) General view of the performance in resinous net method (after H. Koyama)

Fig. 1 Placement of borrowed earth on soft ground using sheet and resinous net

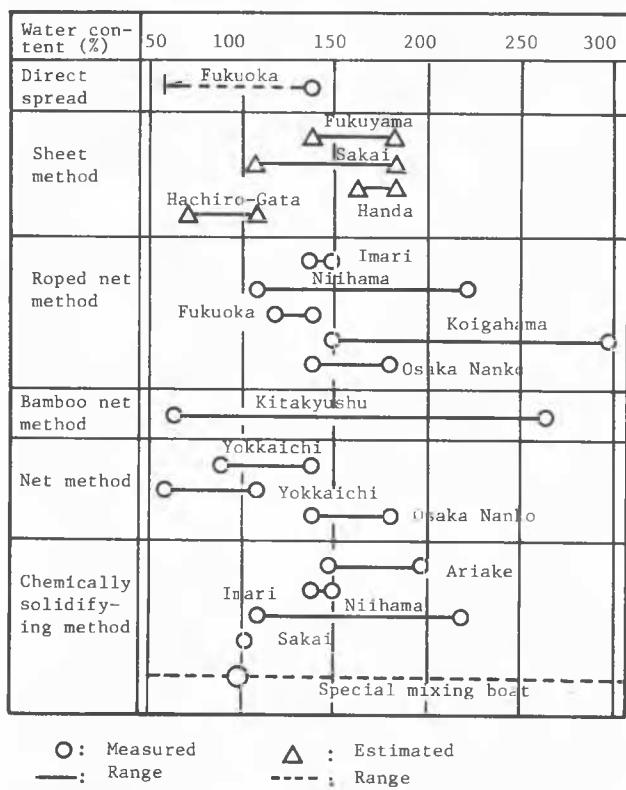


Fig. 2 Experiences of the placement of earth on alluvial soft clay grounds using geotextiles (Kobori, 1977)

cannot be defined for a continuous settlement due to consolidation as well as for the matter that the bearing capacity can be mobilized after causing a large curvature of the textile as far as the sheet or net is used on the soft clay ground.

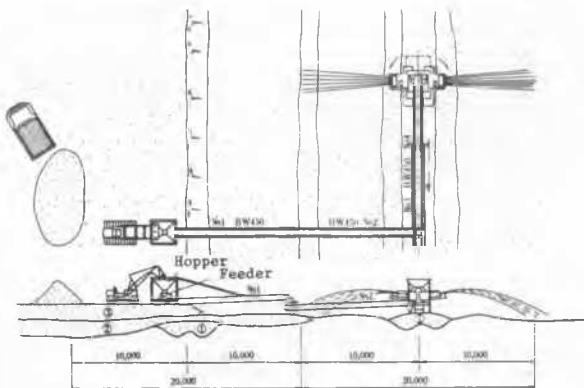


Fig. 3 A double jet-conveyer (Shimizu et al., 1977)

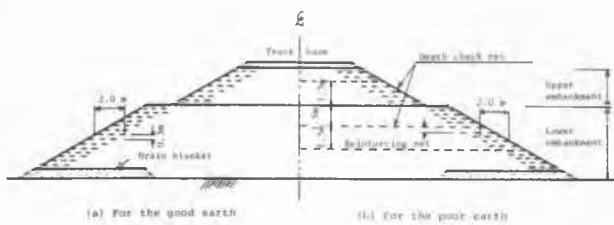


Fig. 4 Embankment standard by Japanese National Railway (Uezawa, et al., 1975)

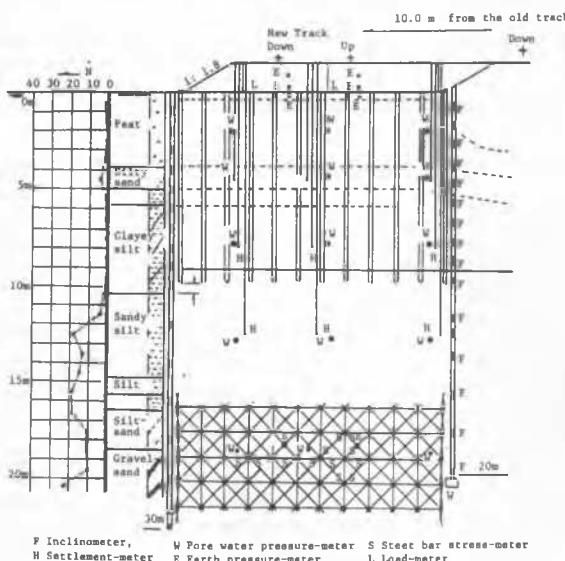


Fig. 5 Method using tied pile groups and steel net (Shima, 1981)

Another recent practice is a use of resinous net as an identifying material for the earthwork control in embankment which has been standardized by Japanese National Railway Corporation (Uezawa et al., 1975), as shown by Fig. 4. It is very few in Japan to adopt geotextiles for reinforcing the embankment itself as the foregoing work item (c) except for this example because of the cost problem.

Previously said geotextiles are used also in the vertical drain in Japan, too. Several kinds of filter fabric of non-woven have been made comparison involving the Swedish drain material, but any authoritative report has never been published for preventing a trouble of unfavorable competition.

Very recently, a unique ground improvement technique was carried out in a peaty ground for a railway ground construction (Shima, 1981). After installing wooden pile groups, all tops of the piles were connected with steel bars and then a steel net was laid on these before placing the embankment earth, as shown by Fig. 5. This method seems similar to a Russian idea found in a book written by Egorov (1962) as shown by Fig. 6. Also, I remember a field test carried out in 1960s by a company who connected the tops of ropes installed into a soft clay ground for the vertical drain instead of cardboard wicks. But, this method was stopped to develop more.

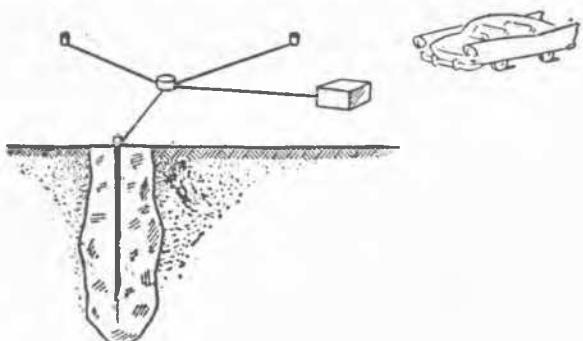


Fig. 6 Tied pile method (Egorov, 1962)

There are many new inventions and contrivances in the use of geotextiles in soil improvement and each method is elected according to the choice of the engineer who is engaged in the respective project. However, I think we should pay attention also to a comparison from the viewpoint of value engineering (VE) considering the utilization of local and natural materials. The use of geotextiles should not be a fashion.

References

- Egorov, A. L. (1962), Translated into Japanese by Toyota, H. (1963). Strength and forms. Tokyo-Tosho Co., p. 112.
- Fukuzumi, R. and Nishibayashi, K. (1970). A soft clay ground treatment using polyvinyl sheet. Proc. 25th Annual Conf., Japanese Soc. Civil Engr., Vol. 3, pp. 122-1-4. (in Japanese).
- Japanese Soc. Soil Mech. and Found. Engrg. (1969). Report by the investigation Committee on Low Embankment (Part 1). (in Japanese).

- Kobori, M. (1977). Discussions on the earth placement on the reclaimed soft ground. Annual Reports of the Res. Inst. of Penta-Ocean Construction Co., 1976, pp. 84-88. (in Japanese).
- Shima, T. (1981). An executive example of pile-net method in a railway embankment. Shallow Soil Stabilization. Sogodoboku-Kenkyujo, pp. 97-104. (in Japanese).
- Shimizu, A. et al. (1977). Basic principle and an executive example using Trical net. Ditto, pp. 51-63. (in Japanese).
- Uezawa, H. and Komine, T. (1975). Reinforcement of embankment using net. Tetsudo-Doboku, Vol. 17, No. 5, pp. 21-24. (in Japanese).
- Yamanouchi, T. (1967). Structural effect of restraint layer on subgrade of low bearing capacity in flexible pavement. Proc. 2nd Int. Conf. Structural Design of Asphalt Pavements, Ann Arbor, pp. 381-389.
- Yamanouchi, T. (1970). Experimental study on the improvement of the bearing capacity by laying a resinous net. Proc. Symp. Foundations Interbedded Sands, Perth, pp. 102-108.
- Yamanouchi, T. et al. (1979). Calculation of settlement of resinous mesh in its application to earth works by the use of slab theory. Technology Reports, Kyushu University, Vol. 52, No. 4. pp. 433-440. (in Japanese).

