Rapport de l'animateur
Discussion leader's report

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La session technique 20 "Parois moulées" a été organisée autour de 3 thèmes principaux :

- Thème n° 1 : New developments for Diaphragm Wall filling materials
  Nouveaux développements dans le domaine des matériaux de remplissage des parois moulées
- Thème n° 2 : Outstanding utilizations of Diaphragm Walls
  Exemples d'utilisation marquante des parois moulées
- Thème n° 3 : Present state and new trends in Rotary Cutter - Diaphragm Wall Construction Technology
  Etat actuel et tendances nouvelles dans le domaine des Haveuses de paroi.

Un thème supplémentaire : - Excavating Fluids for Diaphragm Wall Construction, - Fluides de perforation pour parois moulées, est venu tout naturellement compléter le programme de la session auquel il faut rajouter deux interventions annexes également liées à des problèmes de construction des parois moulées, soit au total dix contributions orales.

THEME N° 1

Les travaux du Professeur YOSHIO OHNE (AICHI Institute of Technology - JAPON), intervenant invité, s'inscrivent dans le cadre plus général de la recherche et mise au point de matériaux de remplissage pour la réalisation de coupure d'étanchéité par paroi moulée. Toute inclusion rigide placée au sein d'un massif déformable peut engendrer des concentrations de contrainte. Le matériau de remplissage idéal pour une paroi d'étanchéité est donc celui qui présente, outre les caractéristiques de perméabilité et de pérennité physico-chimique satisfaisantes, des caractéristiques mécaniques voisines de celles du terrain de fondation.

Ce matériau doit donc avoir des modules d'élasticité instantanés et différés de l'ordre de ceux du terrain meuble encaissant et un potentiel de déformation sans rupture de l'ordre de plusieurs %.

Cette recherche d'un matériau présentant un comportement plastique avec ou sans écrouissage avait été faite principalement, en particulier par les spécialistes Européens, dans le sens de bétons de granulométrie fine continue dont la matrice est constituée d'un coulis binaire ou ternaire ciment, argile, bentonite.

L'originalité du Professeur YOSHIO OHNE et de son équipe est d'avoir recherché et utilisé un liant hydrocarboné pour la réalisation d'un béton asphaltique. Son mérite est d'avoir réussi à mettre au point la technologie de mise en place de ce béton sous boue bentonitique à une température largement supérieure (150° C environ) au point d'ébullition de l'eau du fluide d'excavation et d'avoir résolu le problème de l'exécution des joints entre panneaux.

La réalisation effective de plusieurs coupures étanches démontre que le procédé a atteint, du point de vue technique, le stade industriel. La question de l'expulsion dans le temps de certains composants hydrocarbonés mérite certainement d'être approfondie sous le double aspect du vieillissement du matériau et, dans le cas de réservoirs d'eau potable, des risques liés à la libération d'éventuels éléments nocifs.

L'utilisation d'un béton de fibres métalliques pour le bétonnage d'une paroi moulée circulaire de soutènement, présentée par Mr M. BUSTAMANTE (Laboratoire Central des Ponts & Chaussées - PARIS - FRANCE) constitue une extension au domaine de la paroi d'une technologie mise au point à l'origine pour la réalisation de dallages industriels.

Le béton de fibres avait déjà trouvé une application pour la réalisation des pieux à la tarière creuse travaillant essentiellement à la compression. Il apporte dans ce cas une solution alternative intéressante à la mise en place, toujours délicate sur une certaine hauteur, d'une armature traditionnelle destinée à satisfaire un taux de ferraillage réglementaire.

Dans le cas général des parois moulées de structure il ne peut être question, en l'état actuel de nos connaissances, d'envisager la suppression, même partielle, des aciers de renforcement par l'utilisation d'un béton de fibres. Par contre, et c'est là que les essais exposés par Mr BUSTAMANTE prennent tout leur intérêt, le béton de fibres, en association avec un ferraillage traditionnel, pourra peut-être permettre un comportement meilleur du béton dans sa masse, à la traction, et par suite une amélioration des conditions de fissuration.

Par ailleurs, l'application au cas des parois moulées circulaires de diamètre faible à moyen travaillant essentiellement en compression est particulièrement intéressante. Elle permet la suppression de tout ferraillage traditionnel tout en apportant un élément de sécurité appréciable en cas d'irrégularité géométrique ou de chargement.

THEME N° 2

La réalisation d'un ensemble de fondations lourdes, dont les descentes de charge unitaires peuvent atteindre 77 MN, au moyen d'éléments de paroi, présentée par le Professeur SCHLOSSER (Terrasol Geotechnical Consulting Engineer - PARIS, FRANCE), intervenant invité, rejoint également le thème n° 3, l'excavation des parois moulées ayant été réalisée avec des Haveuses de Paroi. Le terrain situé au-dessus du niveau de fondation, a nécessité un traitement par injection de façon à pallier les désordres provoqués par d'anciennes exploitations de carrière.

Ce traitement a permis, non seulement de limiter les incidences ultérieures sur l'ouvrage des décompressions constatées dans les terrains de couverture, mais de réaliser les travaux de fondation eux-mêmes sans risque de perte totale de la boue d'excavation en cours d'exécution.

On retrouve là un problème commun à l'excavation de toute paroi moulée quelle que soit la technologie utilisée mais qui prend une importance toute particulière dans le cas de l'utilisation des Haveuses de paroi en raison des volumes et débits de boue instantanés nécessaires, de la vitesse d'avancement et des grandes dimensions de la machine.

La paroi moulée d'étanchéité réalisée à plus de 60 m de profondeur maximale dans un terrain alluvionnaire constitué, sur un pourcentage important de la hauteur, par des empilements de blocs de rocher dur de toutes dimensions, s'inscrit parmi les quelques ouvrages les plus difficiles exécutés dans le monde à ce jour.

Cette coupure étanche présentée par Mr M. GANDAIS (Entreprise BACHY, Levallois-Perret, France) a été réalisée avec une technologie d'excavation du même type que celle adoptée pour la construction de la paroi moulée du barrage de NEW WADDELL - ARIZONA (U.S.A.) en particulier dans la falaise d'andésite de la rive gauche. Ces deux exemples récents démontrent la faisabilité de la paroi moulée dans des conditions extrêmes de difficulté.

On peut regretter que le projeteur ait opté, comme matériau de bétonnage de la paroi, pour un béton rigide dont la résistance à la compression simple interdisait la réalisation des joints entre les panneaux en venant remordre avec la benne d'excavation pour l'exécution des panneaux secondaires, sur le béton des panneaux primaires encadrant. Mais on peut également se demander si, compte tenu de l'hétérogénéité exceptionnelle du terrain de fondation, l'obligation d'utiliser la méthode classique du tube-joint n'a pas été finalement un élément positif pour la conduite de l'excavation permettant le guidage de la benne lors de l'exécution d'un panneau, dans le volume vide ainsi réservé en extrémité du panneau précédent.

THEME N° 3

Les contributions de Mr E. STOTZER (BAUER Spezialtiefbau - SCHROBENHAUSEN, FRG), intervenant invité, et de Mr M. GUILLAUD (Entreprise SOLETANCHE, NANTERRE, FRANCE) permettent de faire un point de l'état actuel de la technologie d'excavation des parois moulées au moyen de Haveuses à évacuation continue des déblais de perforation par circulation inverse, et des évolutions en cours de ces matériels.

A cette occasion, il est intéressant d'esquisser un bilan schématique de l'utilisation de ces machines conçues et développées dans le but d'étendre le domaine d'application des parois moulées tant du point de vue technique qu'économique.

Profondeur

- Possibilité de réaliser des parois très profondes dans des conditions économiques satisfaisantes en raison de l'évacuation en continu des déblais d'excavation.

- Nécessité de mesure précise de la position réelle de la machine au cours de l'excavation pour un contrôle continu et une correction éventuelle de la trajectoire.

Cette exigence est également fondamentale pour pouvoir réaliser les joints entre panneaux en venant remordre sur le béton du panneau précédent.

Si des progrès dans ce domaine des mesures ont été réalisés, il reste encore beaucoup à faire pour contrôler effectivement les déviations en rotation non détectables par les inclinomètres habituellement utilisés.

Nature des terrains

- Possibilité de pénétrer couramment dans des terrains de résistance mécanique plus élevée qu'avec les équipements classiques d'excavation à la benne.

Cette possibilité nécessite d'être précisée par les remarques suivantes :

- les valeurs de résistance à la compression simple des roches pouvant être excavées dans des conditions normales de coût et de délai, avec les outils de coupe commercialement utilisés à l'heure actuelle, sont plus voisines de 60 à 70 MPa que des 100 à 120 MPa annoncés parfois par les utilisateurs. Pour approcher ces dernières valeurs il faut se tourner vers les outils de coupe diamantés avec les incidences économiques correspondantes.

- la présence de blocs durs dans un terrain cohérent où la résistance de la matrice est insuffisante pour les maintenir solidement en place de façon à permettre leur destruction, ou dans un terrain alluvionnaire hétérogène, ne permet pas raisonnablement l'utilisation des machines de havage.

Les dispositifs originaux présentés par Mr STOTZER permettent cependant de broyer des galets approchant 20 cm.

- A l'autre extrémité de l'échelle des duretés, les terrains collants tels que les argiles molles constituent un matériau particulièrement rebelle à la technique du havage en raison de son aptitude à enrober de façon tenace les outils de coupe.

- L'utilisation des haveuses dans les terrains granulaires comportant une fraction importante dans le domaine des silts et des argiles, ou dans les terrains faiblement cohérents susceptibles de se déliter en cours du processus d'excavation, peut se trouver rapidement limitée par les difficultés rencontrées pour débarrasser la boue de forage de cette fraction fine en suspension.

