Communications tardives
Late papers

SASAKI H., NOTO S., OHTA M. et SATO S.

Hokkaido Development Bureau, Japon

Full scale experiment of a reinforced earth wall

Experimentations sur un mur en terre armée

This paper describes full scale experiment realized on a Reinforced Earth wall. The measurements were taken to determine the earth pressure on the facing, the displacements of the facing the tensile forces in the strips, and the friction between the fill material and the strips. Nearly all the experimental values of stresses in the earth and in the strips are lower than the design values, except some of them which may be due to foot heaving. Facing deformations are very little and soil-strip friction is higher than estimated.

I - Introduction

La méthode terre armée a été introduite au Japon il y a plusieurs années. Beaucoup d'ouvrages ont été réalisés sur la base de cette méthode dans la région de Honshu. Comme on le verra plus loin, elle a été adoptée à Hokkaido pour la première fois comme mesure temporaire pour les travaux d'amélioration de la Route Nationale 12, dans la région de Kamuikotan, ville d'Asahikawa.

Bien que les techniques de protection selon la présente méthode soient généralisées, rares sont les données constatées en ce qui concerne, par exemple, la grandeur et l'état de la répartition de la poussée du sol agissant sur la peau, la manière de déterminer le coefficient du frottement entre le sol et les armatures, la grandeur de la contrainte de traction s'exerçant sur les armatures et sa répartition dans le sens de sa longueur. Par conséquent, au stade actuel, il paraît primordial d'effectuer, à chaque exécution de travaux, un essai de recherches pour contrôler de façon satisfaisante l'exécution. Il convient également, sur la base des résultats obtenus, d'éclaircir les questions soulevées dans la présente procédure de plan et de tenter d'améliorer celle-ci dans la mesure du possible.

Le présent rapport a pour objet à la fois de vérifier si le mode de conception en vigueur comporte des problèmes et d'évoquer les matières qui pourraient exiger une attention particulière lors de la mise en oeuvre de cette conception, compte tenu de ce qu'implique la notion des problèmes déjà décrits, en se basant sur la comparaison entre les valeurs théoriques et celles réellement mesurées, suivant les résultats des essais obtenus à l'aide de divers instruments de mesure installés sur les peaux et les armatures et par d'autres moyens, sur le chantier en question.

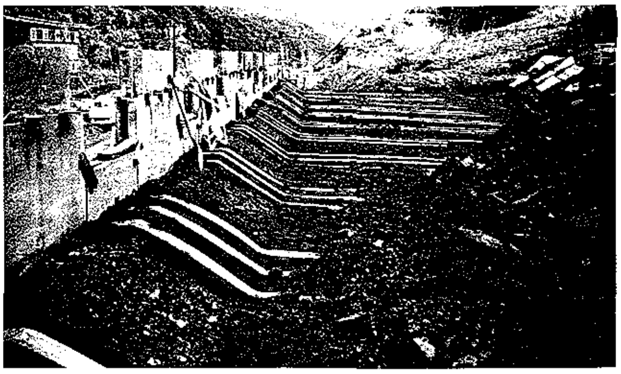


Photo 1 Pendant l'exécution des travaux

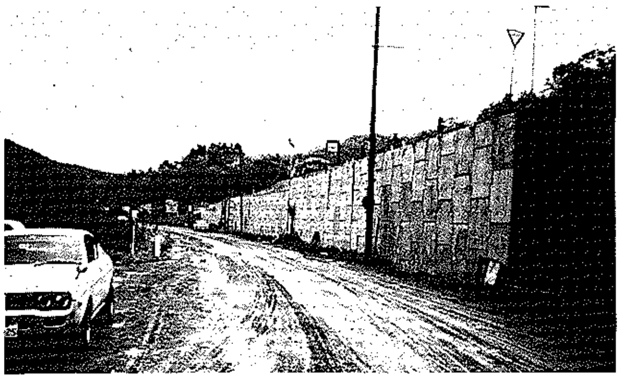


Photo 2 Lors de l'achèvement des travaux

Les photos 1 et 2 montrent l'état des travaux de construction, pendant leur exécution et à leur achèvement.

II - Sommaire des recherches

Les recherches ont été effectuées sur deux sections choisies dans le lot 1 (hauteur de mur : 5,25 m) et le lot 3 (hauteur de mur : 4,5 m) de la zone concernée.

Les sujets de recherches et leur contenu sont détaillés ci-après, la figure 1 montrant la façon de disposer les instruments de mesure.

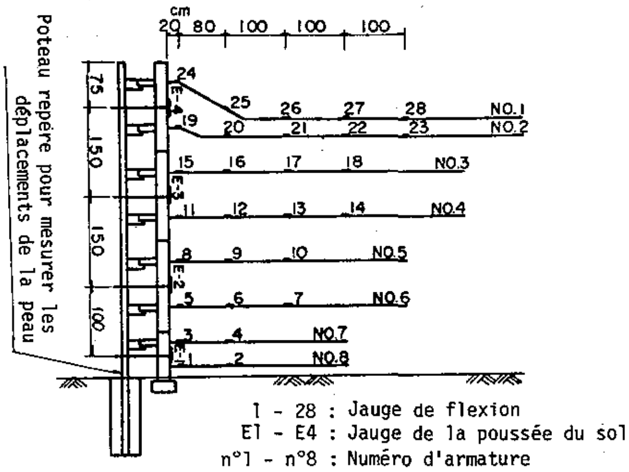


Figure 1 Pose des instruments de mesure (Lot 1)

(1) Poussée de sol horizontale agissant sur la peau

Les mesures ont été effectuées au moyen d'une jauge de poussée de sol attachée au centre du flanc d'un bloc de béton qu'on emploie comme peau. Et les valeurs ainsi mesurées ont été comparées aux valeurs théoriques de Coulomb en ce qui concerne la grandeur et la distribution de la poussée de sol agissant sur la peau.

(2) Déplacement horizontal ou vertical de la peau

Le déplacement de la peau a été observé au moyen des mesures de la distance comprise entre le poteau repère en acier, installé sur l'extérieur du mur et la peau.

(3) Contrainte d'arrachement provoquée dans les armatures

Pour mesurer la contrainte d'arrachement produite dans les armatures, on attache des jauges de déformation à intervalle déterminé sur les deux faces des bandes d'acier servant d'armatures. De plus, pour empêcher sa destruction par une machine roulante, ladite jauge est protégée par un élément protecteur en gomme dure.

(4) Coefficient de frottement entre le sol et les armatures

En vue de vérifier ce coefficient, on a préalablement enseveli une armature pour l'essai d'arrachement. Quand les travaux de remblai sont terminés, on la retire au moyen d'un vérin hydraulique.