O.G. Ingles, Panelist

GEOTECHNICAL FABRICS FOR SOIL ENGINEERING

The last ten years have seen a remarkable growth in the use of synthetic fabrics in soil engineering. It is not, however, the applications that are new, so much as the materials. Manufacturers today produce cheaply geotextiles with a wide range of properties suitable to almost any specific need of the soil engineer. Never before has the engineer had such material options at his command, and he is only now beginning to learn how to exercise those options wisely.

Fabrics received a ready acceptance because they conformed in application to certain long-known (hence "self-evident") principles for soil stabilisation: particularly the 'sandbag' concept, whereby a weak material is strengthened by wrapping it in some thin but stronger natural or artificial membrane - a concept at least 200 years old.

The challenge to the soil engineer today is to know how to choose his geotextile material in such a way as to maximise its useful properties, and construction economy, whilst minimising subsequent in-service risk. The necessary guidelines for this selection are not yet well known or perhaps even established. I propose therefore to offer some comments drawn from personal experience which may provide a helpful starting point when assessing the practicability of incorporating geotextiles in design soil structures.

Literature has accumulated rapidly since 1975 (cf. Lewis, 1978). There are now many published papers and some specialty conference proceedings; recently, the first book has appeared ("Construction and Geotechnical Engineering Using Synthetic Fabrics", Koerner & Welsh, Wiley, 1980) - which even so does not cover much of the relevant literature then available.

The first principles in choosing a fabric are surely

- (i) is it required to be durable or not?
- (ii) is it necessary that it be permeable, or impermeable, or not?

Durability is still an unresolved question for geotextiles because their recent development has not allowed time for extended trials in practice; and accelerated tests are not trustworthy guides for major works. At present polyamide, polypropylene, polyester and polyvinylchloride geotextiles all appear to perform well in soil (one field trial by the author using a polypropylene geotextile is still in perfect condition after 7 years in service); but it is widely conceded that long term exposure to sunlight can be very deleterious: the author has seen polyethylene which was buried only 1 - 2 cms

below surface rotted in 3 years. Suitable durability tests and criteria are poorly defined or even non-existent at the present time.

Project economy will obviously be affected by the choice of permeable or impermeable fabric - there is no point in paying for material that is not "working". If the purpose of the fabric is to prevent crushed rock penetrating into soft soil, an open mesh comparable to the stone size will be adequate. If it is necessary to restrict water movement, then clearly an impermeable membrane is needed. Yet even in this latter case, if it is necessary only to restrict (not prohibit) water transfer, total impermeability will not be an imperative - with appropriate savings. A typical situation of this sort is where seasonal or intermittent flooding occurs, and soil layers must be protected for limited periods against excessive softening by moisture uptake. The design principles for such cases have been described (Ingles, 1977).

There are three basic fields of application for fabrics in soil engineering, each with somewhat different membrane requirements:-

- (i) Water Control Structures e.g. drains, dams etc.
- (ii) Load Bearing Structures e.g. roads, foundations, walls etc.
- (iii) Erosion Resistant Structures e.g. channels, slopes etc.

The current situation for each appears to be as follows.

(i) Major economies for dams now appear within reach, given more research into the water transmission and filtration properties of geotextiles. Their use as internal filters replacing conventional graded sands has so far been restricted to smaller earth dams (Giroud et al., 1977; Loudiere, 1977; McDonald et al. 1981) but this was due chiefly to a lack of long term filtration performance data and filter design criteria appropriate to geotextiles, plus the criticality of these structures. Now, the initial success of such structures has been complemented by the development of excellent fabric filter design criteria by I.C.I. (Lawson, 1979) and Schober & Teindl (1979), permitting the selection of geotextiles that satisfy both permeability and piping constraints for subsurface drainage systems. This major advance applies as yet only to rather low hydraulic heads, but the trend is clearly very encouraging. A modified version of these filter criteria was used recently to determine the optimal hydraulic properties for a geotextile which was to be incorporated under the downstream drainage layer of a 30 m. high ash dam. The ash ($D_{85} = 0.055 \text{ mm}$, $D_{15} = 0.02 \text{ mm}$)

required a filter capable of withstanding a service head of 10.5 m. water. Conventional granular materials were too expensive, hence geotextiles were considered. Laboratory tests were made to model the flow and piping characteristics of the ash when placed adjacent to various geotextiles, and the new filter criteria referred to were wholly validated.

(ii) In the case of load-bearing structures, there is great conflict in the available literature as to the reinforcing effect of geotextiles. Basically, whether a geotextile acts only as a separator, or as a separating and reinforcing layer in the structure depends on the degree of strain applied to the geotextile in the soil, elastic modulus, creep, and anchorage of the geotextile, soil/textile friction, textile location and placement technique.

To obtain a reinforcing response from the geotextile, strains developed in the soil mass must activate the load carrying mechanism in the geotextile. The paramount property here is the elastic modulus of the geotextile. But creep is also very important (Al-Hussaini, 1977). Continuous, uniform strength may not be essential, since a single strong geotextile strand will act quite similarly to the metal strips used in the Terre Armee method for soil reinforcement (cf. Bell & Steward, 1977; Broms, 1977). Nevertheless, the picture seems to be emerging that geotextiles when placed by conventional methods in the soil do not reinforce in a plane parallel to the fabric itself; which makes it imperative to anchor the fabric, as suggested by Broms, and as also applied practically in such systems as the MESL (Ingles & Lawson, 1977). Lack of coplanar reinforcement probably arises from inadequate stress level on the geotextile, slip between the soil and reinforcing strands (cf. Brown et al., 1980) or creep, or simply an inadequate elastic modulus. All these aspects are currently being researched by manufacturers so that if required, geotextiles of very superior strength (up to 100 tonnes/m.) and friction characteristics are now available to the user. It must be emphasized however that proper placement is always vital (Yamanouchi, 1970; Mitchell, 1979).

(iii) Critical for erosion control is the edge detailing. The writer has seen numerous cases where inadequate detailing completely destroyed the illusion of protection afforded by a fabric. The proper role of a geotextile is here underneath a conventional protective layer, be it stone or concrete, or vegetation (e.g. grass). Even in these cases, fabrics are not immune to attack by yabbies or termites etc., and it must be remembered that the protected soil may also need a superficial pesticide application.

As mentioned before, the pertinent materials criteria for all geotextile applications are still ill-defined, and will probably only emerge with practice. The designer should proceed with prudence, yet not confuse this with conservatism which would deny him a very remarkable and versatile resource. Its growth potential in engineering practice has been estimated as very great (Ingles, 1978), and for the practical engineer it is well worth noting that the recent Delphi survey of future world advances in soil placement and improvement rated as 7th and 8th most desirable but 2nd and 5th most feasible :-

"Horizontal reinforcement nets for embankments on weak soil" and "In situ installation of soil reinforcing members"

respectively. In the same survey

"Embankments of soil encased and stacked in tough, durable membranes to eliminate the need for drying and compaction"

was rated the 5th most desirable and the most practicable of all innovative developments suggested. (Bell, 1976).

Clearly, there is a great practical advantage immediately at hand with these new materials, wisely used.

REFERENCES

- Al-Hussaini, M.M. (1977) "Field Experiment of Fabric-reinforced Earth Wall" C.R. Coll. Int. Sols Textiles, E.N.P.C. Paris, I, 119-121.
- Bell, J.R. & Steward, J.E. (1977) "Construction and Observations of Fabric Retained Soil Walls" C.R. Coll. Int. Sols Textiles, E.N.P.C. Paris, I, 123-128.
- Bell, J.R. (1976) "Future Advances in Soil Placement and Improvement" A.S.C.E. Ann. Conv., Preprint 2775
- Broms, B.B. (1977) "Polyester Fabric as Reinforcement in Soil" C.R. Coll. Int. Sols Textiles, E.N.P.C. Paris, I, 129-135
- Brown, S.F., Brodrick, B. & Pappin, J.W. (1980) "Permanent Deformation of Flexible Pavements" Tech Rept. DAERO 77-G-061, 78-G-114, University of Nottingham
- Giroud, J.P., Gourc, J.P., Bally P. & Delmas, P. (1977) "Behaviour of a non-woven Fabric in an Earth Dam" C.R. Coll. Int. Sols Textiles, E.N.P.C. Paris, II, 213-218
- Ingles, O.G. (1977) "The Permeability of Geotechnical Fabrics - its Reduction and Modification to suit Particular Uses", C.R. Coll. Int. Sols Textiles, E.N.P.C. Paris, II, 323-328.
- Ingles, O.G. & Lawson, C.R. (1977) "MESL - A New Appraisal" Proc. 9th Int. Conf. Soil Mech. & Found. Engg., Tokyo, I, 555-560
- Ingles, O.G. (1978) "Soil Stabilisation - The Next One Hundred Years" Proc. Symp. Soil Reinforcing and Stabilizing Techniques, N.S.W. Inst. Tech., Sydney, 365-383
- Koerner, R.M. & Welsh, J.P. (1980) "Construction and Geotechnical Engineering using Synthetic Fabrics" Wiley-Interscience. N.Y.
- Lawson, C.R. (1979) "Membranes in Geotechnics". 6th Asian Reg. Conf. Soil Mech. and Found. Engg., Singapore, 1-22
- Lewis, M. (1978) "Bibliography on Construction Fabrics", Oregon State University, Oregon
- Loudiere, D. (1977) "The Use of Synthetic Fabrics in Earth Dams" C.R. Coll. Int. Sols Textiles, E.N.P.C. Paris, II, 219-233
- McDonald, L., Stone, P. & Ingles, O.G. (1981) "Practical Treatments for Dams in Dispersive Soil" Proc. Xth Int. Conf. Soil Mech. and Found. Engg., Stockholm, 2, 355-360
- Mitchell, J.K. (1979) "Ground Reinforcement Techniques-Overview" A.S.C.E. Seminar on Ground Reinforcement, Washington D.C.
- Schober, W. & Teindle, H. (1979) "Filter Criteria for Geotextiles" Proc. 9th European Conf. Soil Mech. and Found. Engg., Brighton
- Yamanouchi, T. (1970) "Experimental Study on the improvement of the Bearing Capacity of Soft Ground by Laying a Resinous Net" Proc. Symp. Founds, on Interbedded Sands, CSIRO, Perth, 144-150