Les installations usuelles de traitement de la boue ne permettent pas d'extraire ces ultra fins. Il faut pour cela utiliser des équipements beaucoup plus complexes de centrifugeage difficilement compatibles avec une exploitation courante et, en tout cas, grevant considérablement le fonctionnement. Par suite, plus ou moins rapidement l'augmentation de la teneur de la boue en éléments très fins conduit à rejeter cette dernière avec les problèmes correspondants d'évacuation (impact sur l'environnement) et de coût (fournitures, transport, mise en dépôt).

Evolution et développements en cours

On peut conclure des deux interventions sur la question des haveuses à une évolution et un développement des machines dans deux directions :

- une course au gigantisme (profondeur, épaisseur) provoquée par les projets considérables lancés au JAPON. Trois super machines se préparent pour se disputer un marché pour le moment géographiquement localisé. Il en résultera très certainement des avancées dans le domaine de l'automatisation et surtout du contrôle et de la correction des trajectoires mais on peut être dubitatif quant aux extensions des utilisations possibles de telles machines exceptionnelles.
- l'élargissement du domaine d'application des haveuses à des roches de résistance à la compression très au-delà des valeurs économiquement abordables à l'heure actuelle. L'adaptation aux machines de havage présentée par Mr STOTZER, des technologies utilisées pour les tunneliers montre que les 150 MPa sont d'ores et déjà une réalité qu'il est possible de prendre en compte dans les projets futurs.

Mais en face de ces perspectives presque futuristes la contribution de Mr GANDAIS nous ramène à des réalités plus terre à terre. Il n'aurait jamais été possible de réaliser la coupure étanche qu'il a présentée avec une machine de havage aussi puissante et élaborée soit-elle dans l'état actuel de nos connaissances.

Le matériel classique d'exécution des parois - bennes et trépans de poids et de forme diverses - reste pour le moment irremplaçable pour la réalisation de paroi dans des terrains alluvionnaires, éboulis de pente, etc . comportant des blocs de toutes dimensions.

L'excavation dans les terrains silteux, silto-argileux ou argileux avec un matériel benne permet de résoudre sans problème majeur les phénomènes d'enrichissement de la boue en éléments ultra fins.

Comme dans tout domaine il n'existe pas de solution unique permettant de couvrir tout l'éventail des cas qui peuvent se présenter.

Parois profondes, chantiers de paroi de grande importance, rocher (jusqu'à une certaine limite de dureté) constituent le domaine d'application naturel des machines de havage.

Chantiers d'importance moyenne, terrains hétérogènes à blocs, argiles molles, terrains silto-argileux, restent encore pour le moment l'apanage des techniques traditionnelles d'exécution de paroi.

THEME SUPPLEMENTAIRE

La contribution du Dr L. MARTAK (Stadt WIEN - AUSTRIA) vient tout naturellement compléter les contributions des thèmes 2 et 3. Qu'il s'agisse de boue dynamique ou de boue statique, les caractéristiques des boues bentonitiques tant du point de vue rhéologique que de leur comportement dans l'excavation elle-même au cours du forage, ne sont pas de natures différentes.

La très courte intervention de Mr GASHMAN (U.K.) a le mérite de rappeler le développement actuel parallèlement à la boue bentonitique très généralement utilisée jusqu'à ce jour pour l'excavation des parois, de fluides de perforation à base de polymères, beaucoup plus courants, pour le moment, en boue dynamique qu'en boue statique.

Le développement des machines de havage a conduit le spécialiste à examiner de plus près l'application à l'exécution des parois moulées d'un fluide de perforation cantonné jusqu'alors à l'industrie du forage. L'évolution à la baisse du prix des polymères a largement contribué à ce réexamen.

L'utilisation des boues à base de polymères en remplacement des boues bentonitiques est prometteuse mais est loin de constituer pour le moment une solution universellement applicable. Chaque cas doit être abordé comme un cas particulier en fonction de la granulométrie du terrain, sa teneur en éléments fins, sa perméabilité, etc .

Mais l'expérience acquise sur un certain nombre de chantiers est prometteuse en particulier du point de vue économique.

CONTRIBUTIONS DIVERSES

L'intervention de Mr D. COUMOULOS (CASTOR - ATHENS - GREECE) vient conforter et compléter les observations faites sur nombre de chantiers quant à la qualité des joints de paroi obtenus dans le cas des parois plastiques en venant remordre avec la benne sur le matériau du panneau précédemment exécuté. On peut espérer que la quantification de ces observations par les essais de perméabilité réalisés contribuera à conforter les projeteurs sur la validité d'une telle solution.

Le dispositif provisoire de soutènement présenté par le Professeur MATOS FERNANDES (PORTO UNIVERSITY - PORTUGAL) est un excellent exemple de l'ingéniosité de l'ingénieur confronté à une situation inattendue. Il ne peut que contribuer à renforcer la confiance du projeteur dans la paroi moulée comme dispositif de soutènement pour les fouilles profondes.

The new diaphragm walling technique
Une nouvelle technique de paroi moulée

ERWIN STOETZER, Bauer Spezialtiefbau GmbH, Schrobenhausen, Germany

SYNOPSIS: For many years the excavation of diaphragm walls by the conventional grab method has been common-place but in recent years the employment of cutters has increased at a remarkable rate. The reason for this change of emphasis is clearly related to the demand for greater accuracy and more thorough quality control during the excavation of diaphragm walls. The need to achieve greater depths in increasingly complicated soil conditions including hard rock without excessive vibration has also without doubt influenced the trend in the development of the cutter.

1. INTRODUCTION

The construction of diaphragm and cut-off walls by grabs is nowadays a commonly used technique. During the last decade, however, the limitations of the established grab technique have become more and more evident, as underground construction has reached increasing depth and more difficult ground conditions have been encountered:

The result of this is as follows:

- low output due to intermittent excavation between upward and downward travel of the grab

- vibrations and their transmissions to adjacent structures as a result of chiselling in hard soil layers

- inconsistent verticality, particulary during chiselling, in hard formations.

These problems can be resolved by the trench cutter technique, which has been developed during the last few years.

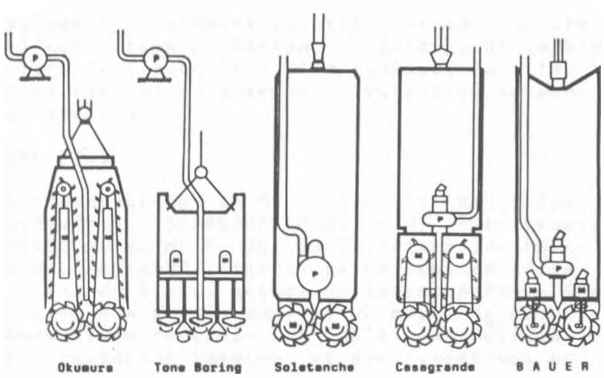


Fig. 1 Comparison Cutters

2. DEVELOPMENT OF THE CUTTER TECHNOLOGY (fig.1)

During the early 1960's two Japanese companies, Tone Boring and Okumura developed techniques, which allowed soil to be eroded and broken up into smaller particles, and after having been mixed with bentonite slurry to be pumped to the surface. This equipment was able to penetrate relatively loose soils to a depth in excess of 50 metres.

Tone Boring developed cutting blades rotating around a vertical axis in opposite directions for operational stability. Okumura used two cutting wheels rotating around horizontal axis and which were driven by a chain. Both systems removed the mud by way of a suction pump located outside the trench or by airlift.

This concept of wheels rotating around a horizontal axis was further advanced some years later, by the introduction of more powerful and simpler hydraulic motors in preference to electric ones. As this system was designed for much higher power output the first cutter project in rock was successfully completed in Paris 1975 by the French company Soletanche.

The arrival of the cutter technology in Europe prompted other European companies to greater activity during the 1980's. In 1982 the company Casagrande of Italy developed a trench cutter with a chain drive, similar to that of Okumura of Japan. Sone advances in the cutter technology were achieved in 1984 with the development of a rock cutter by company Bauer of West Germany. The cutter wheels are driven by vertically mounted high speed hydraulic motors. A special gear system converts the high speed rotation of the motors into slower rotation with a transmission ratio of approximately 1:100. With this system, the torques can be increased to standard designs.

3. THE CUTTER TECHNIQUE

3.1 Frame and cutter wheels (Fig. 2)

The Slurry Trench Cutter consists essentially of a heavy steel frame with two hydraulic drives each mounted at the base of the frame. Cutter wheels, specially designed for various soil types are mounted on the drives.

The rotating cutting wheels erode and break up the soil in a continuous process, mix the soil fragments with Bentonite slurry and move it towards the suction intake. A high capacity mud pump transports the soil Bentonite mixture to the desander. (Fig. 3)

The production of the trench cutter is largely dependent on: (Fig. 4)

- the crowd force exerted as a result of the weight of the cutter
  and
- the torque of the cutting wheels.

For a high torque/crowd force ratio the cutting teeth are rapidly eroded without achieving any cutting process. For a low ratio, the teeth are prone to digging into the soil and causing the wheels to stall.