III - Résultats des essais et observations

(1) Poussée de sol s'exerçant sur la peau

Par comparaison entre la charge de remblai et les résultats d'une mesure de la poussée de sol agissant sur la peau, au stade de travail de la terre armée, on obtient une dispersion comme l'indiquent les figures 2 et 3.

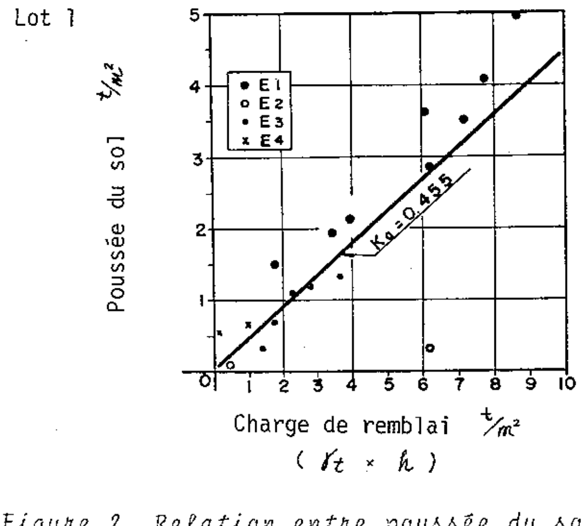


Figure 2 Relation entre poussée du sol et charge de remblai

Lot 3

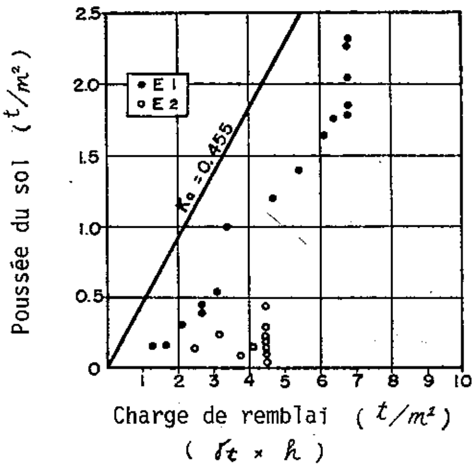


Figure 3 Relation entre la poussée du sol et la charge de remblai

Ces figures comprennent une ligne droite correspondant au coefficient de la poussée active de sol de Coulomb. Les valeurs mesurées sont proches des valeurs calculées (lot 1), mais légèrement inférieures pour le lot 3.

Dans les figures 4 et 5, les valeurs mesurées pour la poussée du sol immédiatement après achèvement du remblai, puis un peu plus tard, sont reliées les unes aux autres suivant la position de l'attache de la jauge de poussée de sol, de façon à présenter un réseau de répartition et à être comparées aux résultats de mesure des déplacements horizontal et vertical de la peau.

Du fait que la mesure devait être effectuée en se référant à l'état du remblai en cours étant donné le peu de commodité du chantier, il est inévitable que les valeurs mesurées pour les déplacements ci-dessus sont minimisées, et que la précision de ces mesures n'est pas absolument rigoureuse. Il est donc difficile de procéder à une observation quantitative. Cependant, étant donné que le déplacement horizontal a tendance à s'incliner en dehors en partant du bord inférieur de la peau, on peut présumer qu'une poussée du sol à l'état actif s'est produite au remblai arrière de la peau.

A l'examen de la répartition de la poussée du sol à l'achèvement du remblai et par la suite, on constate des valeurs mesurées très grandes dans la répartition de la poussée du sol, d'une part aux 31e, 61e et 84e jours après achèvement du remblai pour le lot 1, et d'autre part, aux 93e, 116e et 145e jours pour le lot 3. Ensuite, la poussée du sol tend à décroître avec le temps (figures 6 et 7).

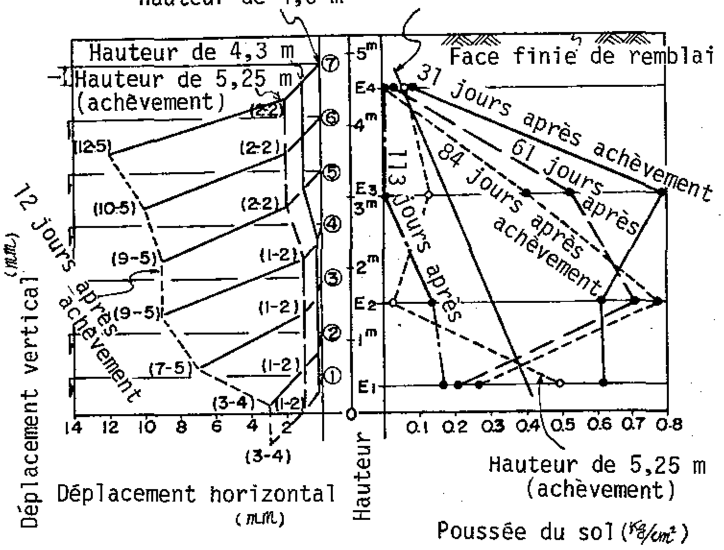
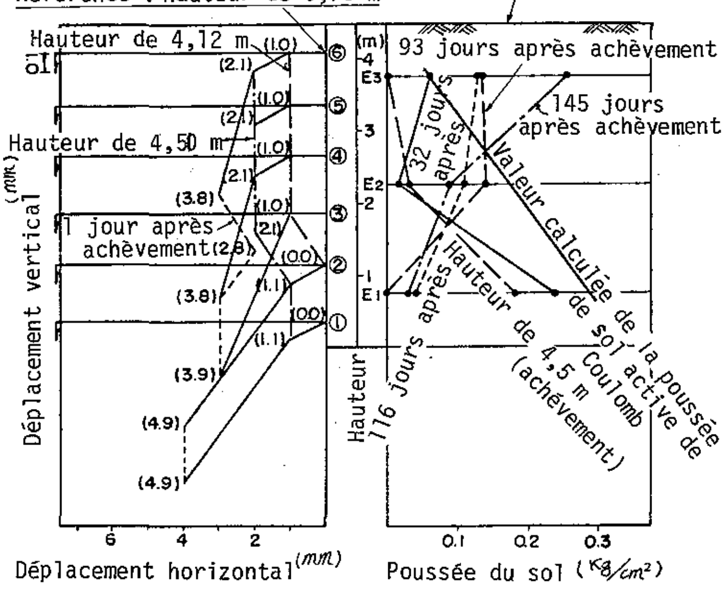


Figure 4 Déplacement de la peau, grandeur et répartition de la poussée du sol s'exerçant sur la peau (lot 1)



(Les chiffres entre parenthèses indiquent les montants de déplacements (mm) dans les axes X et Y.)

Figure 5 Déplacement de la peau, grandeur et répartition de la poussée du sol s'exerçant sur la peau (lot 3)

On ne peut connaître la cause réelle de ces importantes fluctuations dans la poussée du sol. Mais, à considérer qu'une poussée du sol fort importante a été constatée dans les deux lots, et ce toujours en hiver, on peut en déduire que s'y est ajoutée une pression provoquée par la congélation de la boue autour de la jauge de poussée du sol ou de la terre gelée.