H. Cambefort, Chairman

Je profite de ces essais de JURAN et al. qui montrent que les barres d'acier ne peuvent pas être cisaillées par le sol pour rappeler que une telle flexion et non un cisaillement, se retrouve toutes les fois que les barres sont plus résistantes que le milieu environnant. Les colonnes balastées peuvent donc être fléchies et comme leur résistance à la flexion est petite on peut se demander comment elles peuvent augmenter la résistance au cisaillement d'un massif.

Pour comprendre leur mode d'action, je considère un remblai susceptible de provoquer

un glissement dans le sol qui le supporte. Si l'on dispose des colonnes balastées sous le remblai, celles-ci vont prendre une partie de son poids, le complément de celui-ci étant supporté par le sol. Il suffit alors que le nombre des colonnes soit suffisant pour que la contrainte résiduelle appliquée directement à la surface du sol ne soit pas susceptible d'amorcer le glissement.

Ainsi l'efficacité des colonnes balastées est indirecte. Elle est due à leur force portante et non à leur résistance à la flexion mais absolument pas à leur résistance au cisaillement puisque un sol mou ne peut pas les cisailler, contrairement à ce que fait croire la fig. 42 du State of the Art.

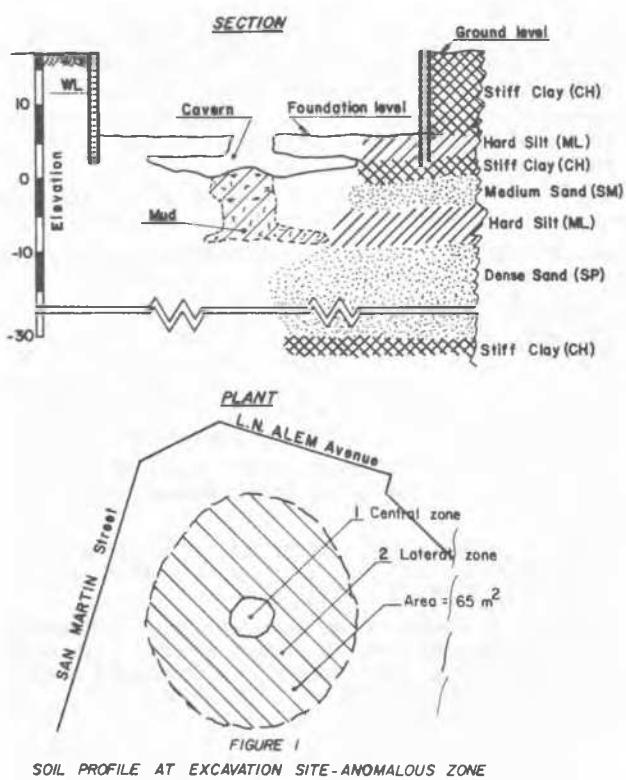
C.A. Micucci (Oral discussion)

A COMBINED CASE OF SOIL IMPROVEMENT Une Case Combinée d'Amélioration de Sol

INTRODUCTION

This contribution presents a combined problem of improvement by drainage, stabilization by means of stone columns, and preload of foundations carried on in a large excavation to allow foundations be made.

The soil profile at excavation site is shown in Fig. 1. Soil investigations showed that there were two levels of water energy. The first, at elevation +12 was phreatic line. The second corresponding with a confined aquifer situated in the dense sand layer arised up to elevation +8



SOIL PROFILE AT EXCAVATION SITE-ANOMALOUS ZONE

Drainage

To excavate it was necessary a drainage system. Excavation was considered as a partially penetrating well in to a gravity aquifer with a circular source. For the drainage of the upper levels, seven deep wells penetrating in hard silts were supplied. To avoid the uplift from the artesian aquifer, three relief wells were installed in to dense sand.

Piezometers demonstrated that stabilization of phreatic line at elevation +5 took ten days. The piezometric from the artesian aquifer was lowered at elevation +6,20.

A Surprise

When excavation was completed, a new trouble appeared. It were a cavern and a hole, both filled with mud (see anomalous zone in Fig. 1). Some old wooden sheet piles buried in the mud were discovered; then it was thought that an old locomotive water supply was made visible. In view of these difficulties other improvement was accomplished.

Stone Columns

The installation of grouted stone columns was decided and executed as shown in Fig. 2.

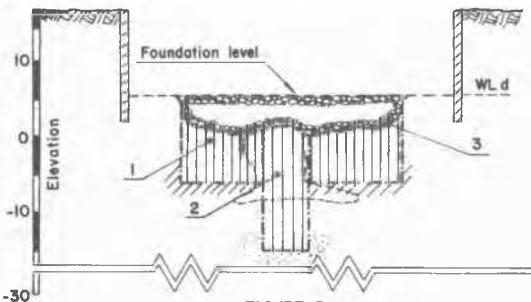


FIGURE 2
ANOMALOUS ZONE - STONE COLUMNS TREATMENT

- 1 - Lateral zone 33 stone columns in Hard Silt
- 2 - Central zone 5 stone columns in Dense Sand
- 3 - Gravel fill

In central zone five 18" diameter long columns up to dense sand were placed. In lateral zones

thirty three 20" diameter short stone columns reached into hard silt. Columns were grouted after the completion of the work.

K.R. Datye and S.S. Nagaraju (Oral discussion)

BEARING CAPACITY OF STONE COLUMNS

The discussion relates to equation 32 on Page 297 of the general report where a parameter of 25 is suggested for the relation $\sigma_v = F.S.$

$\sigma_v = F.S.$. The load test data presented in our paper seemed to agree with the above relation as far as the sectional area in the upper part of the stone column was concerned. However, when the test results were re-evaluated, they revealed σ_v of the order of 45-58 Tons/m² in the portion of the stone column below 2 m depth. (Table 1). This implies a value of 45-58 in lieu of 25 for rammed columns. Even if allowance is made for the load transfer in the upper part of the column which may bring down the value of the parameter to some extent, there is reason to believe that significant soil improvement has been achieved in the annulus around the stone column during the ramming process.

Table 1 : Vertical stress in stone columns at different levels.

| Column Location | Bagging Plant I | | | Bagging Plant II | | |
|------------------------------------|-----------------|------|------|------------------|-------|-------|
| Nominal diameter mm | | 600 | | | 400 | |
| Yield load, t* | | 25.8 | | | 27.52 | |
| Sectional area, ** sq.m | a | b | c | a | b | c |
| | 0.89 | 0.44 | 0.39 | 1.13 | 0.68 | 0.57 |
| Vertical stress, $\sigma_v, t/m^2$ | 28.9 | 58.5 | 65.2 | 27.3 | 45.1 | 106.7 |
| Design shear strength $C_u t/m^2$ | 1 | 1 | 2 | 1 | 1 | 2 |
| σ_v/C_u | 29 | 58.5 | 33 | 27 | 45 | 53 |

a At 1 m depth below ground

b At 3 m depth below ground

c At 6 m depth below ground

* Yield load is taken as the load at which $\Delta s / \Delta p$ increases by an order of magnitude compared to the previous stage of loading. Δs is the observed increase in settlement for an increase in load of Δp

** Sectional area is calculated on the basis of actual consumption of stone and sand. Stone and sand in the proportion of 2:1 was measured loose and compacted volume taken as 0.8 of loose volume of stone and sand. For purposes of analysis, the column cross section is considered

Preloading

By means of a "tube a manchete" system the foundation was preloaded up to 200 KPa in order to ensure an adequate behaviour.

uniform between the depths where set is measured.