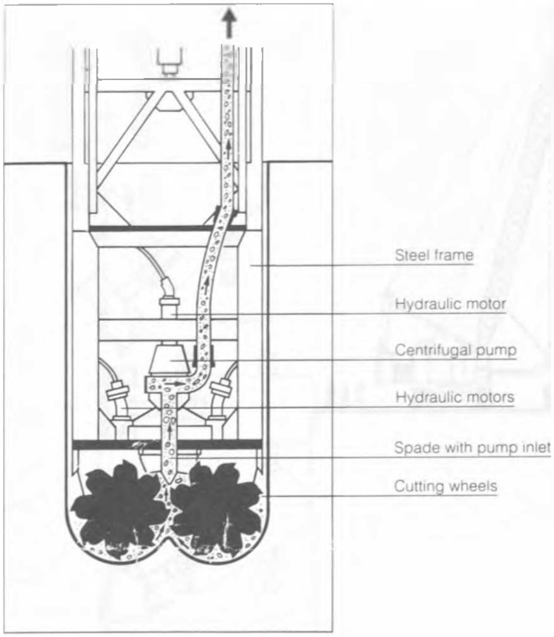


Fig. 2 Slurry-Trench-Cutter material transport

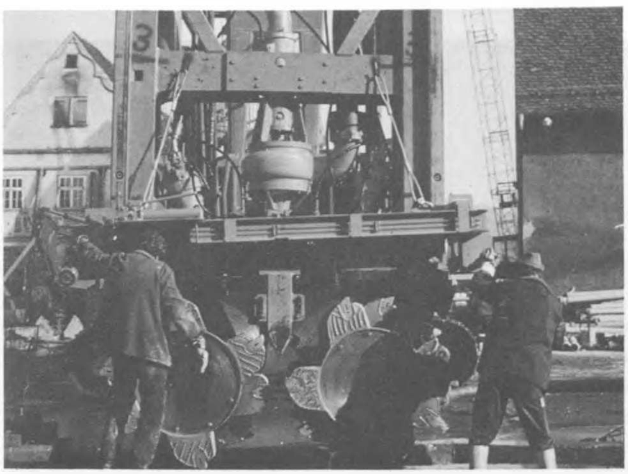


Fig. 3 Cutter wheels and mud pump

Maximum production is achieved by optimizing the torque-crowd force ratio through a carefully controlled operation.

The slurry charged with eroded material is generally pumped through hoses with diameters ranging from 5 to 6 inch. To avoid blockages, maximum soil particles must not exceed half of the hose diameter. Stones or rock fragments in excess of 2 to 3 inch must be crushed by the dynamic forces of the cutting wheels. In order to produce these necessary forces, the rotational speed of the cutting wheels can be increased. (Fig. 5)

Elastic shock absorbers, built into the cutter

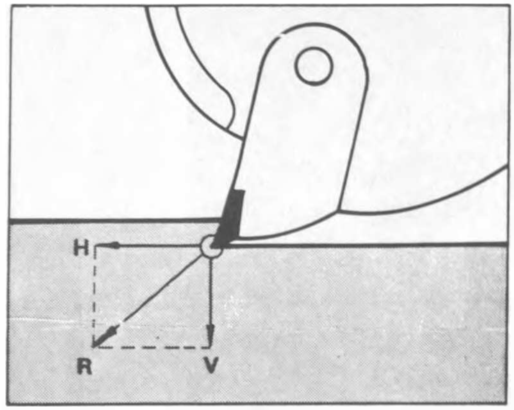


Fig. 4 Force components of cutting wheels

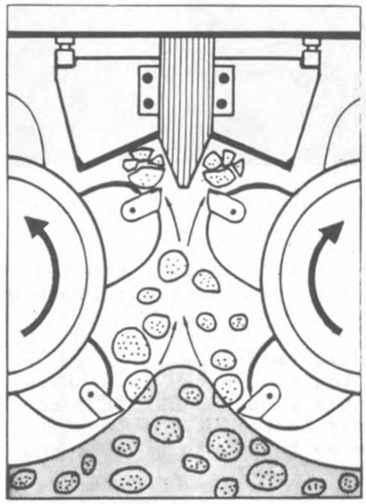


Fig. 5 Material transport to pump inlet

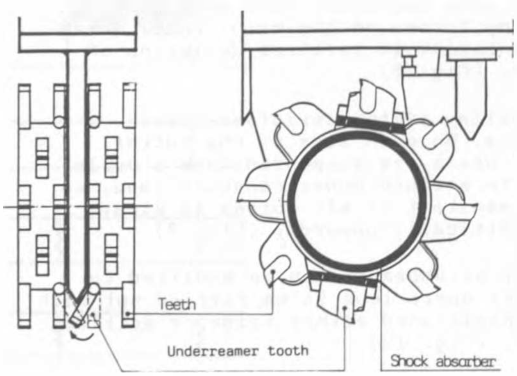


Fig. 6 Cutting wheel with underreamer tooth and shock absorber

drums protect the hydraulic drives against shock damage. (Fig. 6)

A further problem in the cutter technology is

Fig. 7 Underreamer tooth

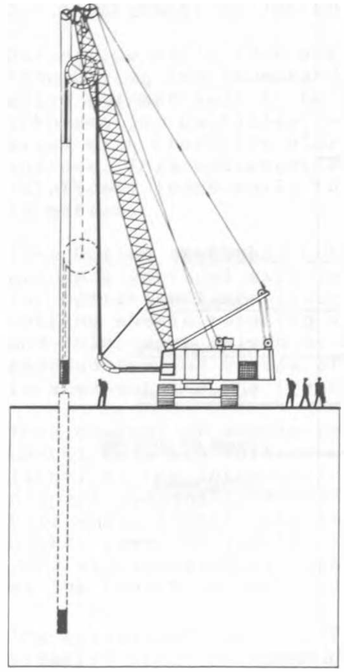


Fig. 8 Cable and hose tensioning device

the ridge of soil or rock left between the two cutting wheels. Initially this had to be crushed by the crowd pressure.

With the development over recent years of an automatic underreamer tooth, company Bauer have been able to resolve this problem. During each rotation the underreamer tooth folds automatically outwards and removes the ridge of soil and rock. (Fig. 7)

3.2 Hose and cable pulley

During cutting, both hydraulic and suction hoses must be able to travel continuously with the cutter into the trench and must not transmit any forces on the main frame, which then could result in vertical deviation of the cutter. (Fig. 8)

The hose pulley system satisfies these requirements. On each side of the cutter, bundles of hoses are suspended from a pulley connected to a winch under constant load, so that the resultant of all forces is always directed vertically upwards. (Fig. 9)

This pulley arrangement can be modified to allow cutter operations to be carried out with the crane positioned either aside or astride the trench. (Fig. 10)

3.3 Operating and control system

As a continuous process of excavation, the trench cutter technique is suited for continuous automatic data recording. The following data can be recorded simultaneouly and transformed into operational commands: (Fig. 11)

Fig. 9 Hose tensioning device for cutting depth up to 100 m

- cutter depth and rate of penetration
- rotational speed and torque of wheels
- crowd pressure on cutter teeth
- volume of slurry flow
- verticality in both directions

3.4 Environmental characteristics

The slurry trench cutter is completely free of noise and vibrations. Chiselling is not required and the excavation of a trench is possible in close proximity to sensitive adjacent structures. In many soil types the spoil can be completely separated from the slurry, so that the

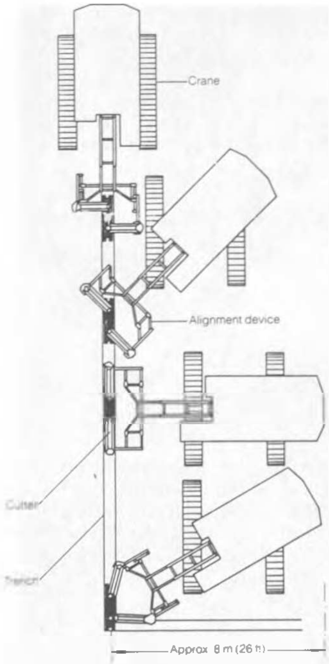


Fig. 10 Alignment for congested sites

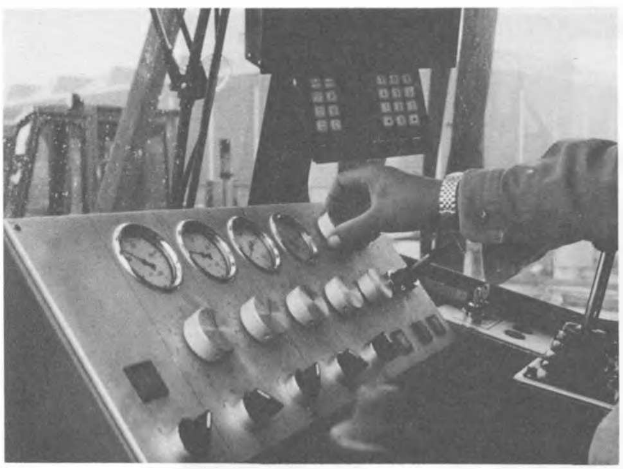


Fig. 11 Operating Panel

dumping of contaminated spoil at special tips is no longer required. (Fig. 12)

3.5 Joints

The cutter technique does not require the use of tubular steel or precast stop ends. During the excavation of the secondary panel each end of the primary panel is overcut by up to 200 mm. (Fig. 13)

Fig. 12 Congested site condition

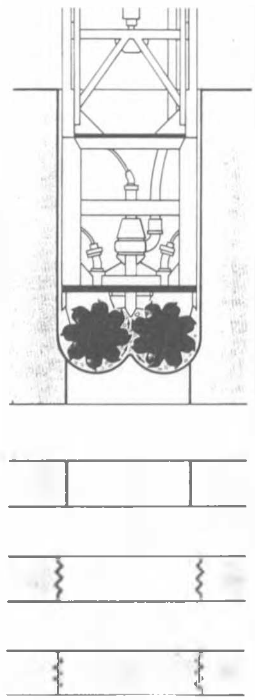


Fig. 13 Overcutting joints

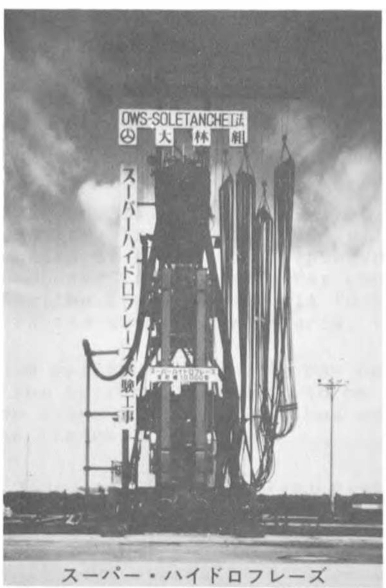
4. NEW DEVELOPMENTS

4.1 Deep walls

Conventional diaphragm walls are in general not more than 1.5 m thick and 80 m deep. There are, however, new developments in Japan of almost gigantic dimensions where our Japanese colleagues are planning to construct, for example, for a ventilation shaft of the Tokyo Bay crossing project, a circular diaphragm wall of 100 metres diameter, 135 metres depth with a wall thickness of 2.8 m.