La répartition de la poussée du sol à l'achèvement du remblai, présumée la plus proche de la répartition réelle, indique des valeurs inférieures aux valeurs calculées suivant la poussée de sol active de Coulomb, et prend une forme différente d'une répartition triangulaire.

Il se peut qu'une telle répartition ait lieu parce que les nombreuses armatures insérées dans le remblai empêchent l'activité du sol.

Dans ces conditions, appliquer la théorie de la poussée de sol active de Coulomb au projet d'un remblai de terre armée peut paraître inadéquat. Mais, comme actuellement il n'y a aucun procédé de plan de substitution, on ne peut qu'employer la théorie de Coulomb pour connaître la poussée de sol s'exerçant sur les armatures au niveau de la phase de projet.

Pourtant, dans les régions froides où la congélation est susceptible d'entraîner une expansion anormale et donc de provoquer une poussée du sol dépassant la valeur prévue, il faudrait étudier tout spécialement ce point au niveau du projet.

Les figures 4 et 5 montrant qu'avant l'achèvement du remblai, la face de peau se déplace un peu en direction horizontale et verticale. Mais la quantité en est jugée tout à fait insignifiante, ne comportant pas le moindre danger.

Par ailleurs, bien que les données d'observation du remblai après achèvement fassent défaut, on peut supposer qu'il n'y a presque pas de changement d'après la situation du chantier.

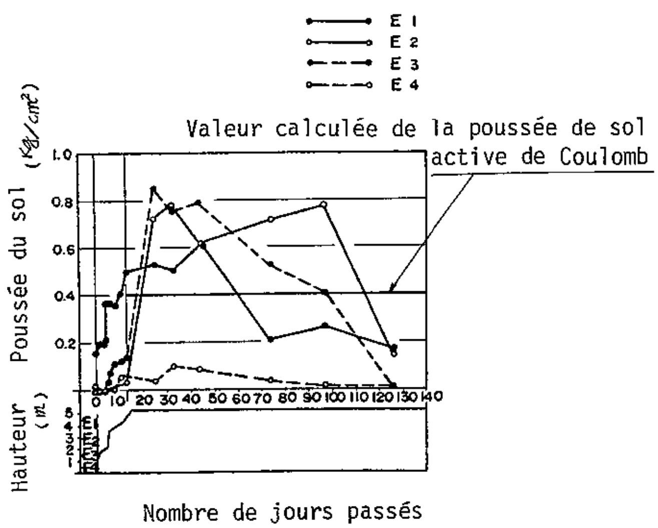


Figure 6 Changement horaire de la poussée du sol agissant sur la peau (lot 1)

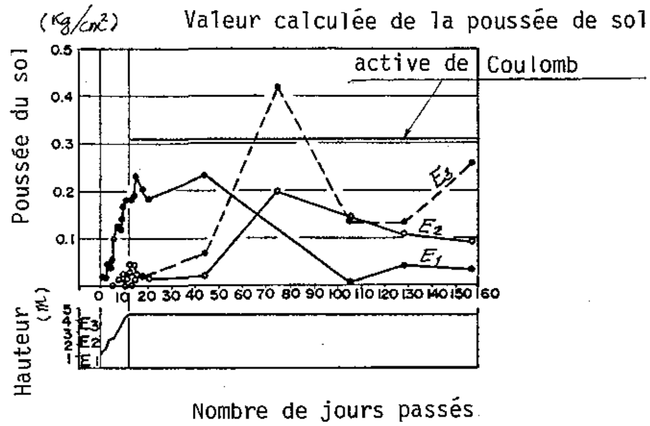


Figure 7 Changement horaire de la poussée du sol agissant sur la peau (lot 3)

(2) Contrainte de traction s'exerçant sur les armatures

Des mesures ont été prises continuellement pendant et après l'exécution du remblai. Les résultats apparaissent dans les figures 8 et 9. Ils montrent que la contrainte de traction produite dans les armatures pendant l'exécution du remblai augmente au fur et à mesure que la charge de remblai s'accumule, et qu'elle a encore tendance à s'accroître légèrement, après achèvement du remblai.

Cependant la quantité concernée étant extrêmement minime, on présume qu'elle converge à une certaine quantité déterminée lors d'une certaine "intimité" entre le sol de remblai et les armatures.

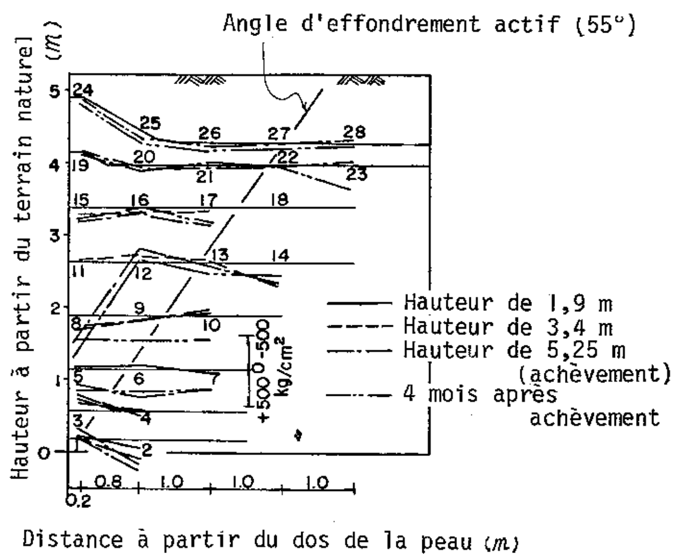


Figure 8 Résultats des mesures de la contrainte axiale (lot 1)

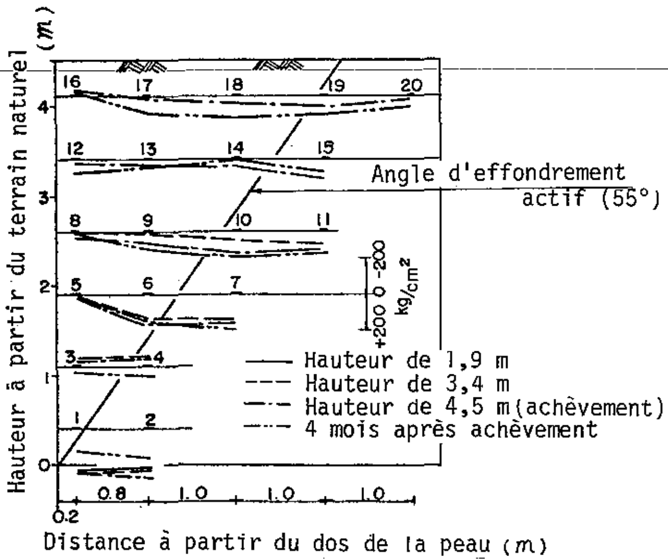


Figure 9 Résultats des mesures de la contrainte axiale (lot 3)

Le tableau 1 compare la valeur de projet à la valeur mesurée maximale de la contrainte de traction produite dans les armatures. Le tableau montre que, sauf un exemple du lot 1, les valeurs mesurées sont toujours inférieures aux valeurs prévues.