The analysis does not account for skin friction transfer. Analysis (using elastic solutions for compressible piles) for vertical displacement of a point at 2 m below top, located at a distance $d/2$ from the centre of the pile, shows a displacement of the same order of magnitude as the column displacement, hence skin friction transfer is not considered in estimating the vertical stress in the stone column.

Factor of Safety:

Further a factor of safety of 3 appears to be over conservative for rammed columns when a ramming 'set' criterion is used and single column load tests are used to verify column quality. A factor of 2 can very well be used when good control is exercised. This may be even reduced to 1.5 when the stone column derives support from the lateral over-burden pressure for example when stone columns are used in the central strip below an embankment.

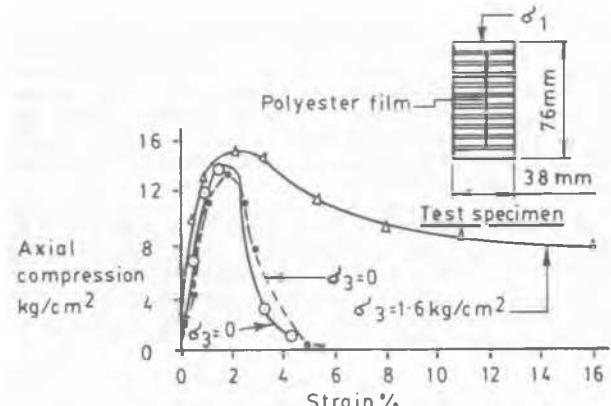


Fig. 1. Lime flyash soil polyester film confinement

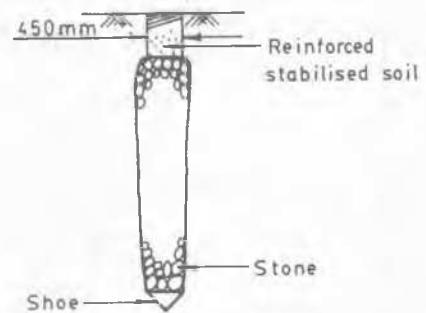


Fig. 2. Composite stone column

Use of polyester reinforced zone in upper sections:

Experience has shown that the most vulnerable part of the stone column is the top zone; say 0-2 meters. An alternative is being explored

H. Aboshi (Oral discussion)

A PRACTICAL EXAMPLE OF SOIL STABILIZATION BY PRECOMPRES- SION TECHNIQUE

To keep pace with the industrial growth in Japan since the 1950's, there has been rapid development of man-made lands in the coastal region of the country. The total area is estimated to be about 113,000 ha, which is about 1.5 % of the total area where man can live in the country.

As almost all of these reclaimed lands have been constructed on soft alluvial clays on the coast, the geotechnical engineer has always been confronted by "poor ground problems". And, as a result, many kind of soil stabilization methods, including vertical drains, have been developed. At an earlier stage, one of the most important concern for the engineer is how to construct these fill-up lands without failure. However, as their experience proceeds, their engineering requirements have become up-graded.

Usually, in making these new lands, reclamation or fill-up to 5 m above sea level is laid on the sea bottom 10 to 15 m deep, the sea bottom consisting of 20 to 30 m of soft alluvial clays.

Total consolidation settlement in such cases, usually reaches 4 to 6 m. Under such conditions, the engineer is asked to decrease the residual settlement both by the reclamation fill itself and the load of structure constructed afterwards, up to negligibly small.

Here I should like to show an example. 1st slide shows the construction site of a large-scale sewage treatment plant on a reclaimed land on the western coast of the city of Hiroshima and this photograph shows a stage of soil stabilization by the precompression technique. You can see on the fill-up land, about 6 m high surcharge fill and a dewatering pipe line system used to lower the ground water level by 9 m. This is a stage of excavation after the stabilization was finished.

2nd slide shows the construction of structures, which consisted of a large caisson, 80 m long, 40 m wide and 27 m deep, and a long water treatment pool, 230 m long, to be exact, 40 m wide and 18 m high. This long water treatment structure does not tolerate differential settlement, even though the total settlement by consolidation of 30 m thick clay can reach several meters.

which involves use of lime-fly ash stabilised soil reinforced by spirals or loops of polymer. Fig. 1 shows the test results. Considering the remarkable ductility of the material, columns can very well be designed for σ_v of 100-150 T/m². A typical composite column is shown in Fig. 2.

Ordinarily, such water treatment pools are constructed on pile foundations, but considering the negative friction on piles and the difference of settlement between the structure and the connecting pipe lines, it was decided to use the precompression technique with sand drains to accelerate consolidation. Not even a single pile was driven.

Series of consolidation tests, using a separate-type consolidometer have been carried out, in order to predict the residual settlement after precompression in the field. Fig. 1 shows data on a surcharge-rebound-recompression settlement, as a model test of the practical precompression technique. In the figure, curve I shows the consolidation by a constant load equivalent to the surcharge in the field. On the other hand, in curves II, III, IV, the surcharge is once removed to zero at a certain degree of consolidation, and after their rebound stage, a load equivalent to the structure is applied. Curve V is the case in which the load of structure is maintained from the beginning until the end. You can see that all of the rebound-recompression curves fall in between the curves I and V.

The most important finding is the relation between the gradient of creep settlement and the precompression method.

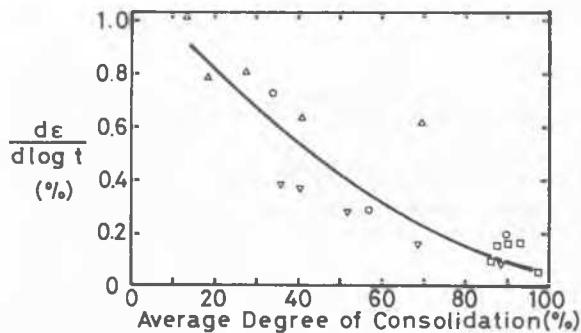


Fig. 2 Gradient of creep settlement.

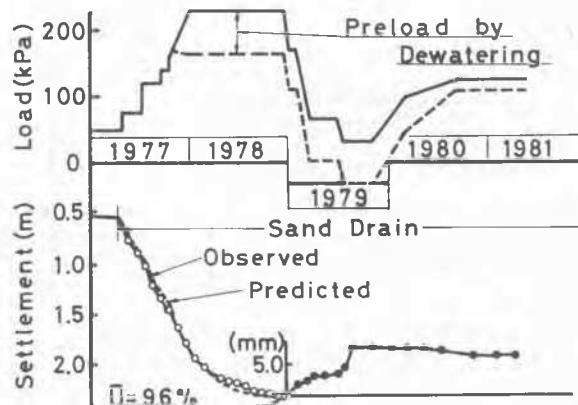


Fig. 3 Time-settlement curve in the field.

The gradient clearly shows the tendency to decrease with the increase of the degree of consolidation at the time of rebound stage. Fig. 2 shows the relation between the average degree of consolidation at the time of rebound and the coefficient of secondary creep settlement. There seems to be a unique relationship, even though both data from total layers and separate portions are included in the same Fig..

In order to meet the design requirements concerning residual settlement in the field, 95 % of consolidation by pre-compression is adopted. Fig. 3 is the measured settlement-rebound-recompression curve in the field, in which the re-

bound and recompression stages are shown in a different scale. You can see that the settlement by recompression caused by the load of structure is practically zero, in spite of its large total consolidation settlement. Soil stabilization work has been proved perfectly successful. By replacing pile foundations with the precompression technique, total cost of construction has been reduced by about 5,000,000,000 Yen (that is about 25 million US dollars).

Ref. Aboshi, H. et al (1981): Report to Symp. Geotechnical Aspect of Offshore & Nearshore Structures, Bangkok

G. den Hoedt (Oral discussion)

FILTER SLEEVES FOR PREFABRICATED VERTICAL DRAINS

In the State-of-the-Art Report of Session 12 (soil Improvement) four criteria to be met by filter sleeves have been specified (p. 277).

The second criterion is that "Fine soil particles should be retained to prevent clogging of flow channels in the core".

Although it seems to be a very safe method to prevent clogging, the choice in favour of such a retaining filter might lead to undesired consequences that will prevent a good functioning of the drain.

As most band-shaped drains nowadays are installed by displacement methods, it is generally assumed that a zone of smear is created around the hole caused by the mandrel.

When after some time the hole closes around the drain, this remoulded zone of decreased permeability will be located just against the drain filter.

When no fine particles at all are allowed to pass this filter, the entrance resistance of the drain - which in general is already high for the retaining type of filters - is still increased.

When on the other hand the filter allows the passing of fine particles carried along with the excess pore water flow, a natural graded filter is formed in the soil around the drain. This extra filter zone has a higher permeability than the natural soil, thus satisfying another important drain criterion probably not always met by the retaining paper filters.