System Tone Boring

Fig. 14



System Obayashi - Soletanche

Fig. 15

Again Tone Boring of Japan have constructed the first machine and have carried out initial tests for these dimensions. (Fig. 14)

The development of similar Giant Cutters is under way of other companies like the joint effort of Soletanche of France together with Obayashi of Japan. (Fig. 15)

This technology is certainly of future interest for large underground storage facilities, caissons for bridge foundations, anchor blocks and so on.

4.2 Hard rock cutter

Special cases, like cut-off walls in dams require sometimes the penetration into hard rock.

The use of teeth even reinforced by Tungsten Carbide plates has proven inefficient and therefore expensive if rock strength exceeds about 500 bar. Working towards resolution of this problem a joint effort by Bachy of France and Bauer of Germany has produced the hard rock cutter. (Fig. 16)

The basic concept consists of replacing teeth by roller bits located around the cutting wheels. Recent full scale tests in granite and very hard limestone have demonstrated interesting production rates in rock with unconfined compressive strength over 1800 bar. (Fig. 17)

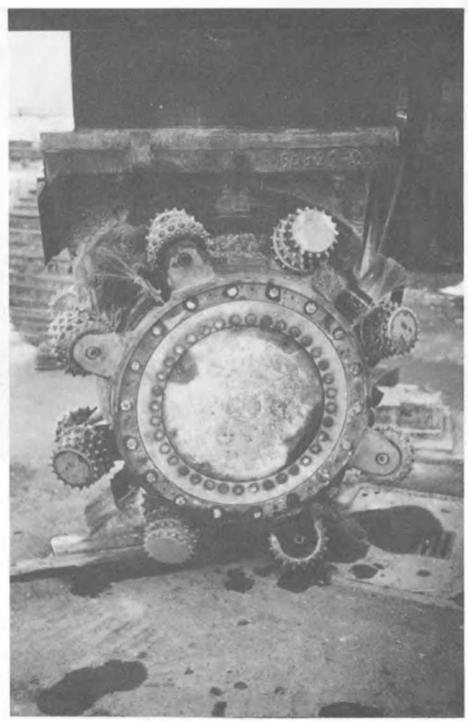


Fig. 16 HR-Cutter

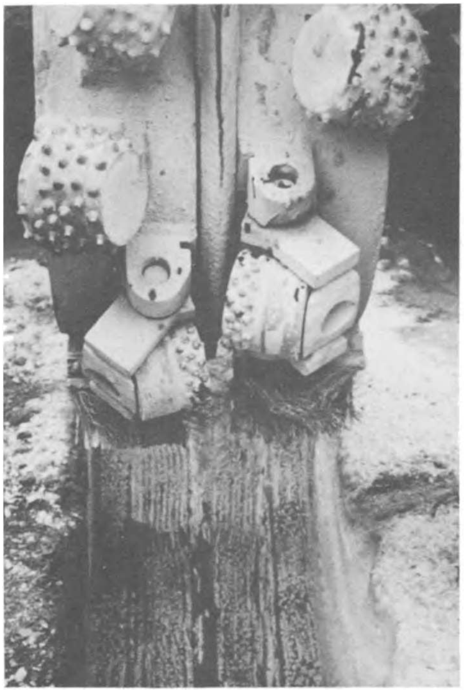


Fig. 17 HR-Cutter
excavating granite

The Pehuenche Dam cut off diaphragm wall (Chile)
Le voile étanche en forme de paroi moulée du Barrage de Pehuenche

M.GANDAIS, BACHY, Levallois-Perret, France

This short contribution concerns the technical choice of the excavation method that has to be made to construct a diaphragm wall in the foundation of the Pehuenche dam to act as a watertight cut off. The work is nearly completed now in soil conditions which are among the most difficult that can be found in the world for constructing a diaphragm wall.

Location

The site is located in Chile and is part of the Rio Maule hydraulic scheme which already includes the Colbun and Machicura dams. Upstream of these dams, the Pehuenche dam is under construction on the rio Melado which is a tributary of the rio Maule. The dam will be 85 m high above its foundations when it is completed. Watertightness is ensured by a clay core and, in the foundations, by the concrete diaphragm wall cut off (Fig. 1).

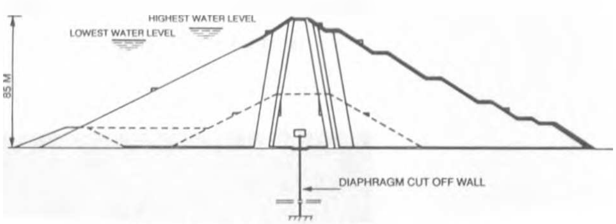


Fig. 1 Cross section of dam

Geotechnical features

The dam is located in a valley with very steep sides and the rock is composed of andesites and combinations of basalts and andesites. These natures of rocks are characterized by an extremely high compressive strength mostly measured in the range of 150 MPa to 200 MPa with some values even higher.

The rock valley is filled with a very thick glacio-fluvial deposit made of a combination of sand-gravel-cobbles (Fig. 2 & 2) and huge boulders which can be several meters in diameter (Fig. 4).

The diaphragm wall is excavated in this deposit which is very compact, made of materials of the same nature of rock as the sides of the valley and which has a thickness over 60 m at the deepest point.

Fig. 2 Typical ground aspect

Fig. 3 Typical cobble extracted from the trench

Fig. 4 Type of boulder encountered

General specifications

The general specifications to take into account for the construction of the diaphragm wall can be summarized as follows :

. minimum thickness of 60 cm of concrete over the whole area of the wall,

. maximum deviation of 15 cm over the total depth of panels,

. socketing in bed rock : minimum of 60 cm measured perpendicularly to the bed rock surface,

. systematical checking of the bedrock profile by core drilling at the bottom of each excavated panel to ascertain that the diaphragm wall would not be locally resting on top of a large boulder or on an overhanging part of the sides of the valley to prevent pervious areas from remaining underneath.

Excavation methods

Several excavation techniques can be implemented today to construct a continuous watertight cut off and it is the skill of the specialist to select the best appropriate method to the ground conditions to be encountered. The more difficult the ground conditions are the more essential the choice is for the feasability and the economy of the project.

The techniques available today are :

- boring of secant piles,
- excavation with cable operated clam shell grabs,
- excavation with hydraulically operated clam shell grabs,
- excavation with rock mills, eventually equiped with rock crusher,
- excavation with the very hard rock cutter which we develop at the moment with the Bauer Company of Germany.

Selection of the method

Our selection was the results of an elimination approach :

- secant piles were rejected for several reasons :
  - . large diameter needed to ensure the required thickness of 60 cm of concrete,
  - . multiplicity of joints which are obvious weak points,
  - . difficulty to excavate in cobbles and boulders at great depth without deviation.
- hydraulically operated grabs are very poorly effective in cobbles,
- rock mills were rejected for two main reasons :
  - . teeth of the normal tools do not bite in rock as hard as 150 MPa to 200 MPa in compressive strength,
  - . cobbles are too large to be crushed by the stone crusher.
- the very Hard Rock Cutter was not considered primarily because it was not fully operational at the moment. On the technical aspect, the efficiency of the VHR in a ground that contains very hard cobbles which can be loosened at the bottom of the excavation still has to be proved.

Paradoxically for our time, our choice has been the conventional method which consists in excavating with Bachy KL 1000 heavy self guided cable operated grabs and with very heavy chisels (11 tons) (Fig. 5 & 6). We perfected the method by applying localized blasting. To take into account possible cumulative deviations a thickness of 1 meter was chosen.

Fig. 5 Guide for blast holes drilling

Fig. 6 Cable grab and heavy chisel

Final quantities

The work will be successfully achieved very soon and the final quantities will be close to the following figures.

- surface area of the wall : 5 100 m²
- thickness : 1 m
- maximum depth of panel : 64.8 m
- length of individual panels : 4.4 to 6 meter (see disposition on Fig. 7).