Cependant, dans le cas d'une jauge à fil résistant n°11 au contact et de la peau, d'une hauteur d'environ 3 m près du centre du remblai du lot 1, on constate lors de l'achèvement du remblai et également 4 mois plus tard, une contrainte de traction d'environ 1 300 kg/cm^2, valeur voisine de la contrainte admissible.

Comme nous l'avons déjà vu, la valeur mesurée de la poussée du sol de E_3, montrée dans la figure 4, est assez grande et les contraintes de traction relevées par les autres jauges appartenant à la même armature n'indiquent presque pas de différence. Ces constatations nous amènent à considérer que cela doit être imputé à une concentration de forces extérieures très fortes, dues à une cause inconnue jusqu'à présent. Dans l'ensemble, la répartition des contraintes de traction produites dans les armatures paraît décrire une courbe convexe à l'intérieur du remblai. Mais son maximum ne paraissant pas nécessairement coïncider avec la ligne d'effondrement actif (angle d'effondrement actif projeté θ = 55°), on est amené à supposer que cela est peut-être dû à une modification du plan d'effondrement actif, provoquée par le nombre élevé des armatures insérées dans le remblai, comme cela a déjà été décrit.

n° d'armature	Lot 1 (kg/cm)		Lot 3 (kg/cm)	
	Valeur projetée	Valeur mesurée	Valeur projetée	Valeur mesurée
n° 1	115.2	95	230.7	149.7
n° 2	297.6	244	471.1	107.2
n° 3	422.2	320.3	595.7	141.8
n° 4	546.8	(1,334)	720.3	279
n° 5	671.4	464.7	844.9	355
n° 6	795.9	549	645.6	331
n° 7	613.1	246.8	-	-
n° 8	613.1	309.7	-	-

Tableau 1 Contrainte axiale d'après le plan et la mesure

(3) Coefficient de frottement entre le sol et les armatures

Le tableau 2 montre les résultats des recherches du coefficient f de frottement entre le sol et les armatures.

Pour les présents travaux, la grandeur f a été prévue égale à 2/3 tan ϕ , mais les mesures ont montré que f est plutôt égal à tan ϕ , proche de 0,4.

Un facteur de sécurité Fa = 2,0 étant également prévu, l'angle de frottement intérieur ϕ = 22° de la matière de remblai qu'on emploie ici paraît tellement fiable qu'il permet de prévoir de façon presque certaine une valeur f = tan ϕ (pour le coefficient de frottement employé pour déterminer la longueur des armatures), basée sur les résultats d'essai.

Lot	Lot 1		Lot 3	
Valeur projetée	0.27			
Valeur d'essai (moyen)	0.35	0.43	0.36	0.34
	0.37			

Tableau 2 Coefficient de frottement entre le sol et les armatures

IV - Conclusion

Les résultats des mesures et les observations ci-dessus sont résumées de la manière suivante.

a) Pendant l'exécution du remblai, la poussée du sol agissant sur la peau est d'une valeur du même ordre ou inférieure à celle qui est calculée selon la théorie de poussée du sol de Coulomb. Mais, après achèvement du remblai, la production d'une forte poussée, probablement due au soulèvement de la terre gelée en hiver conduit à considérer toutes les influences d'un tel soulèvement lors de l'application de cette technique dans les régions neigeuses.

b) Les déplacements de la peau dans les directions horizontale et verticale sont extrêmement faibles. Par conséquent, on peut en déduire la sécurité des constructions bâties selon cette technique vis-à-vis des charges statiques.

c) La contrainte de la traction née dans les armatures est, dans la plupart des cas, d'une valeur moindre à celle prévue. Mais si on considère le fait que des valeurs proches de la contrainte admissible sont reconnues par endroits, il faut être très attentif au cylindrage et aux autres opérations durant la phase d'exécution.

Bien que la forme de la répartition de la contrainte de traction décrive une courbe convexe sous le sol, la valeur maximale ne s'en trouve pas nécessairement sur la ligne d'effondrement actif de Coulomb.

d) Les mesures indiquent que la valeur du coefficient de frottement entre le sol et les armatures est $f = 0,37$. Dans le cas présent, alors que l'on considère un angle de frottement intérieur de la matière de remblai égal à $\phi = 22°$, la valeur est voisine de $f = \tan \phi$.

Trois années après l'achèvement des travaux, aucun événement imprévu ne s'est encore produit.

Comme on vient de le voir, les valeurs mesurées se trouvant généralement du côté de sécurité de leur valeur de projet dans les récentes recherches, la technique terre armée peut être adoptée pour n'importe quels travaux de remblai, à condition que soit effectué un contrôle suffisant au niveau de l'exécution. De par le passé, nous avons travaillé essentiellement sur des constructions soi-disant rigides et ceci nous rend instinctivement mal à l'aise face à une construction réputée souple, comme suivant la présente technique.

Par ailleurs, bien que la sécurité du comportement des constructions suivant cette technique en période sismique soit vérifiée de manière expérimentale, de nombreuses inconnues restent à résoudre, relativement aux constructions réelles.

BROWN R.E., JONES J.S. et MAYNE P.W.

~~Law Engineering Testing Company, USA~~

DIMAGGIO J.A.

Federal Highway Administration, USA

A parametric study of stone column behavior

Etude paramétrique du comportement des colonnes ballastées

INTRODUCTION

Nous avons fait une étude paramétrique de "finite element" au moyen de courbes hyperboliques d'effort déformation, dans des conditions de drainage et de nondrainage, pour évaluer quantitativement les paramètres majeurs qui affectent le comportement d'une colonne de pierre unique; nous avons trouvé que les courbes de déformation par charge d'une colonne de pierre pourraient être nomralisées convenablement an tracant le rapport déformation verticale/diamètre de la colonne en fonction du rapport charge axiale/tangent de l'angle de frottement de la pierre pour chaque sol ayant une résistance, au cisaillement différente. On a trouvé que plus de 50% de la charge axiale était transféré de la colonne de pierre à l'argile, sur une profondeur d'un diamètre de colonne. A mesure que s'abaisse la résistance de l' argile anvironnante, la charge axiale est transférée plus bas le long de la colonne.

Prediction of stone column load-settlement behavior in soft clay should utilize effective stresses under both undrained and drained conditions to bracket the true behavior. In addition, since normally-consolidated clay deposits typically exhibit an increasing shear strength with depth, prediction methods should represent this phenomenon in order to realistically model field situations.