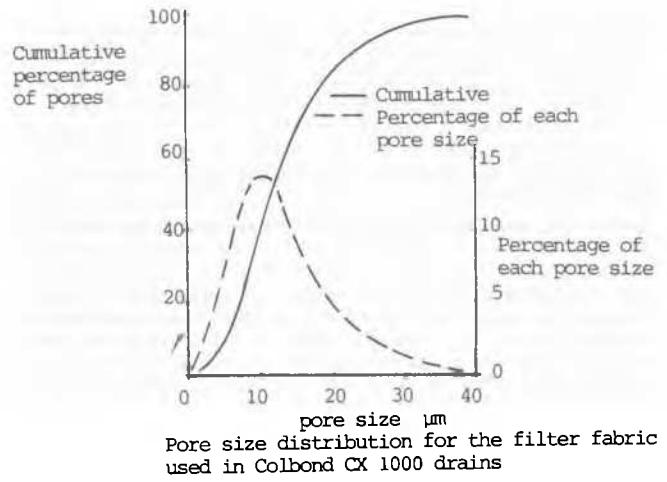
Laboratory investigations with the Enka drain tester (details of which are given in my paper 4/22 "Laboratory testing of vertical drains") have shown that the Colbond CX 1000 drain discharged some clay in an initial period of about 2 hours only.

After that period the clay discharge ceased completely, whereas the water discharge stayed on a good level.

Of course, the fabric pore size cannot be chosen too large, because then probably a continuous piping

will occur, including heavier particles that are not so easily entrained by the upwards directed water stream, so that clogging might occur.

The figure shows a graph of the pore size distribution for the polyester filter fabric used in the Colbond CX 1000 drain.



Pore size distribution for the filter fabric used in Colbond CX 1000 drains

In a laboratory test with a particular type of sandy clay as surrounding soil the particles that passed the filter were under 5 μm diameter, of which 95% under 2 μm.

The graded filter zone appeared to have a thickness of the order of 10 mm around the drain.

Summarizing, I conclude that:

- to the best of our knowledge special purpose polyester filters (as used in the Colbond CX 1000 and probably in some other drains) behave better than completely retaining paper filters;
- more knowledge should be gained about the (hydraulic) conditions near drains in the field.

S. Hansbo (Written discussion)

ON FILTER SLEEVES OF PREFABRICATED DRAINS

In the discussion, Mr den Hoedt questioned the second criterion for filter sleeves of prefabricated drains given in the state-of-the-art report, namely that "fine particles should be retained to prevent clogging of flow channels in the core". He claims that a filter material which allows the passing of fine particles with the pore water flow would contribute to forming a natural graded soil filter around the drain. He also presents what he

considers to be the best filter fabric (the one used in Colbond CX 1000 drains). This filter has a pore size which falls almost completely within the grain size limits of the silt fraction (about 70% corresponding to medium silt). Although Mr den Hoedt has found, in a laboratory test, that this filter fabric retained all soil particles with grain size above 5 μm I find it difficult to believe that this can be true in practice. I can see no

reason why particles with grain size up to 20 μm would not pass through the filter, e.g. due to the suction created when the mandrel is withdrawn from the soil. This seems particularly probable when the soil contains layers or pockets of silt.

If silt and clay particles enter into the channels of the drain core this could have a disastrous effect on the discharge capacity of the drain. Thus, the prefabricated drains now existing on the market have a discharge capacity of maximum 20-25 m^3/year . With the definition of discharge capacity given in my paper this corresponds, for a 100 mm by 4 mm drain, to an overall value of longitudinal permeability of 50000-60000 m/year ($1.5 - 2 \text{ mm/s}$). If, for example, silt should fill the channels the overall value of permeability would decrease to less than 30 m/year (10^{-6} m/s), i.e. the drain would be almost clogged. According to investigations carried out at the Department of Geotechnical Engineering, Chalmers University of Technology, the paper filter is superior to a

synthetic filter. Thus, these investigations show that the discharge capacity is reduced when the filter sleeve is changed from paper to synthetic material.

One reason for this decrease may be that synthetic material is more easily squeezed into the channel system than the paper. Another reason may be that fines, because of the higher permeability of the synthetic filter, will pass through the filter and cause a reduction of the channel volume.

As shown in my paper in Session 12 (12/22) of this Conference, there is no need for a high filter permeability as long as the specific discharge capacity q_w is as low as that of the present types of prefabricated drain. Efforts to improve the prefabricated band-shaped drains should be concentrated on increasing their discharge capacity as much as possible, in other words on decreasing their well resistance. This is by far the most important problem concerning the prefab drains of today.

A.F. van Weele (Oral discussion)

ON VERTICAL DRAINS

Nowadays, we know sanddrains, installed by driving (displacement type) or by jetting (non-displacement type) and many different kinds of prefabricated drains. Too often, all these different drains are considered to be of equivalent quality and thus also of equal capacity. This results in one layout for the drains and a competition in price per linear meter of drain only. This is incorrect.

Every draintype has its own filterresistance in the soil. This resistance in combination with the drain's circumference determines to a great extent the amount of excess porewater that will enter the drain under a given excess pressure. The supplier of prefabricated drain-material should be able to inform his clients about the magnitude of the filterresistance of his material as a function of time. So far, however, most are not. Tests carried out in The Netherlands learned that some of the better known drainmaterials show a very large increase in filterresistance within 24 hours when subject to a constant hydraulic head. In some cases the amount

of inflowing water was less than 2% of that at the start. Other materials show hardly any change with time. A second important property is the longitudinal capacity of the drain. If this capacity is large, as e.g. for tube drains, there is hardly any excess pressure in the drain itself. For several types banddrains, however, the longitudinal capacity is limited so that in compressible soilayers with a large supply of excess water, excess pressures will develop in the drain. Under such circumstances, the resulting consolidation period of the soil may be extended substantially. Suppliers of these materials are often not at all aware of the requirements their materials have to fulfil in order to function properly. They are happy with good results elsewhere and think that what applies in one soil condition applies to all possible soil conditions. This, however, is far from the truth. Also the users should be more knowledgeable in order not to compare apples and pears.

M. Wallays (Written discussion)

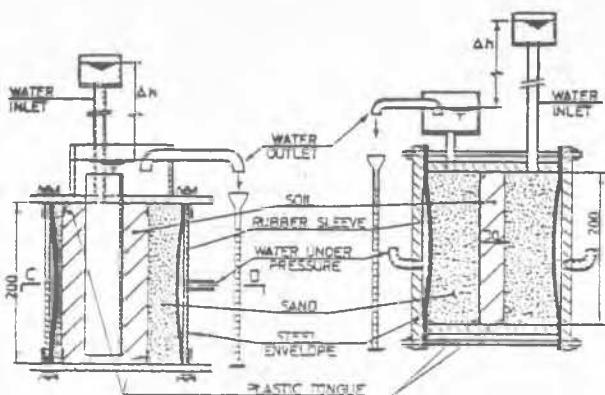
SOIL IMPROVEMENT BY PRECOMPRESSION - VERTICAL DRAINS

Because some decreases versus time of the water flow through prefabricated drains have been observed in tests under constant gradient, it is wondered whether the use of filter envelopes, allowing the finer soil particles passing through, is not advisable? In our opinion, such a decrease of the water flow is due to the decrease of the soil permeability caused by the consolidation, and is not provoked by the migration of fine particles inside the soil mass towards the drain. Fig. 1 gives the scheme of the equipment manufactured in 1974 at University of Liège at Franki request, in order to check whether the Kjellman carboard drain and the plastic Franki drain are not clogging during the percolation. The test is made with water percolating under a height $\Delta h = 2,50 \text{ m}$ through a 30 mm thick soil sample. The soil and the drain are maintained under a lateral pressure of 0,2 MPa. Three types of remolded soil in a soft state were placed round the drain: loam, fat clay and bentonite mixture. During the percolation, the flow decreased during 2 to 3 months and stabilized afterwards. After the equipment has been dismounted, the drains appeared fully serviceable (fig. 2 and 3) but the soil samples had consolidated and had taken a diabolo shape (fig. 4, from left to right: loam, clay,

bentonite mixture), due to the fact that the deformation along the edge is prevented. These observations suggest to check whether the flow decrease is not due to the drain. Therefore, additional equipment (fig. 5) was manufactured in order to test, without drain, a soil sample in the same conditions as hereabove. The sample has the same initial 30 mm thickness and a length equivalent to the perimeter of the soil sample placed round the drain at its half-thickness. A similar decrease of the water flow was experimented. These tests make unquestionable that the decrease of the water flow is caused by the decrease of the soil permeability provoked by the consolidation process. With soil samples, only 30 mm thick, it is unlikely that the migration of fine particles inside the soil mass could influence the water flow in a significative way, particularly in cohesive soils as fat clay and bentonite mixture. The decrease of the soil permeability has been also evidenced on the field, during the consolidation of a soft alluvial layer accelerated by Kjellman carboard drains (De Beer et al, 1973).