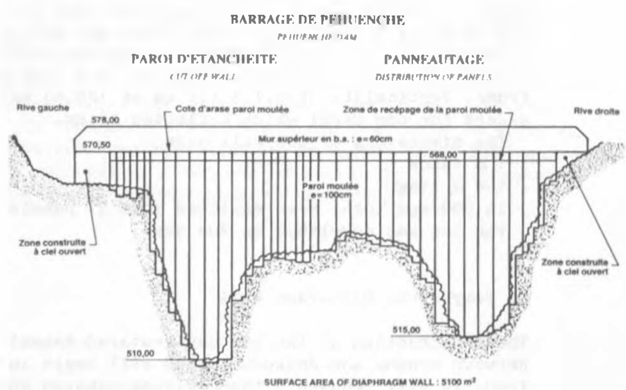
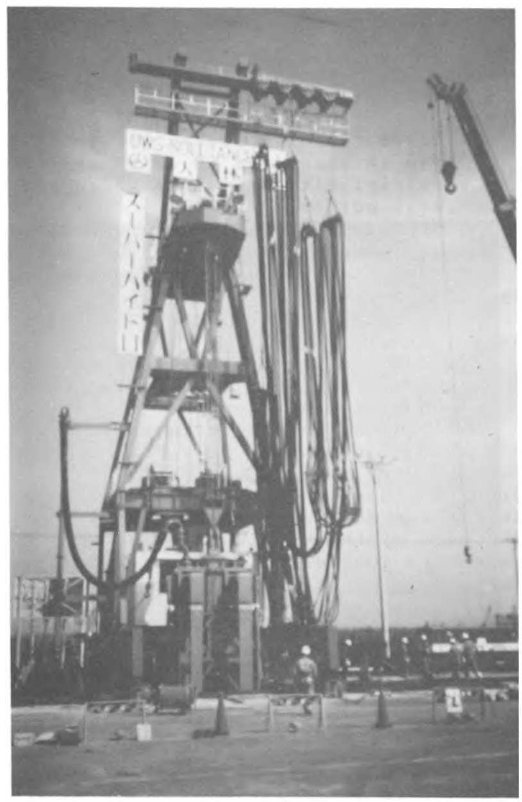


Fig. 7 Longitudinal profile of the diaphragm wall

Discussion

MAURICE GUILLAUD, Solétanche, Nanterre, France



The present contribution describes two recent achievements of very deep diaphragm walls, both carried out by using the Hydrofraise System.

Recently, The British "International Construction" Magazine quoted the 120 m deep diaphragm wall in 1987 at Navajo Dam (New Mexico, USA) as the deepest ever. Actually the world record seems to stand at 151 m, reached by a test section diaphragm wall at Tokyo in November 1987. This short paper presents the outlines of those two projects.

1. Navajo Dam

This dam, a 115 m high zoned earth embankment was completed in 1962. From the very start of impounding, there was more leakage than expected on both abutments. In 1986 the Bureau of Reclamation specified remedial works including a positive cut-off diaphragm wall on the left abutment down to the sandstone substratum at a maximum depth of 400 ft (120 m). The diaphragm wall was constructed by using a Hydrofraise type diaphragm wall rig.

The machine used the reverse circulation mill-cutter system. It was fitted with a special 30 m long guiding frame. Verticality of 0.1 % (12 cm at 120 m) was achieved except for one panel which deflected 15 cm.

The dimensions of the wall were :

. 1 m thick
. 140 m long
. 11,900 sqm total area splitted into 33 panels.

The job was completed in 204 days.

2. Tokyo Test Diaphragm Wall

The construction of the new cable-stayed Akashi Bridge between Honshu and Shikoku Island will begin in March 1990. It will require a huge cylinder-shaped anchor block approximately 82 m diameter and maximum 75 m deep. This cylinder will be a 2.20 m thick diaphragm wall.

Also to be started in 1990, the Trans-Tokyo Bay Crossing will need an artificial island to be built, the core of which being an approximately 150 m deep, 110 mm diameter cylindrical diaphragm wall.

To be qualified for these projects, the Contractors have to make a full scale test proving their capability to perform such deep diaphragm walls. Ohbayashi passed successfully the test by constructing five, 2.40 m thick, 3 m long panels, one of them at 151 m depth, using a Super-Hydrofraise machine of the same type as for Navajo, but mounted on a 150 ton-frame. The verticality was monitored by a series of inclinometers and all drilling information was plotted on a computer chart. The verticality achieved was more or less 0.3 % (more or less 5 cm at 151 m).

An ingenious structural solution to support a deep excavation

Une solution de structure ingénieuse pour stabiliser les parois d'une fouille profonde

M. MATOS FERNANDES, Faculdade de Engenharia, Universidade do Porto, Portugal

This contribution describes a deep excavation for seven basements of a new building carried out in Lisbon in 1982 in which an original and ingenious method of support has been used.

Figure 1 shows a cross section of the excavation, 27 to 29 m deep, 40 m wide and 40 m long. Very stiff tertiary formations are reached at the site under about 8 m of weak soils. As shown in the figure, the initial design established the use of an anchored diaphragm wall, for the upper portion of the cut, and nailing with shotcrete in the zone corresponding to stiffer soil.

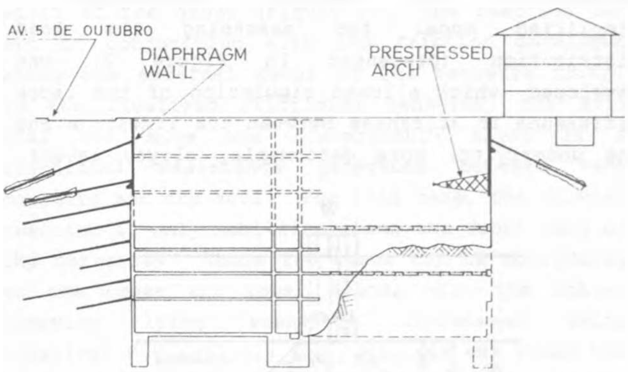


Figure 1. Cross section of the excavation

After the construction of the first anchor level at the face close to the building shown in the right side of Figure 1, its owner prohibited further installation of any anchors or nails inside his property. Several solutions to overcome this unexpected problem were then discussed. The adopted one was conceived by Professor Edgar Cardoso, an expert on large bridges design, and developed by the staff of the contractor (TEIXEIRA DUARTE, S.A.).

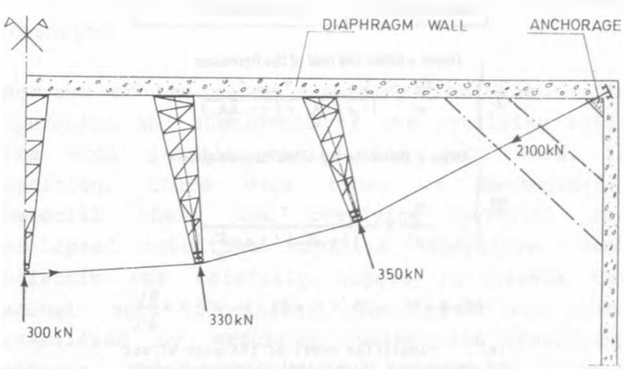


Figure 2. Plan scheme of the support system

The solution is illustrated in Figures 2 and 3 and consists of a polygonal tendon of 14 high strength steel strands prestressed to 2100 kN, coupled to a system of five struts applying to the wall forces ranging from 300 to 350 kN. The tendon is anchored at the two extremities of the cut face with the help of a steel plate previously embbeded in the diaphragm wall. The prestress of the system is applied by operating hydraulic jacks installed on the struts.

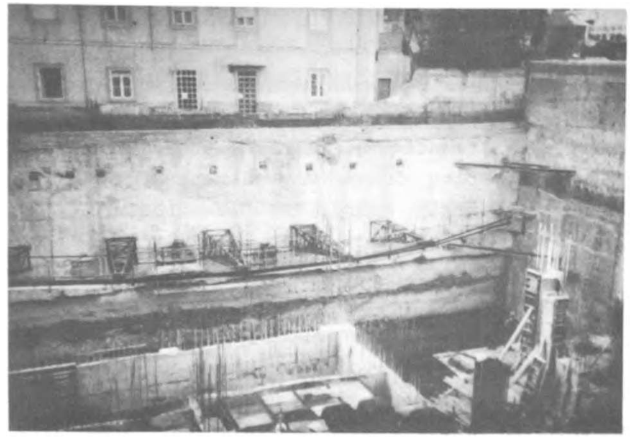


Figure 3. A view of the support system

As shown in Figure 1, in the subsequent stages, the excavation first progressed in depth at the left side, allowing for the construction of the foundations and the basements structure which was then used to support conventional struts for completion of the excavation.

To conclude, one should state that this interesting solution does represent an excellent contribution from an outstanding structural designer to a field usually dealt with by geotechnical engineers.

Invited discussion
Discussion d'un conférencier invité

F.SCHLOSSER, Terrasol, Puteaux, France

The extension of the Montparnasse railway station, in Paris during 1988-1989, to service high speed trains (TGV Atlantique), required the construction of high capacity foundation elements to support the particularly rigid super-structure, as discussed by Schlosser et al. (1989). The work includes the construction of a cover slab over the existing platform and rails to provide space for a parking facility, garden and park, and to provide support for three buildings. Because of the space limitations imposed by the numerous columns required to support the new platform area, the anticipated high support loads, and by the preference of the architect, the structure was comprised of a system of arcatures parallel to rails, and columns upon which rested large transversal pre-stressed beams.

The foundations for the platform are embedded in a layer of coarse-grained limestone, located at a depth of approximately 22 m below the platform surface, with an average depth in the vicinity of the park and garden area of 12 m. A cross-section of the typical geology for the garden and park areas is shown in Figure 1. The upper 1.5 m of the limestone layer had been quarried for building stone, with some of the cavities being backfilled with the residue after the mining had ceased. Propagation of the voids formed during mining have led to some local collapse of the gravelly marl located above the limestone.

The columns in the park and garden areas are founded on the limestone and supported by a series of barrettes. Each column is supported by between 3 and 6 barrettes, each with the dimensions of 1 m by 2.65 - 3 m, and an on-center spacing ranging from 1.8 to 2 m. It was necessary to pre-treat, by grouting, the mined areas of the limestone and the decompressed layers located in the marl beds prior to construction of the barrettes.