A parametric finite element study utilizing hyperbolic stress-strain curves was conducted for drained and undrained conditions to quantitatively evaluate the major parameters affecting the behavior of a single stone column. Long-term drained behavior occurs due to consolidation of the soft clay and dissipation of positive pore pressures through the stone columns.

Numerous researchers have modeled stone column systems in soft ground environments. Such investigators include: Hughes, Withers, and Greenwood (1975), Balaam, Booker and Poulos (1976), Engelhardt and Golding (1975), McKenna et al (1975), Datye and Nagaraju (1976), and others. Although finite element techniques are often used in these analyses, the shear strength characteristics of the soft clay are often represented by a single value (S_u) for the entire deposit. More commonly, however, soft ground consists of normally-consolidated deposits where the undrained shear strength of the soil may be better represented as a shear strength-overburden ratio (S_u/σ'_v).

In addition, stone and clay stress-strain characteristics are commonly represented by linear or bilinear functions. However, a better approximation of the column behavior should be achieved by a nonlinear formulation which more closely approximates the true stress-strain behavior.

FINITE ELEMENT MODEL

The finite element method (FEM) used in this study has been described by Desai (1974 and 1975). The model uses hyperbolic stress-strain functions developed by Duncan and Chang (1970). Vertical loading was applied incrementally to a single isolated column to simulate the loading during a field load test. The length of the column was approximately 9.2 meters and the base of the column was firmly supported. The model also employed interface elements to allow slippage between the column and soft ground. These slip elements were assigned the same properties as the clay matrix.

SOIL AND COLUMN PROPERTIES

The important parameters for the prediction of stone column behavior have been discussed by Hughes et al (1975). These parameters along with the parameter variations used in this study are listed below.

Stone Effective Friction Angle
$35° \leq \phi_s \leq 45°$

Column Diameter
$0.9M \leq D \leq 1.2M$

In Situ Clay Shear Strength
undrained case $0.18 \leq S_u/\sigma'_v \leq 0.36$
drained case $C=0$, $\phi'_c=30°$

At Rest Earth Pressure Coefficient
all cases $K_o=0.6$

The undrained and drained deformation moduli were represented by using a hyperbolic formulation. The moduli and shear strength each increased with depth. Poisson's ratios of 0.49 and 0.35 were assumed for undrained and drained loading conditions, respectively

PREDICTED RESPONSE

The results of the FEM study for undrained conditions are shown in Figure 1. Stress-strain behavior was found to give congruent results if the axes were normalized to account for column strength ($q_a/\tan\phi'_s$) and column diameter (p/D). As would be expected, greater load capacities are possible for soils with higher S_u/σ'_v ratios.

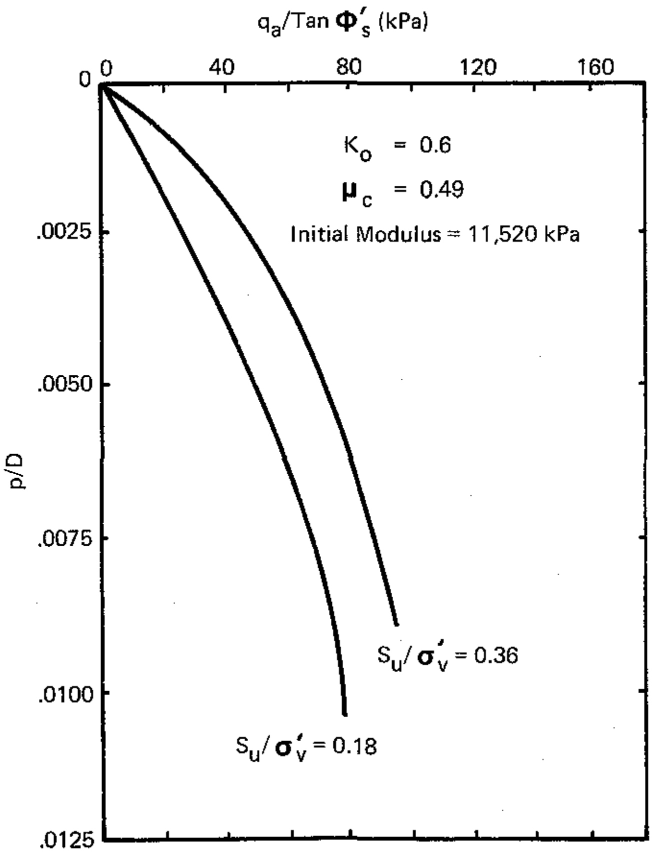


Fig. 1. Undrained Load-Deformation Behavior

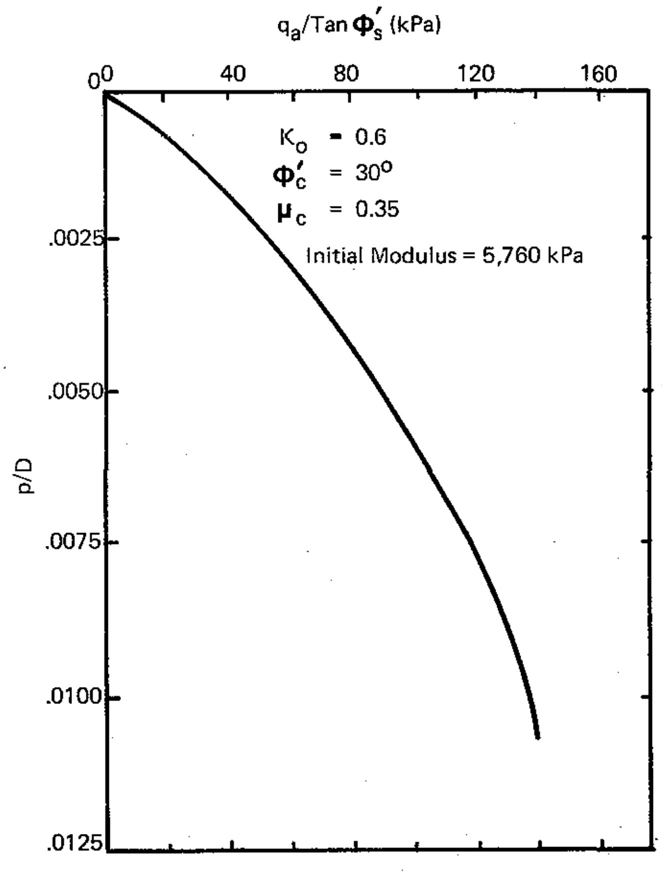


Fig. 2. Drained Load-Deformation Behavior

Predicted drained response is presented in Figure 2. Since drained shear strengths are greater than undrained strengths in normally-consolidated soils, the stone column carried greater loads under drained conditions.