The use of envelopes allowing the finer soil particles passing through the filter is not without danger, because

ELEVATION A-B



SECTION C-D

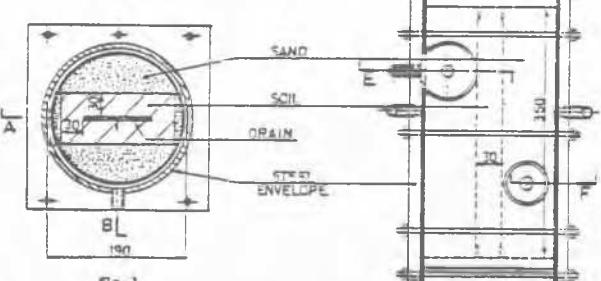


Fig. 1

Fig. 4

FIG. 5

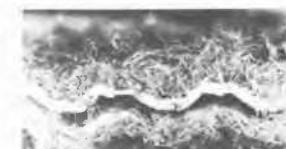


FIG. 2

FIG. 3



+



K.W. Cole (Oral discussion)

VERTICAL DRAINS - PREDICTION OF PORE PRESSURE DISSIPATION RATES

In the section on vertical drains in his general report Professor Mitchell drew attention to the unsatisfactory state of prediction of pore pressure dissipation rates.

During the last few years Ove Arup & Partners have had the responsibility for forecasting the rate of settlement of three embankments at Sandwich, Queenborough and Belfast. At all three sites the rates were critical to the overall construction periods of the respective works, as the embankments had to be substantially consolidated (95% primary consolidation complete) before the foundations of adjacent bridgeworks were permitted to be placed.

In dealing with these particular sites we have been fortunate in that the significant strata

have been remarkably uniform in their drainage characteristics and this has enabled us to cross-check field and laboratory test data with back analyses of the actual performance of the embankments.

While it is well described in published texts (e.g. Rowe, 1968) that the coefficient of consolidation c_v changes with changing effective stress, it has not hitherto been easy to undertake computations using more than one value (or perhaps two values) of c_v , because of the laborious and error-prone nature of the work. However by using a computer for the step by step calculation of the progress of pore water pressure dissipation the values of c_v can be related to the current values of the effective stress and

In our opinion, the installation of a vertical drainage is effective when the following conditions are fulfilled :

- The relative volume of soil displaced when driving the mandrel must be as low as possible. The mandrel bottom must have a slight enlargement, in order it to cut up the smeared and remolded soil thickness along the mandrel, when lifting the latter up.

- The filter envelope of prefabricated drains must form an effective filter, allowing water to flow through and soil particles to be stopped. The transverse permeability of the envelope has little influence because its very low thickness.

- The vertical permeability of the drain (the core in the case of a prefabricated drains) must be suitable to the flow and the percolation path, in order the water head loss to be low enough. Aboshi and Yoshikuni (1967) developed a suitable method, in order to take into account the decrease of the consolidation speed due to the well resistance.

- The permeable carpet, through which the water flows from the drains to the downwards ditch, must offer a suitable low resistance to the water flow. Until now, not much attention has been paid to this point. Nevertheless, calculated assessments show that a 1 m thick sand carpet can too much slow down the water flow, when the flow and the maximum percolation path are large, so that in some cases the use of drainpipes is needed.

References :

Aboshi H and Yoshikuni H, (1967), A study of the consolidation process affected by the well resistance in the vertical drain method. Soil and Foundation (VII), 4, 38-58, Tokyo.

De Beer E, Wallays M, Paquay J and Veillez A, (1973), Consolidation accélérée au moyen de drains en carton. Proc 8th ICSMFE, (2.2), 31-36, Moscow.

Université de Liège, Service Géotechnique, (1974-1976), Procès-verbaux d'essais demandés par Pieux Franki, (not published).

a greater precision in forecasting is therefore possible (Cole, 1980).

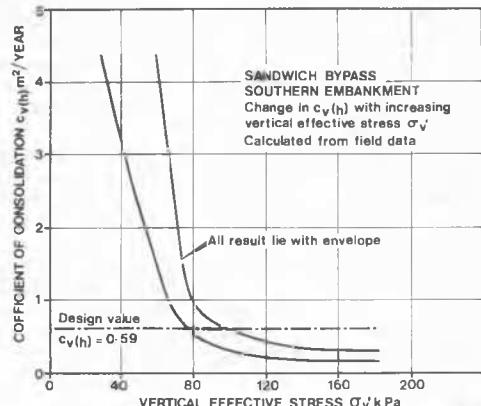


Fig. 1 Sandwich By Pass. $c_v(h)$ against σ'_v

Figures 1, 2 and 3 show the correlation between field and laboratory measurements of the $c_v(h)$ against vertical effective stress obtained from the three sites, at all of which vertical drains were used. For the Sandwich site the calculations were manually done and a single "average" value of $c_v(h)$ was adopted. Although at the time this seemed a reasonable approach it overestimated by a factor of about three the rate of dissipation in the later (and more crucial) stages of construction.

At Queenborough see Figure 2, the problem was more fully recognised, and two values of $c_v(h)$ for manual calculation were chosen. Again these proved to be somewhat on the optimistic side as the rate of dissipation in the latter stages of construction was overestimated.

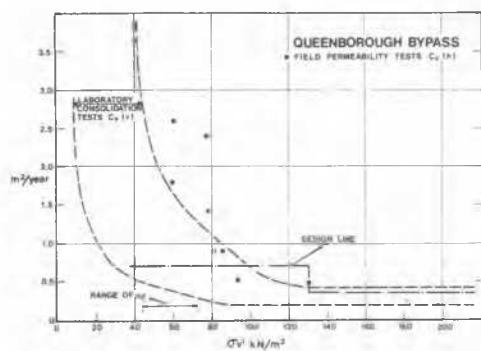


Fig. 2 Queenborough By Pass. $c_v(h)$ against σ'_v

O.G. Ingles, Panelist

THERMAL STABILIZATION La Stabilisation Thermale

I wish to say a few words on the subject of thermal stabilisation.

This method has been practised successfully in Romania and in Russia, but never attracted much world attention because of costs. Nowadays, with the high price of fuel, many think it should be forgotten. But on the contrary, we may be on the brink of important new developments in

For the Belfast site the field and laboratory test values of $c_v(h)$ were used to predict the performance of the trial embankment with good results as shown in Figure 4. The back-calculated values of $c_v(h)$ were also shown to be close to the predicted value of $c_v(h)$ as shown in Figure 3.

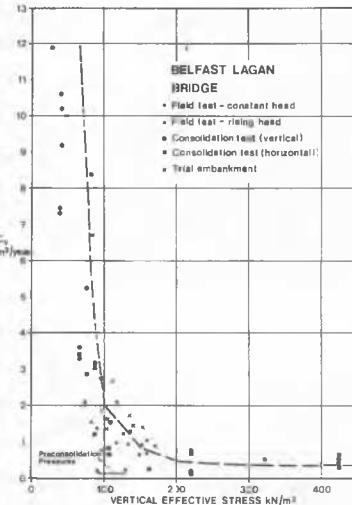


Fig. 3 Belfast Lagan Bridge Approach. $c_v(h)$ against σ'_v

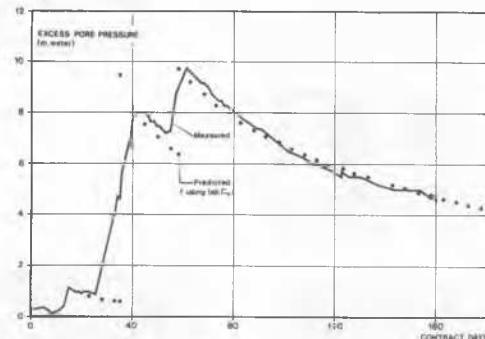


Fig. 4 Belfast. Lagan Bridge Approach. Excess head against time.

It is thus concluded that with careful testing to obtain realistic values of c_v over the full range of stress change that will be imposed, and by the use of a computerised calculation procedure, it is possible to make reasonably accurate predictions of the rate of dissipation of excess pore water pressure.

References: Please see paper 1/12

thermal stabilisation. They are already foreshadowed by several papers in this conference. After all, if one can achieve a material as hard as brick from loose soil, using less than 1.1/2% fuel oil as "additive", is it not more attractive, more energy efficient, than 8 to 10% cement?

The problem with thermal methods has been fuel economy.