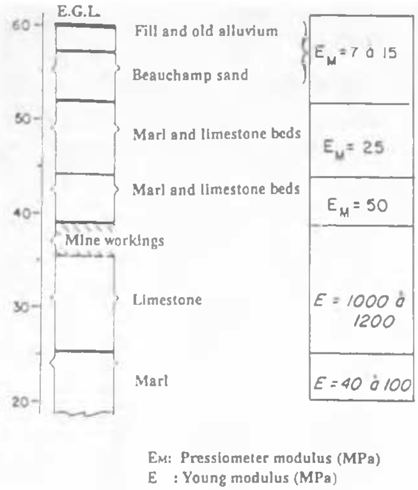
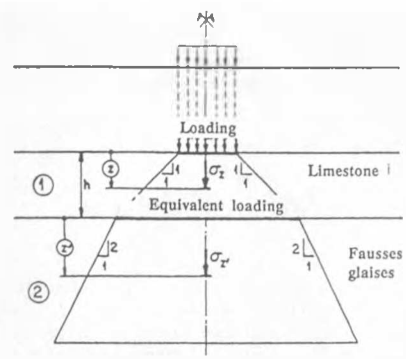


Fig. 1 : Typical cross-section of the Parks and Gardens area

To evaluate the stiffness of a barrette group, it was first necessary to devise a method for determining the vertical deformations. A simplified model for assessing the load distribution (presented in Figure 2) was developed, which allowed simulation of the large difference in stiffness between the limestone and the underlying, more deformable, clayey layers.



Loading	Equivalent loading
Magnitude p	Magnitude p'
Dimension B x L	Dimension B' x L'

① Depth z below the roof of the limestone

$$\frac{\sigma_z}{p} = \frac{1}{\left(1 + 2\frac{z}{L}\right)\left(1 + \frac{2z}{B}\right)}$$

② Deph z' below the top of the "fausses glaises"

$$\frac{\sigma_{z'}}{p'} = \frac{1}{\left(1 + \frac{z'}{L'}\right)\left(1 + \frac{z'}{B'}\right)}$$

$$B' = B + 2h \quad - \quad L' = L + 2h \quad - \quad p' = p \times \frac{BL}{B'L'}$$

Fig. 2 : Simplified model of the slab effect for assessment of vertical stresses beneath the loaded area

This approach resulted in a 40% decrease in the settlements computed at the center of a barrette group, with respect to that obtained by applying the Boussinesq theory, and showed a good correlation to the vertical stresses calculated for a uniformly loaded circular footing with an equivalent area, placed on top of two superimposed strata. By considering the lateral friction mobilized along the barrette, the calculated settlements were reduced by more than 50%, with respect to the simplified model.

Because of the complex soil-structure interaction, it was necessary to determine the equivalent stiffness coefficients of the barrette groups. To evaluate these stiffness coefficients, it was assumed that the actual behavior was located between perfect monolithic behavior and idealized frictional behavior. For the perfect monolithic behavior, the soil moves with the barrettes (no relative displacement) and the frontal passive resistance can develop along the entire equivalent width of the group (Figure 3). The reaction can act in conjunction with the friction developed along the external faces of the barrette group. In the idealized frictional behavior, the soil does not move and consequently there is a frictional resistance generated between each barrette and the soil. For this case, the frontal reaction is only mobilized along the front face of the barrettes. These two cases can be considered as the upper and lower bounds with the actual behavior lying somewhere in-between. Using classical P-y analysis for piles, it was found the equivalent stiffness for the horizontal force/ displacement (K_h) ranged from 0.8-1.2 times the likely value for a load applied along the small direction of the barrettes and from 0.8-1.5 times the likely value when the load is applied in the long dimension. The coefficient for the moment/rotation (K_m) was found to range from 0.9-1.3 when the load is applied in the long dimension.

Because of the voids resulting from the mining operation and subsidence of the overlying soil, the soil profile varied for each zone. In addition, there were zones of decompressed material where the overlying material had collapsed into the cavities. Therefore, each borehole was carefully logged to assess the actual soil conditions. The area was then stabilized by grouting, using the following methods. The voids left by the mining process

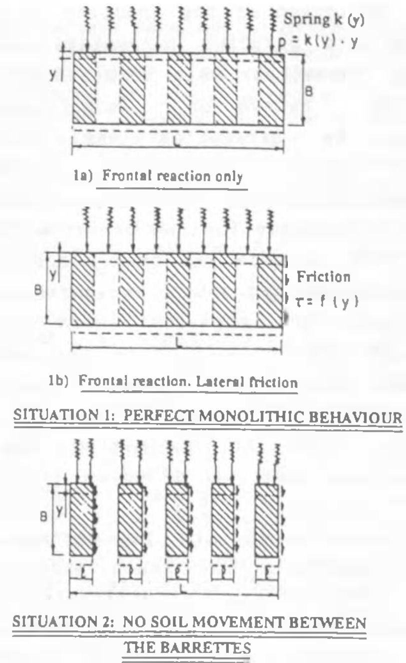


Fig. 3 : Behaviour under lateral horizontal loads

were backfilled by low pressure mortar grouting. Additional grouting was performed in the mined area to fill the remaining voids resulting from shrinkage of the original mortar. The areas in the limestone surrounding the foundations were also grouted to fill the fissures and small voids. Finally, the decompressed zones caused by subsidence of the marl was also grouted to improve its engineering characteristics. The layout of the grouting pattern (both primary and secondary) is summarized in Figure 4. The primary grout was placed under gravity pressure while the grout for the secondary boreholes was injected under low pressure.

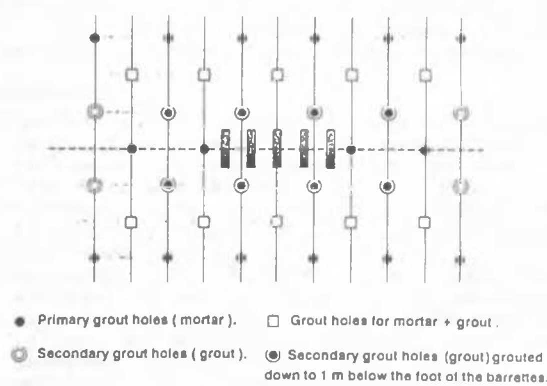


Fig. 4 : Layout of grout holes around a group of barrettes

After the completion of the grouting process and analysis of the grouting parameters, the deep foundations (barrettes) were constructed using a process of excavation, substitution and replacement. The excavation was formed in conjunction with a bentonite slurry. New bentonite was substituted for the original slurry and the reinforcements for the barrettes were put in place. The concrete was then placed using a tremic pipe and leveled off to the correct height. The boreholes were excavated with a BC 30 hydromill and the inclination of the excavation was checked continuously during the cutting process, using inclinometers. Special consideration was given to minimizing the urban enviromental problems, such as noise.

The reconstruction of the Montparnasse train station represents a remarkable example of a successful solution to a difficult problem, incorporating numerous tasks and difficulties, such as the diversion of existing railway tracks, strict task completion dates, and a complex network of overlapping tasks. In addition, the problems were further complicated by the uncertain nature of the soil characteristics in the vicinity of the job site. Remarkably, the station was able to function normally throughout the entire project.

References

Schlosser,F.,Simon,B.&Morey,J.(1989).High capacity barrettes in a region of old underground quarries.Proc. Int. Conf. on Piling and Deep Foundations, London, 119-130, Balkema, Rotterdam.

Discussion

E.T.STEPKOWSKA, Institute of Hydroengineering PAS, Gdańsk, Poland

Study in microstructure of bentonite suspensions was performed at King's College London in cooperation with Dr S.A. Jefferis, inspired by the late prof. J.K.L.Nash. It indicated that the microstructure depends on initial concentration of the slurry, on stirring energy, stirring time and on storing time.

Brebent and Berkbent bentonite suspensions were prepared at various concentrations (4% to 10%), various stirring energies (2000 RPM, 5000 RPM and 7000 RPM), various stirring times (5 and 15 min). They were stored for various times ranging between 0 and 8 months (Stępkowska and Jefferis, 1982, 1983, 1988, see paper 7/24).

Results of these investigations may be summarized as follows:

(1) Stirring at a high energy causes the delamination or even desintegration of smectite particles, thus an increase in specific surface, which may sorb an increased amount of water and result in an increased viscosity of the suspension.

(2) The influence of stirring time is similar to that of stirring energy. This may be jointly described as the influence of stirring action, where "action" is defined as energy multiplied by time. Thus wheter the stirring energy or the stirring time is increased by a given factor, the result should be the same.

(3) At an increased concentration of the bentonite suspension the disturbance of bentonite particles may be easier: there is a higher probability of interparticle contacts, which at stirring may lead to stripping of separate smectitic unit layers which form a particle, or even to stripping of separate layers (tetrahedral silica layer or octahedral gibbsite or brucite layer) from the unit layer itself. Thus an increased concentration has qualitatively a similar result as an increased stirring action.

(4) The influence of storing time is complex: initially the particle delamination proceeds, i.e. a decrease in particle thickness. This process is reversible and an increase in particle thickness starts to follow after ca. two months. This is connected with an important change of aggregation state of the suspension: clay particles in parallel arrangement form clusters, these form aggregates, which are aggregated further, forming a more or less regular "aggregate lattice" (in analogy to crystal lattice) with vacancies or macropores filled with water and/or water vapour and in unsaturated state filled with air. It was found in scanning electron microscope (SEM) study that the regularity of shape of these aggregates increases with the storing time of the suspension. The aggregation process started in unstored suspensions but the shape of the aggregates was not regular or their regularity was poor. After several month storage there were found some aggregates of an almost regular cubic form, separated by plane parallel fissures. The width of these fissures was the greater, the bigger the aggregates were and in some places it was evident that those aggregates were composed of smaller ones. It is interesting to note, that aggregation reached the macroscopic level: for microstructural studies the suspensions were dried in glass cylinders, $\emptyset$=5cm, h=5cm, either at room temperature or at 45°C.