The transfer of load through the column is shown in Figure 3 for various stages of loading. It is interesting to note that as the ultimate capacity was approached, approximately 85 percent of the total axial load was transferred to the clay within the upper 1.5 meters of the column and only four percent of the ultimate load ever reached the base of the column.

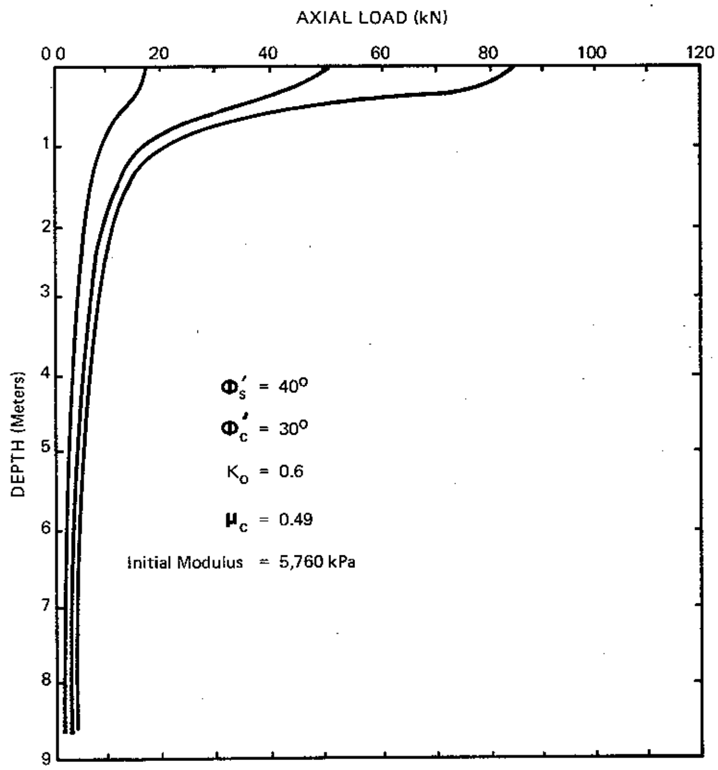


Fig. 3. Stone Column Load Transfer

Based on the results obtained from the analyses, single column settlement at less than one-third of the ultimate load may be approximated using the following empirical equation for the range of parameters normally encountered in practice:

$$p = q_a D/(F \tan\phi'_s \; s_u/\sigma'_v)$$

Where

F = 70,000 kPa for undrained loading
= 35,000 kpa for drained loading

CONCLUSIONS

It was found that stone column load settlement curves could be adequately normalized by plotting vertical settlement/column diameter versus axial load/$\tan\phi'_s$(stone friction angle) for each soil having a different shear strength.

It was also found that over 50 percent of the axial load was transferred from the stone column to the clay within a depth of one column diameter. As the strength of the surrounding clay decreases, the axial load is transferred further down the column. The occurrence of this phenomenon was noted by Hughes and Withers (1974) in laboratory models and Jones (1972) in field tests. The prevention of the upper bulging prior to failure may substantially increase the load carrying capacity of the column as suggested by Jones (1972). Such methods may include (1) casing the upper portion of the column and (2) grouting or stabilizing the top of the column.

LIST OF SYMBOLS

D = Initial diameter of stone column (meters)

σ'_v = Effective overburden stress
F = empirical settlement coefficient (kPa)

K_o = Coefficient of earth pressure at rest

ϕ'_s = Effective friction angle of column material

ϕ'_c = Effective friction angle of clay soil

q_a = Applied axial stress (kPa)

p = Settlement of column at ground level (meters)

S_u = Undrained clay shear strength

REFERENCES

1. Balaam, N., Booker, Jr., and Poulos, H. "Analysis of Granular Pile Behavior using Finite Elments," Sydney Univ., Oct. 1976, NTIS.

2. Datye, K. and Nagaraju, S., "A New Technique for Ground Improvement," New Horizons in Construction Materials, 1976, pp 187-198.

3. Desai, C.S. "A Finite Element Code for Soil-Structure Interaction and Foundations," March 1975, Virginia Polytechnic Institute.

4. Desai, C.S. "Numerical Design-Analysis for Piles in Sand" ASCE GT6, June 1974, p 613.

5. Duncan J.; Chang, C.Y., "Nonlinear Analyses of Stress and Strains in Soils", ASCE Journal, SMFD #5, Sept., 1970, pp 1629-1653.

6. Engelhardt, K. and Golding, H. "Field Testing to Evaluate Stone Column Performance in a Seismic Area" Ground Treatment by Deep Compaction, Geotechnique, VOL XXV, No. 1, March, 1975, pp 61-70.

7. Jones, J.S. "Bearing Capacity of Cylindrical Aggregate Piles," Thesis, Univ. of Virginia, June 1972.

8. Hughes, J., Withers, N., Reinforcing of Soft Cohesive Soils with Stone Columns,"Ground Engineering, 1974 May, pp-42-49.

9. Hughes, J., Withers, N., and Greenwood, D. A Field Trial of the Reinforcing Effect of Stone column in Soil," Ground Treatment by Deep Compaction, Geotechnique, VOL. XXV, No. 1, March, 1975, pp 31-44.

10. McKenna, J., et at. "Performance of an Embankment Supported by Stone Columns in Soft Ground" Ground Treatment by Deep Compaction, Geotechnique, VOL. XXV, No. 1, March, 1975, pp 51-60.

SIMONS H. et KRUEGER P.
Technische Universitaet Braunschweig, RFA

Behaviour of «reinforced earth» in theory and practice

Comportement de la terre armée en théorie et en pratique

Résumé

Dans le cadre d'un projet de recherches de plusieurs années des recherches théoriques d'après la methode des éléments finis ont été exécutées. Aux trois chantiers le comportement contrainte -déformation des ouvrages en terre armée ont été mesuré. De plus on a fait des essais sur modèle dans l'echelle 1 : 5 et 1 : 20. On constate un comportement de la charge de rupture tout à fait différent entre des ouvrages étroits et des ouvrages larges.

Introduction

The Lehrstuhl fuer Grundbau und Bodenmechanik of the Technical University of Braunschweig is working at a research project in order to optimize and develop the construction method "Reinforced Earth". This research project is promoted by the Minister for Research and Technology.

In the course of this research program the stresses and strains which appear in the structures are measured in order to get to know more thoroughly the mechanical behavior of this composite construction. For the purpose of verification and transmission of the measuring results a suitable finite-element-program was developped, by which the stress-strain behavior in structures of reinforced earth was verified.

Furthermore models in the scale 1 : 5 and 1 : 20 had been erected, in order to be able to study the influence of different lengths and arrangements of the reinforced strips as well as the failure load behavior of such structures. Smooth and ribbed strips had been installed in the models.