But this problem is easily solved by the fuel engineer today. Moreover, it is possible to achieve the thermal stabilisation of clays at very low temperatures (around 550°C, less than red heat). At these temperatures not only is clay swelling destroyed, but the clay surface is activated. Three papers in this conference (Queiroz de Carvalho, Brandl, and Matsuo & Kamon) have all shown how silica, alumina and ferric oxide the commonest components of all soils when activated by various means, lead to good soil stabilisation. Prof. Brandl's paper is especially excellent (in my opinion one of the finest papers in the Session) and I commend it for reading.

What I wish to emphasise here however, is that the necessary activated state can also be produced by low heat. Recently, myself and a coworker, Lim, succeeded in converting such diverse clays as bentonite and kaolin, as well as ordinary soils, into water-resistant durable materials of high strength (exceeding 1×10^4 kPa dry and 8×10^3 kPa soaked) after room temperature curing

H. Cambefort, Chairman

AMELIORATION THEMIQUE

La consolidation thermique n'a fait l'objet que d'une communication: celle de notre secrétaire de Dr. Knutsson qui a étudié la résistance du cisaillement des sols gelés.

Il s'agit là d'une recherche très intéressante. Mais il y en a une autre qui l'est tout autant, c'est le comportement du sol au moment du dégel. A ce moment là, en effet, certaines argiles comme celle de Mexico par exemple se liquéfient. Il en est de même du yaourt. Quant à la gelée de groseilles on n'arrive pas à la congeler à moins 20°C, probablement à cause de sa forte teneur en sucre. Il semble que cette liquéfaction soit le propre de certaines structures des gelées colloïdales. On le constate avec les cerises qui suivant leur espèce se liquéfient ou non, comme les argiles. Avant de consolider le sol par congélation, il faut donc s'assurer que le dégel ne peut pas être catastrophique.

La cuisson des sols a été bien moins utilisée que la congélation. Je ne connais que les exemples donnés, il y a un peu plus de vingt ans, par le Prof. Stanculescu.

H.L. Jessberger and W. Ebel (Written discussion)

SIMPLIFIED TEST CONDITIONS FOR DESCRIPTION OF THE STRESS-STRAIN BEHAVIOUR OF FROZEN SOILS

For the investigation of the soil behaviour more and more highly sophisticated methods are used by the scientists. As our last President Prof. Fukuoka mentioned in his presidential address we should not forget to perform relatively easy tests which give soil parameters for the description of the soil behaviour. Based on these parameters a first calculation of a structure can be made and the parameters can be used for fixing the test conditions of highly sophisticated test methods. The presented test methods are related to the soil improvement by ground freezing. The basic idea of this proposal is to perform so-called "Reference Tests" which enables us to identify the frozen soil due to the stress-strain behaviour for comparison with other frozen soils. It should be emphasized that the proposed test method is not a standardization and that it is necessary to perform the

for 7 days. I have called this process "accelerated laterisation", for although a stabilisation, it closely resembles that phenomenon in its effects and operation. The quality of the product is readily conveyed by striking two cylinders of the treated soil together - the good ring of a strong material is heard, from what was once loose loam.

Although I do not agree with the structural interpretations of Matsuo & Kamon (and which are also at variance with the observations of Swartz, Yates & Tromp in this Session), electron microscope photographs of the new "accelerated laterisation" products have many similarities to those shown by Matsuo & Kamon. The strengthening products have an essential needle-like habit. Electron probe analysis of these products, however, has led to quite different conclusions from those of Matsuo & Kamon, and in more substantial agreement with the conclusions of Queiroz de Carvalho and of Brandl. A detailed account is to be presented to the International Conference of Soil Mechanics, Mexico City, 1982.

J'ai essayé de le copier pour arrêter un glissement de talus, mais sans succès car les sols étaient saturés. Nous étions sortis du domaine d'utilisation du procédé.

Puisque j'en suis aux effets de la température, je rappelle qu'une augmentation de celle-ci augmente la vitesse du fluage, probablement par suite de son action sur la double-couche. Peut-être y-a-t-il là une explication aux tassements des petites compactages de Linköping.

Par contre les déplacements des murs de soutènement consécutifs aux variations de température ne seraient sans doute dûs qu'à la dilatation des sols. Il semble en être de même pour les oscillations de pieux traversant des sols mous saturés. Mais quand les fondations profondes traversent des limons non saturés, leurs oscillations sont peut-être dues aussi à la thermo-osmose, phénomène qui a eu sa période de succès autrefois et dont l'étude est passée de mode aujourd'hui.

Reference Tests in addition to other project related tests.

Two types of tests were selected: The unconfined compression test and the unconfined creep test. The creep tests are introduced because the creep behaviour of frozen soils can be important depending on the magnitude and duration of loading.

The Reference Test will be performed on cylindrical soil samples. The slenderness ratio should be $h/d = 2/1$ with a minimum diameter of 5 cm. The tests should be performed at a constant temperature $T = -10^\circ \pm 0,5^\circ\text{C}$.

From Table 1 can be seen that two compression tests are selected with different strain rates. Due to the strain rates the duration of the fast compression test will be 15 minutes whereas

the duration of the slow compression test will be 12.5 hours.

TABLE 1 COMPRESSIVE STRENGTH TESTS

$$\begin{aligned}\sigma_a: \dot{\epsilon} = 0,02 \quad [\%/\text{min}] \\ \sigma_b: \dot{\epsilon} = 1,00\end{aligned}$$

| SOIL | σ_a | σ_b |
|------|----------------------|------------|
| | [N/mm ²] | |
| SAND | 6,0 | 12,0 |
| SILT | 2,5 | 5,0 |

In addition two creep tests are selected with different loads, related to the compressive strength of the fast compression test (Tab. 2). The duration of the creep tests should be at least one day for the highly loaded test and three days for the low loaded test if state of failure will not be reached earlier. The data in Table 1 and 2 are valid for the two soils of Fig. 1.

TABLE 2 CREEP TESTS

| SOIL | $0,4\sigma_a$ | t_f | $0,7\sigma_b$ | t_f |
|---------|----------------------|-------|----------------------|-------|
| | [N/mm ²] | [h] | [N/mm ²] | [h] |
| SAND | 4,8 | (100) | 8,4 | 3 |
| SILT | 2,0 | (100) | 3,5 | 3 |
| TIME[D] | | 3 | | 1 |

Fig. 1 shows possible creep test results for

H. Cambefort, Chairman

CLOSURE OF SESSION AFTER ORAL DISCUSSION

Voici venu le moment de conclure cette longue séance. Cela est difficile car l'amélioration des sols, fait partie de l'Art de construire et non des calculs. J'ai éprouvé les mêmes ennuis à la séance équivalente du Congrès de Moscou. Aussi il n'est pas étonnant que la conclusion de celle-ci puisse convenir aujourd'hui, en y changeant simplement le nom d'un orateur. Comme elle est ancienne j'en rappelle les points essentiels:

1 - Il faut souvent avoir d'autres connaissances que celles propres à la Mécanique des sols pour trouver un procédé d'amélioration valable,

2 - Les raisonnements a priori basés sur nos connaissances actuelles des sols sont quelquefois faux,

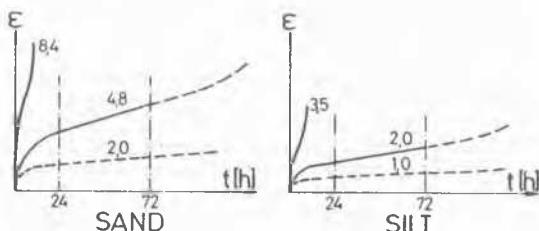
3 - Les procédés d'amélioration sont nombreux mais ils ne sont pas forcément interchangeables. Par ailleurs un même procédé peut être satisfaisant dans un certain sol et pas dans un autre pourtant semblable à première vue. Enfin,

4 - Le "savoir faire" des hommes de chantier est presque toujours plus important que la théorie.

Ce dernier point est très important, car il interdit les spécifications détaillées qui ne peuvent pas prévoir tous les cas possibles d'hétérogénéité des sols, à laquelle il faut s'adapter au fur et à mesure de l'avance-

sandy and silty material. The highly loaded creep test should definitely lead to the transition between secondary and tertiary creep behaviour. The low loaded creep test will show the steady-state creep behaviour of the second phase within 3 days, which means that the creep strain rate remains constant.

FIG.1 CREEP BEHAVIOUR



From the compression tests significant soil data as compressive strength and secant modulus can be obtained whereas the creep tests give strain at failure and the corresponding time at failure.

More detailed information concerning sample preparation, test conditions, data analyses together with the evaluation of number of "Reference Tests" is already published /1/.

/1/ Jessberger, J.L., Ebel, W. (1980): Proposed methods for Reference Tests on Frozen Soil, Proceedings Second International Symposium on Ground Freezing, Trondheim, Elsevier Scientific Publishing Company, Amsterdam

ment des travaux.