Unstored and dried specimens formed irregular and relatively thin compact flakes, or they desintegrated into irregular pieces. After several month storage the suspensions indicated a uniform drying deformation, initially to a conical shape. The dried residuals of stored suspensions had a bigger volume and a much more regular shape than the unstored ones. Some specimens were ideally cylindrical, some desintegrated into two or three equal parts, some formed a tilted cylinder. It is interesting to note that the macroscopic shape of the dried residuals reflected the form of microscopic aggregates, present in the given suspension. This is again an analogy with the shape of crystals, which reflects their internal crystal structure (compare the cubic shape of the crystals of sodium chloride).

Finally it should be mentioned that bentonite suspensions separate into three microstructural phases: parallel, cluster and floc structure. Also phase separation occured if bentonite particles were desintegrated, during stirring, into smaller units (silica, alumina etc.). After the suitable storing time and/or eventual heating these phases could transform into new minerals: silica into quartz or cristoballite, parallel structure into paragonite, cluster structure into feldspar and floc structure into zeolites.

These new mineral phases were observed in SEM and identified by XRD and eventual chemical composition determination.

Some of the problems discussed above were presented in more detail in Proceedings of this Conference (Paper 7/24 and Discussion). Other problems will be presented elsewhere as the summary of this study.

It should be mentioned in conclusion that all the processes described above may be easily understood if equilibrium of interactions between structural elements is considered, i.e. van der Waals attraction and diffuse layer repulsion. Also contact bonds may be of importance. To fulfill equilibrium condition of balance of forces, the system is subject to change presented above.

REFERENCES

Stępkowska, E.T., Jefferis, S.A. (1982) Various types of microstructure in smectite and their influence on drying behaviour, 9th Conf. on Clay Min. and Petrol., Zvolen, CSRR

Stępkowska, E.T., Jefferis, S.A. (1983) Study in microstructures of clay slurries, Arch. Hydrotechniki, 30, 2, p. 193-211.

Stępkowska, E.T., Jefferis, S.A. (1988) The possibility of zeolite formation in bentonite slurries during storage and drying. Natural Zeolities, Ed. D.Kallo, H.S.Sherry, Akad. Kiado, Budapest, p. 149-159.

Stępkowska, E.T., Perez-Rodriguez, J.L., Justo, Sanchez-Soto, P.J., Jefferis, S.A. (1988) Possibility of feldspar formation in bentonite suspensions during storage, drying and/or heating. Proc. 9th ICTA, Jerusalem, P.C, p. 319-334.

Discussion

D.G.COUMOULOS, Consulting Engineer & Partner, Castor Ltd, Athens, Greece

A number of vertical and inclined boreholes were rotary drilled along the center line of the cement-bentonite (CB) diaphragm wall of a large hydroelectric dam in Greece.
These boreholes were drilled for the purpose of (a) checking the homogeneity of the wall,(b) measuring the permeability of the joints between panels by means of packer permeability tests, and (c) carrying out laboratory tests on cores of the hardened cement-bentonite mixture.

BRIEF DESCRIPTION OF THE CB CUT-OFF WALL

The cement-bentonite (CB) diaphragm cut-off wall has a width of 0.8 m and extends from ground surface to a depth of about 25 meters.
The construction of the wall is done in a series of alternating primary (P) and secondary (S) panels which are excavated using a cement-bentonite slurry. At the end of the excavation this slurry is left in the trench to harden. Once a pair of primary panels are sufficiently hard the intervening secondary panel is excavated. At the same time a small portion of each of the secondary panels, is excavated to ensure a good connection between panels.
The composition of the CB mixture that was used for the contruction of the diaphragm wall was as follows:

Cement Type I

Water-cement ratio w/c = 2.6

Bentônite 16% by weight of cement

Retarder 3‰ by weight of cement

BASIC GROUT MATERIALS

The basic grout materials used had the following properties:

Water: River water	
pH at 20°C	8.4
Cement: Portland Type I	
Blaine fineness, ASTM C 204	3300 cm^2/gr
Time of set, ASTM C 191	140 min
Bentonite (Sodium activated):	
Liquid limit, ASTM D 423	500
Plastic limit, ASTM D 424	40
pH of slurry with water content close to liquid limit	9.8

RESULTS OF FIELD INVESTIGATIONS

In general, pure cement-bentonite hardened material was drilled which yielded core recovery varying between 80 and 100 percent. Sand and sub-angular to subrounded gravel mixed with cement-bentonite material was found at various depths in about half of the boreholes.
The results of the field permeability tests in both the vertical and inclined boreholes showed that the CB diaphragm wall can be considered to be impermeable. In general, the presence of sand and gravel in the cement-bentonite mixture did not affect the permeability of the wall. The highest value of the coefficient of permeability that was observed at two locations, was of the order of 10^{-7}m/sec.

RESULTS OF LABORATORY TESTS

The laboratory tests on cores of hardened cement-bentonite mixture from the boreholes, yielded the following results:

- High water content values varying between a minimum of 110 and a maximum of 220 percent.
- Low dry unit weight values varying between 0.38 and 0.65 t/m^3.
- Specific gravity of grains was found to vary between 2.48 and 2.62.

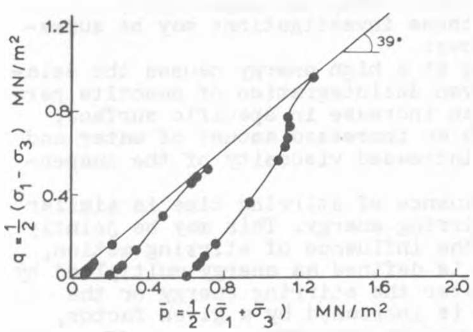


Figure 1. Typical stress paths from triaxial tests on cores from existing CB diaphragm wall

Figure 1 is a typical $\bar{p}$-q diagram of consolidated undrained triaxial tests on cores from the CB diaphragm wall. The angle α is equal to 39 degrees which means that the corresponding φ angle is equal to 54 degrees.
Finally, pinhole tests which were carried out according to the method described by Sherard et al (1976) and by Coumoulos (1976), showed that the hardened cement-bentonite mixture is not erodible.

REFERENCES

Sherard, J.L., Dunnigan, L.P., Decker, R.S., and E.F. Steel (1976). Pinhole Test for Identifying Dispersive Soils. Proceedings American Society for Civil Engkneers, Journal of the Geotechnical Engineering Division, Vol 102, No GT1, Jan 1976, pp 69-85.

Coumoulos, D.G. (1976). Experience with Studies of Clay Erodibility in Greece. ASTM Symposium on Dispersive Clays, Related Piping and Erosion in Geotechnical Projects, Chicago Ill., 27 June - 2 July 1976, ASTM Special Technical Publication 623, pp 42-57.

Invited discussion: Design and construction of asphalt mixture diaphragm

Discussion d'un conférencier invité: Dimensionnement et construction de voiles avec un mélange d'asphalte

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SYNOPSIS:It is a matter of great concern how to control seepage under the dams which are constructed on the comparatively pervious foundation. While concrete diaphragm parallel to the dam axis have often been adopted as one of the effective methods of foundation treatment, some of essential problems involved in the procedure have been pointed out. These are, for instance, differential settlement of foundation around the diaphragm, and loosening and/or cracking in surrounding impervious core which might be anticipated to occure during earthquake due to the difference in magnitude and vibratory mode of deformation between them, this will be caused by different rigid between foundation and diaphragm. In order to avoid such undesirable deformation, concrete diaphragms have ever constructed at the outside of upstream dam as shown in figure 1. This methods however were not only economical but also having technical difficulties on the construction at forming irregular abatments which contact with core zone. Under the circumstances some asphalt mixtures have recently been tried to use as a material of the diaphragm.
In this paper, some of discussion are made, on the design and construction procedures of asphalt mixture diaphragm which were adopted at three earth dams (YAMAMURA[1], SHORI[2] and MAIYAMA) built on the pervious foundations. In all these cases, a narrow trench was first excavated in the foundation and asphalt mixture was poured in it through tremie pipe, in which dry work was done at YAMAMURA dam by pumping out water in the trench, whereas asphalt mixture was poured in bentonite slurry in another two cases. It has no trouble experienced in dry work, but some troubles such as jetting of steam through tremie pipe and small steam explosion in the pipe took place during pouring. These shortcoming were, however, later overcome by detail examinations on the procedure, and a fully safe construction was attained[3,4].

DESIGN OF ASPHALT DIAPHRAGM

1) Flexibility of Diaphragm

It has been known well that the existence of structures which have different rigid in the dam body or foundation may change their vibratory charactristics of entire body of the fill during earthquake. In order to invastigate local and overall behaviors of such structures due to vibration, shaking table tests were performed on model embankments with a diaphragm of different flexibilities.

Tests were conducted on several model fill with different moduli of elasticity of core(Ec) and diaphragm (Ew), flexibility of earthfill being kept constant, where the ratio(Ew/Ec) was varied in five steps as 10, 25, 30, 40 and 100.

Each model was shaked at frequencies from 2 Hz to 12 Hz, in six steps, by increasing table acceleration up to a maximum value of 200 gals.