Investigation with finite elements

General

By means of the finite element method a calculation program was established for the determination of stresses and strains in structures of reinforced earth. For the soil the non-linear stress-strain relationship of Duncan and Chang was applied, for the reinforcement and wall elements an ideal-elastic stress-strain relationship was applied. The wall elements were assumed to be stiff, the reinforcement elements to be flexible. A plain strain was considered, the connection between the single elements was assumed to be stiff and calculated according to the displacement method.

First of all it was investigated how great the influence of possible soil parameters on the calculation results is. It proved that the movements and the tensile forces partly depend considerably on the given parameters. The influence of the parameters on the stresses is, however, small.

Deformations of the wall

The calculated deformations are shown in fig. 1.

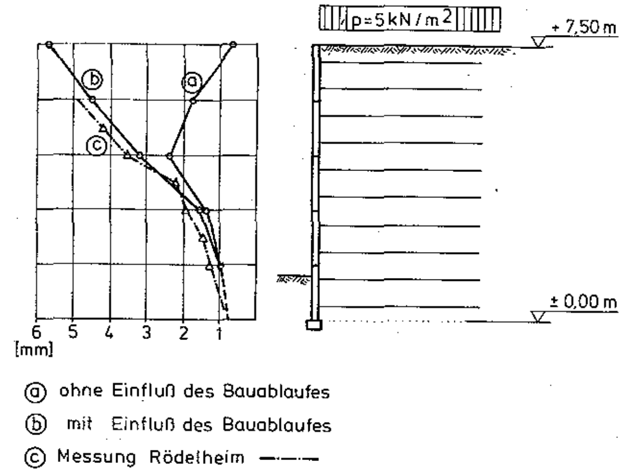


Fig. 1 Horizontal wall movement

Curve a shows the deformation of a wall, if the wall panels would move from their planned state of installation. Actually the movement of the panels below is added to those movements (curve b). The difference between curves a and b is thus a consequence of the construction procedure and has no influence on the state of stress of the soil in the structure. The curve c shows the measuring results of the reinforced earth structure Frankfurt-Roedelheim for a uniform distributed load of 17 kN/m^2.

Flexural stiffnes of the wall panels

Stiff and flexible wall panels had been compared by means of calculations. The stiff wall panels correspond approximately to the concrete wall panels used at present with a thickness of 20 cm; the flexible wall panels correspond to the steel plate facing.

The flexural stiffness of the wall panels has small influence on stresses and strains inside the structure. In the zone of the wall, however, the movements became greater on decreasing flexural stiffness, and as a result of this the earth pressures on to the wall panels became smaller. In the case of the stiff wall the active earth pressure turns up approximatively, whereas in the case of the flexible wall the active earth pressure decreases considerably. Thus the connection force between reinforcement and a flexible wall is smaller than the maximum tensile force in the strips (fig. 2 curve a). Due to the yieldingness of the wall a larger part of the earth pressure is transmitted on to the strips already in the initial zone of the reinforcements.

In the case of stiff walls with stiff connection to the reinforcing strips the maximum tensile force of the strips turns up at their connection point (fig. 4, curve b).

Our measurements in Roedelheim showed for the concrete wall facing a behavior between the curves fig. 4 a and b.

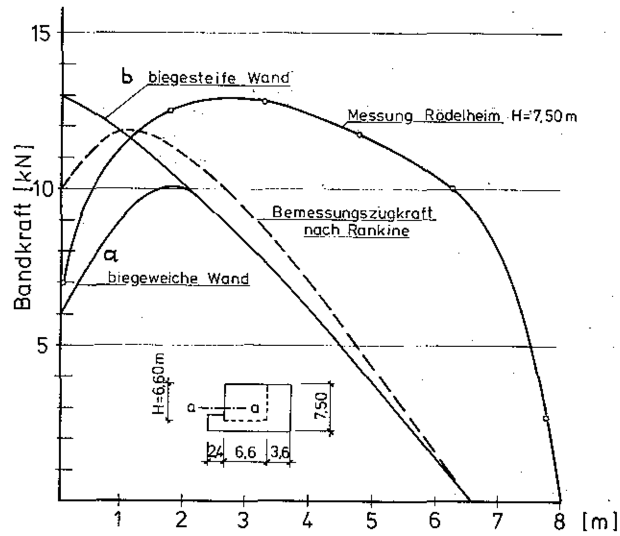


Fig. 2 : Calculated and measured tensile forces at the connection points of the reinforcements

This is to be attributed to the clearance between the joints of the strips with the panels, by which a part of the earth pressure is transmitted to the strips and not to the wall panels.

Measurements in nature

General

On three sites in Herborn, Roedelheim and Sinn following measurements had been performed:

1. Tensile forces in the strips in the 1/5 points

2. Wall movements

3. Wall friction

The following measuring results are derived from the structure Roedelheim (fig. 3). Similar observations had also been made in Herborn and Sinn. On all three sites the ratio of width/height of the structure was $w/h \geq 1{,}0$. On all three structures the fill material for the reinforced earth structure was strongly compacted.

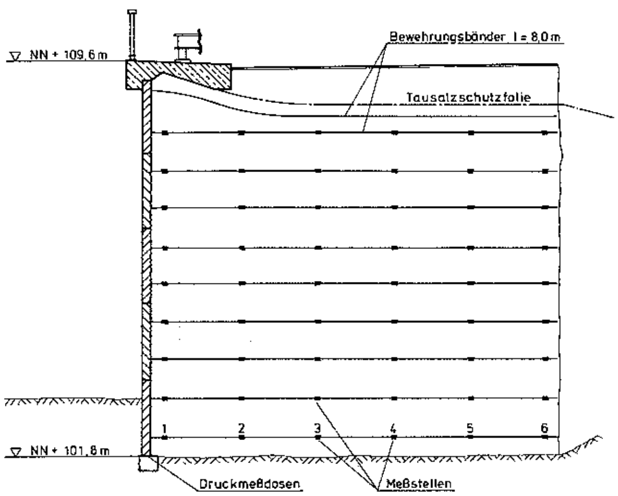


Fig. 3 : Cross section through the reinforced earth wall "Roedelheim"

Tensile forces in the strips

Fig. 2 shows the course of the tensile forces in the strips for the strip layer 7/8. The position of the maximum tensile forces in the strips in 0,45 B is conspicuous. Fig. 4 shows the connecting line of the maximum strip forces. This line runs considerably behind the Rankine straight line $45^o + \varphi/2$.

Due to a high reinforcement percentage and long strips the deformations of the facing wall are so small that an active earth pressure wedge cannot turn up. Thus the maximum strip forces move to the center of the strip.