Quant aux autres points j'ai essayé d'en tenir compte dans l'Introduction en mettant l'accent sur la structure des sols. Il semble en effet évident que le comportement d'un sol est conditionné par celui de son squelette et non par l'évolution des pressions interstitielles qui ne peut introduire qu'un terme correctif. Mais comme il est difficile de préciser la structure d'un sol intact, on accepte facilement des simplifications qui malheureusement ne correspondent pas souvent à la réalité. Quand on suppose, par exemple qu'une argile ne diffère d'un sable que par la dimension de ses grains, plus ou moins bien collés entre eux, on oublie comme l'a rappelé le Prof. Katti, l'environnement électrique qui correspond à ce que j'ai appelé la gelée colloïdale, et qui à lui seul peut conditionner le comportement de la structure.

Pour mieux saisir celle-ci, et ce sera mon dernier mot, il faut faire de la physique.

En levant cette séance, je remercie encore tous les orateurs et plus particulièrement nos Rapporteurs qui n'ont pas ménagé leur peine pour établir des rapports qui n'on pas leur équivalent dans les Annales de ces Congrès.

H. Cambefort, Chairman (Written discussion)

COULIS CHAUX - CENDRES VOLANTES

Tous les résultats qui viennent d'être exposés sont très intéressants et me conduisent à quelques petites remarques:

1 - Tout d'abord de la mise en évidence du cheminement du calcium est remarquable, et fait penser au cheminement des ions que les géologues invoquent souvent. Sa progression avec le temps confirme que la bentonite n'est pas imprégnable avec un coulis, et que les claquages ou encore les fracturations hydrauliques provoquées par une injection peuvent avoir un effet bénéfique insoupçonné.

2 - Les diagrammes aux rayons X montrent bien que la bentonite s'est transformée, mais les modifications des pics ne proviennent probablement pas d'une simple accumulation d'ions dans la double couche. Il s'est passé autre chose et cet autre chose peut être dû à d'autres ions que ceux du calcium, car les cendres volantes qui sont des pouzzolanes doivent avoir une composition chimique très complexe. La nature de la bentonite peut aussi intervenir.

3 - L'exemple cité porte à croire que pour stabiliser un remblai de voie ferrée il est nécessaire d'utili-

ser un coulis d'injection constitué par de la chaux et des cendres volantes. Je peux assurer qu'il n'en est rien car j'ai stabilisé de nombreuses centaines de mètres de voies en utilisant comme coulis des mélanges de ciment Portland et de bentonite. Cela n'a rien d'étonnant car le Portland contenant beaucoup de chaux et les cendres volantes étant pouzzolaniques, les deux coulis sont analogues. Ce sont les conditions économiques qui fixent le choix des constituants. Il faut toutefois se rappeler que la granulométrie des cendres dépend de la nature du charbon qui a été brûlé. Il est donc nécessaire de faire quelques essais préliminaires pour formuler correctement le coulis contenant des cendres.

4 - Enfin avec ce même coulis ciment-bentonite j'ai stabilisé un glissement de talus dont l'argile très gonflante formait un marécage pendant la saison des pluies alors qu'elle était sillonnée de fissures dont l'ouverture atteignait 2 à 3 m à la fin de la saison sèche. C'est à ce moment qu'on a rempli les fissures avec le coulis, afin d'interdire la pénétration des eaux de pluie. Peut-être la chaux du ciment a-t-elle eu un effet bénéfique. Toutefois ces coulis non protégés à la surface du sol se sont fissurés par dessication et il a été nécessaire de procéder à une petite injection d'entretien cinq ans plus tard. Mais une telle dessication n'est pas possible sous une voie ferrée.

D.A. Greenwood (Written discussion)

JET GROUTING - CONTROL OF UNIFORMITY OF TREATED SOIL

Jet grouting was used in 1962 for temporary works on the Concorde aircraft engine facility and at a sewage pumping station in Cornwall, U.K., and as reported by Nicholson (1963) for a water cut-off at Niazbeg Power Station in Pakistan amongst other applications.

Problems were encountered which dissuaded engineers from using the process more widely at that time:

1. Control of average cement content.
2. Control of uniformity of cement in situ.
3. Control of uniformity of column diameter.

Difficulties arose because having drilled the pilot hole to full depth the volume of fluid needed to jet away the soil exceeded the volume of the bore and overflow at surface was inevitable. This carried away cement included in the jet stream. The overflow material was collected and returned to the hole with addition of more cement appropriate to the enlarging hole. With bulking of the eroded soil not all of the added cement and spoil was returned to the hole on completion. The process was critically dependent on rate of erosion in relation to rate of lifting and rate of cement addition. In ground of variable erodibility these difficulties were greatly increased.

The result was somewhat unpredictable and variable strength. Sometimes weaknesses occurred across a specific

horizon or stratum where less favourable ground was encountered leaving a weak band in a column.

Similarly where wide variation of erodibility occurred, e.g. sands enclosing strata of cohesive clay, the column diameter tended to vary significantly. This could be overcome by spending more time in the resistant bands but at cost of aggravating imbalance of the rate of cement addition to rate of lift making control difficult. Very precise knowledge of soil profile detail was needed for this and was not usually available with sufficient accuracy.

The current proponents of jet grouting extol its virtues. The incorporation of air jetting will have improved erosion but they would render a service by publishing much more quantitative data on the reliability of properties of treated soil as a structural material.

With the better understanding of geotechnics since the first use of jet grouting in U.K. twenty years ago engineers, correctly, have become more willing to employ comparatively weak stabilised soil as a temporary structural medium. In earlier days anything less than concrete was not readily accepted.

Nicholson, A.J. (1963). Symposium on "Grouts and Drilling Muds in Engineering Practice", I.C.E. London, pp 108-109 Discussion.

R. Pusch (Written discussion)

"A LEADING PAPER FOR PANEL DISCUSSION ON IMPROVEMENT OF COHESIVE SOILS" (Contribution by R.K. Katti)

It is nice to see that material science in terms of clay mineralogy and physical chemistry are referred to as basic sciences of the presently discussed soil mechanics application. In dealing with saturated clay the author

brings up the almost classical point of the surface charge and importance of electrical double layers, and refers to an expression of the swelling pressure which is unfortunately not understandable and also not correct.

The reason for the last statement is indirectly given by the author himself by pointing out the importance of adsorbed water which, at least at higher densities, determines the swelling pressure. In fact, the hydrogen bond-

ing of water molecules are the main origin of the "expansive forces". So, in principle, the author is right in bringing in basic science for the understanding of the behaviour of soils. But it should be in a proper way.

R. Vučetić and M. Djordjević (Written discussion)

STRAIGHTENING A LEANING TALL APARTMENT HOUSE IN SMEDEREVO BY APPLYING THE METHOD OF WEAKENING THE SOIL Redressement d'un Haut Immeuble de Résidence Penché en Smederevo par la Méthode d'Affaiblissement du Terrain

INTRODUCTION

Attempts to straighten tall buildings, chimneys, towers, etc. have so far been based on two main principles: to improve the soil under the part of the building where consolidation is greater or to transmit the load of that part of the building to greater depths, to less compressible media. In the first case it was done by injecting cement, by chemical and physical action, etc. and in the second case by pile driving. All these methods have their good and bad aspects, but they either could not be used in all cases or they were not successful enough. The Institute for Soil Engineering of the "Kosovoprojekt" enterprise from Belgrade has carried out a daring project of straightening a tall apartment building in Smederevo by applying the method of weakening the soil under that part of the foundation of the building which had previously been less deformed in order to provoke in the weakened zone greater deformation and to bring about the straightening of the leaning building.

APPLICATION

The building in Smederevo is 38.5 m tall with shallow foundation on the depth of 2.5 m and 3.5 m on a basement slab 21.2 x 22 m. The load of foundation on the soil is 190 kPa. The building is located between the river Danube and the Redut hill. The soil consists of clay (CL/CI) to the depth of 5.5 m, under which are muddy sands and mud to the depth over 20 m (the depth of the investigation of the soil). Before the beginning of the work on straightening, the building was leaning towards the Danube about 35 cm. While the consolidation of the building on the side towards the hill was rapid, the side towards the Danube was consolidating constantly in a moderate but unabated rhythm in the course of two years. The measures undertaken to straighten the building were the following. Along the side of the building (towards the hill) 11 slanting boreholes were bored. In that way the boreholes reached a stratum of muddy sand. Air under pressure was pumped through the boreholes and in that way material from the boreholes was taken out (air-lifting). The quantity of the extracted material was controlled in special containers. At the same time a geodetic recording of a net of built-in benchmarks on the building was being carried out, in order to follow its reaction. 5.4 m³ of dry

material was extracted. The project was carried out in November 1978 and lasted 10 days. Since then a constant geodetic survey of the benchmarks on the building is being carried out, and it shows that the building has been consolidating evenly, and that the weakening of the soil under the foundation has successfully solved the problem of stopping the leaning and of returning this high building to the desired slope.