Typical vibratory modes of fill are presented in figure 2(a) for the case Ew/Ec=40 and in figure 2(b) for the case Ew/Ec=25, at a representative table acceleration of α=200 gal and frequency of vibration of f=6 Hz. It is obvious in figure 2(a) that an abrupt change appears in lateral movement of the core around the top of the wall, whereas rather uniform vibratory mode in figure 2(b) shows substantially little influence of the wall on fill movement.

These test results have revealed that such undesirable movement around the wall as indicated in figure 2(a) may be caused mainly by the large difference in flexibility between the wall and the fill, even though the frequency of vibration is another influencial factor to be regarded.

As shaking table tests perfomed here are not thoroughly consistent with similitude requirements, it is supposed difficult to apply the above-mentioned test results directly in the estimation of actual behaviors of dams during earthquake. In view of the fact, however, that additional numerical examinations by FEM have yielded similar results as those presented by experiments, it might be reasonable to conclude that undesirable movement around the wall may not be so significant if the flexibility ratio Ew/Ec is below 25.

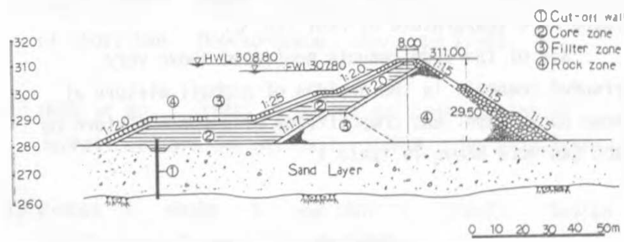
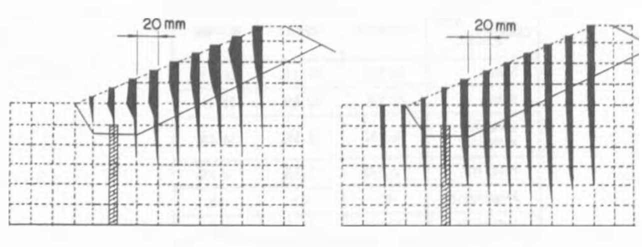


FIG-1 NISSHIN DAM



(a) $E_w/E_c \fallingdotseq 40$ (b) $E_w/E_c \fallingdotseq 25$

FIG.-2 Vibratory Mode of Defomation

Material flexibility of wall was so specified as to satisfy the requirement of $Ew/Ec \leq 25$ in the abovesated three earth dams.

2) Composition of Asphalt Mixtures

In addition to having a specified flexibility, asphalt mixtures for use as a material of wall should satisfy several other requirements. These are for impermeability to control seepage under the dam, stability against creep to plevent asphalt bitumen from being squeezed out by the action of the hydrostatic pressure, and adequate workability to obtain satisfactory flow through tremie pipe.

Flexibility of asphalt mixture change under a constant composition depending mainly on the applied confining pressure and its temperature.

Dynamic triaxial compression tests were carried out to determine the shear modulus of rigidity of the material at an expected lowest temperature of underground water of 12 ℃ and under a confining pressure of 200 KN/ ㎡.

Concerning stability against creep, a specification was settled so as to satisfy the requirement that squeezing of asphalt bitumen never come out under the action of water pressure twice as much as the maximum in-situ hydrostatic pressure, at an expected highest temprature of underground water of 16℃.

Workability of asphalt mixture changes depending on its fluidity, which is affected by the content of paraffin oil and the temperature when it is placed. It was decided in this respect that asphalt mixture of its slope flow value being more than 100cm/min, should be placed at a temperature of over 110 ℃.

All of the requirements mentioned above were arranged commonly in the deseighn of asphalt mixture at three earth dams, and compositions of asphalt mixture on each dam were shown in table 1.

Table-1

Composition of Asphalt Mixture

DAM NAME / Compo-sition	YAMAMURA	SOHRI	MAIYAMA
Sand	63.6%	61.0%	53.3%
Fiter	21.2%	20.0%	28.7%
Straight Asphalt	15.2%	15.5%	14.5%
Parafin	2.5%	3.5%	3.5%
Penetration	63	81	89
Softening	71 ℃	49 ℃	46.5℃

CONSTRUCTION OF ASPHALT MIXTURE DIAPHRAGM

1) Excavation of Trench

The trench was 43 cm wide and excavated to the required depth (about 10~30m), in which the formed section of a trench was filled with bentonite slurry to prevent failure of the vertical walls of granular soil.The excavation was proceeded at every other span of 2 ~5.8 m as shown in figure 3.

2) Underwater Placement of hot Asphalt Mixture

It was alrady anticipated that underwater placement of asphalt mixture heated over 100℃ woud be acconpanied by such undesirabl situations as boiling of water at the contact surface and cooling down and hardening of asphalt mixture which interferes pouring operation.

Laboratory and field test results revealed, however, that cooling of asphalt mixture takes place only at the contact surface with water and never reaches to the interior when the material is successively supplied. This fact confirmed feasibility of underwater placement of asphalt mixture with little objection except for some small boiling of water at the contact surface.

And another problem was to prevent leaking of water into tremie pipe. This was, however, solved by using oil-pressure type sliding value.

3) Construction joint of Diaphragm

Since trench excavation and of asphalt mixture was proceeded alternately by dividing the whole span of the wall in sections, joints were needed to contruct between adjacent asphalt strips. Among various types of joint usually adopted in the construction of concrete diaphragm, such as (1) interlocking type, (2) steel-plate partition type, and (3) precast concrete type, the interlocking type of joint was employed here by making

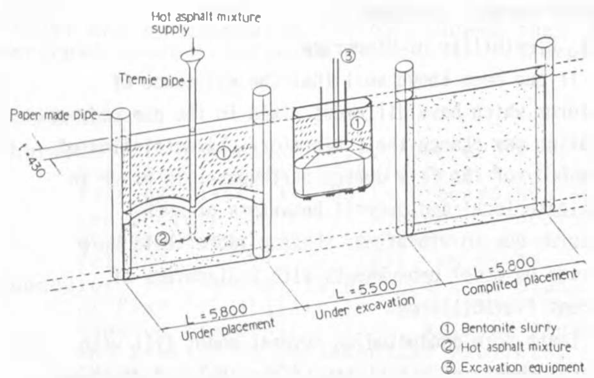


FIG-3 Excavation of Trench

use of paper-made pipes of 43 cm in diameter. Paper-made pipes thus used were for the purpoe of making trench excavation easier after hardening of preceding section of asphalt mixture.

4) Temperature of Asphalt Mixture

Field observations were performed in this project to study temperature change of asphalt mixture during and after placement. Figure 4 shows a temperature distribution in a trench when asphalt mixture is poured through tremie pipe. Temperrature in the interior of asphalt mass, about 10cm below the contact surface with water, indicated around 140 ℃, which is almost equal to the working temperature. In the upperpart of the mass, 4 ~5 cm below the surface, temperature changed repeatedly in the range from 120℃ to 130 ℃ ; this may be caused by thermal convetion within the mass.

Near the contact surface, about 5 cm above it, water also exhibited repeated change of temperature in the range from 60 ℃ to 70℃, whereas almost constant temperature of 20 ℃ was obserbed about 10cm above the surface. These rather uniform, not extremely disordered temperature distributions suggest that material separation due to boiling of water, which had been anticipated in the design stage, is unlikely.

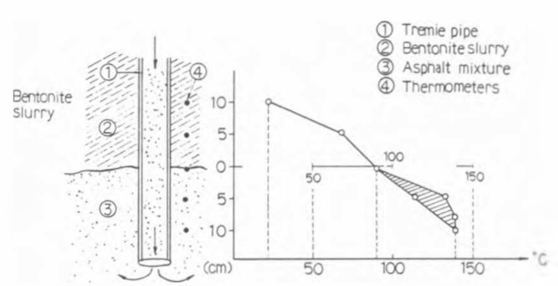


FIG.-4 Temperature Distribution in a Trench During Pouring

CONCLUSIONS

Concluding remarks drawn from the present study are summarrized in the following.

1) In the design of asphalt mixture diaphragm, compositions of asphalt mixtures were determined by taking into account the requirements for moderate flexibility to avoid undesirable deformation within an embankment, stability to obtain satisfactory flow through tremie pipe. Concerning material felixibility, an appripriate standard to obtain uniform fill movement could be set up through laboratory shaking table tests on model fills.

2) Among other things, underwater placement of hot asphalt mixture was a matter of great concren in the construction of asphalt mixture diaphragm. Discssions were made on the effects of boiling of water at the contact surface, cooling and hardening of asphalt mixture during pouring, and leak of water into tremie pipe.

3) To prevent steam jetting and explosions due to leakage, some improvements were proposed for tip-valve function of tremie pipe. The result was rather satisfactory and a fully safe procedure was attained in diaphragm construcion.

4) Field observation revealed that boiling of water and temparature change of the material did not affect pouring operation and thoroughly homogeneous asphalt mass was formed without any sign of material separation.

REFERENCE

1) Bureau of Enterprise of Mie Prefacture,(1975). Design and Planning Report on Yamamura Dam : MIE-Prefecture.

2) Bureau of Enterprise of Aichi Prefacture,(1982). Design and Planning Report on Shori Dam : AICHI-Prefecture.

3) OHNE et al. , (1984). Asphalt cut off Wall Construction of Shori Dam : Doboku-Sekou , Nov., pp.27-36.

4) OHNE et al. , (1988). Design and Construction of Asphalt mixture cut off wall : ICOLD, San Francisco.

5) KIMURA, K., OKADA , N., and OHNE, Y., (1962). Design and Planning Report of Yamamuram Dam : JSCE.