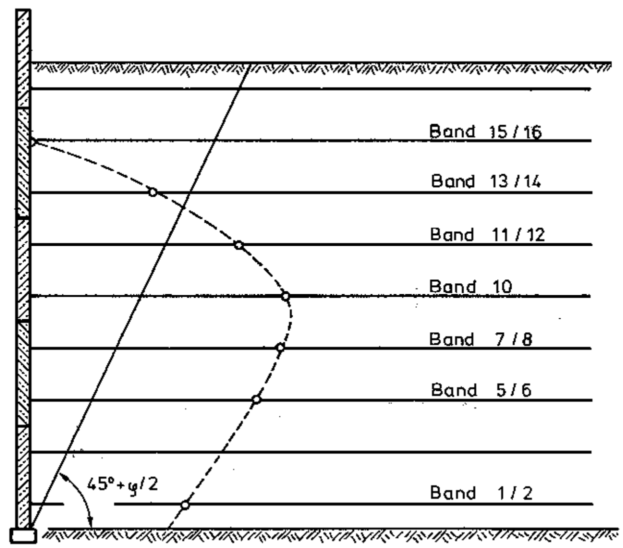


Fig. 4 : Course of the maximum strip forces in Roedelheim

Fig. 5 shows the course of the strip forces in relation to the construction work progress of a reinforced earth structure.

The strip forces increase quickly with growing earth filling. The increase of force corresponds to the earth pressure at rest. The highest strip forces are already attained after an earth filling of 2,0 m. They scarcely increase with furhter earth fillings and in the final state they are considerably below the theoretical values of the active earth pressure.

Deformations

The deformation was plotted for an earth filling of 0,75 m each ($p = 17\ kN/m^2$) in fig. 1. For the filling process the largest deformations amounted to 23 mm. They occured at a height of the wall of 0,4.

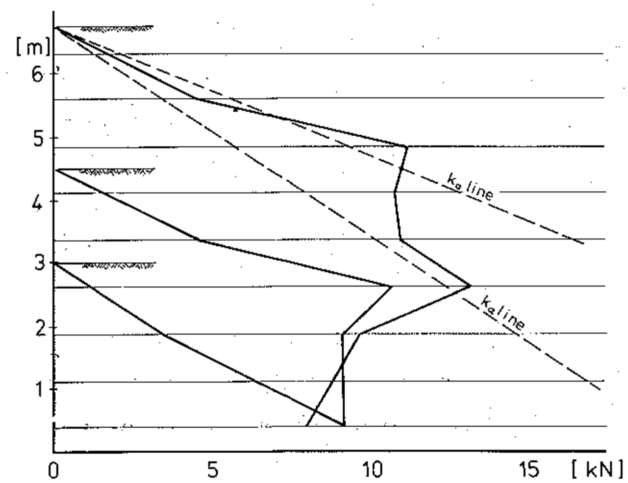


Fig. 5 : Strip forces in relation to the construction work progress

Model experiments

Two models in the scale 1 : 20 and 1 : 5 had been used for the experiments. At the model in the scale 1 : 5 the erection process and influences of live loads had been maesured. The measuring results confirm the site tests.

At the model in the scale 1 : 20 failure load tests had been conducted. Smooth and ribbed strips of different lengths had been installed. The failure process was produced by tensile forces at the facing panels. By this means the ultimate load could be determinded exactly. The strips were designed in such a manner that the state of failure could not occur due to the failure of the strips. The failure process was stopped after a movement of 2 cm, in order to be able to study thoroughly the mechanism of failure.

During the experiments with smooth strips the failure surface developped behind the facing panels in an angle of $45^{\circ} + \varphi/2$.

During the experiments with ribbed panels a different failure load behavior of the structure was observed:

In structures with a ratio $w/h < 0{,}6$ the strips were pulled out of the anchorage zone. The failure surface ran behind the wall in an angle of $45^{\circ} + \varphi/2$.

In structures with a ratio $w/h > 0{,}6$ the failure surface developped behind the reinforced earth body. The interlocking inside of the structure was so great that the whole reinforced earth body slided forward.

The experiments had been conducted on densely compacted sand. Experiments on loosely compacted sand are still running.

FLOSS R. et THAMM B.R.
Bundesanstalt für Strassenwesen, RFA

Field measurements of a reinforced earth retaining wall under static and dynamic loading

Mesures in situ sur un mur de soutènement en terre armée sous chargements statique et dynamique

Résumé

Le plus important mur de soutènement en terre armée jamain construit en Allemagne est situé sur l'autoroute A8 à hauteur de Fremersdorf, Sarre, prés de la rivière de même nom. L'ouvrage s'élève à une hauteur de 6 à7,30m sur une longuer de 970m; des travaux de rectification du lit de la Sarre permettent d'exécuter les travaux au sec. Le terrain naturel du site est en grés aves une couche susjacente de sable et de gravier. En raison de l'envergure du projet, il a été décidé de saisir l'occasion d'étudier le comportement en place pendant la construction au moyen d'essais de surchage statique et dynamique effectués sur une section préparée à cet effet et où le mur atteint une hauteur d'env. 7,30m. Outre les instruments de mesure permanents ont été installées des armatures passives destinées à être mises en tension à des moments déterminés afin d'étudier le progrès de la corrosion.

La rapport présente et discute les résultats obtenus. L'évulation des résultats des essais montre que les hypothése faits sur la justification de la stabilité interne de remblai en terre armée formulées dans le "recommandations provisoires 'allemands pour l'application du mode" sout entirêment vérifiées et laisse même une certaine marge de sécurité.

1 Introduction

The largest reinforced earth retaining wall so far being built in Germany is on Highway Route A8 near Saarbrücken (Fremersdorf), close to the river Saar. The wall is of about 6 to 7,30m height and 970m length and could be built in the dry due to waterway regulation works of the Saar. The existing soil in-situ consists of sandstone with overlying sand and gravel. Since the wall is within the influence of annual highwater of the Saar, certain special construction measures had to be taken into account:

- a free draining stone fill
- special backfill material having a high permeability
- filter fabric to allow free drainage, while at the same time retaining the fines, and
- protection against corrosion problems due to the water of the river.

Because of the size of this project it was decided to study the behaviour of the wall during construction and during static and dynamic surcharge loading within an instrumented test section where the wall is approximately 7,30m high. The purpose of this study was to utilize the obtained data to the preliminous German standards for reinforced earth structures [1]. In this report some of the results obtained will be presented.

2 Wall and Backfill Materials

A typical cross-section of the Fremersdorf wall is shown on Fig. 1. The wall was constructed and calculated according to the German standards.

The joints between the concrete facing panels were covered on the backface with a filter fabric to allow free drainage of the backfill. On top of the reinforced earth body there is a PVC-foil to prevent the penetration of surface waters containing calcium chloride into the construction. About 60 so-called "dead"-reinforcing strips were placed all over the retaining wall in order to investigate in certain time steps the corrosion progress of the reinforcing strips.

The backfill consists mainly of two different granular materials:
- a specified filter gravel (Fig. 1, ③) having a permeability of approximately 10^{-3} m/sec

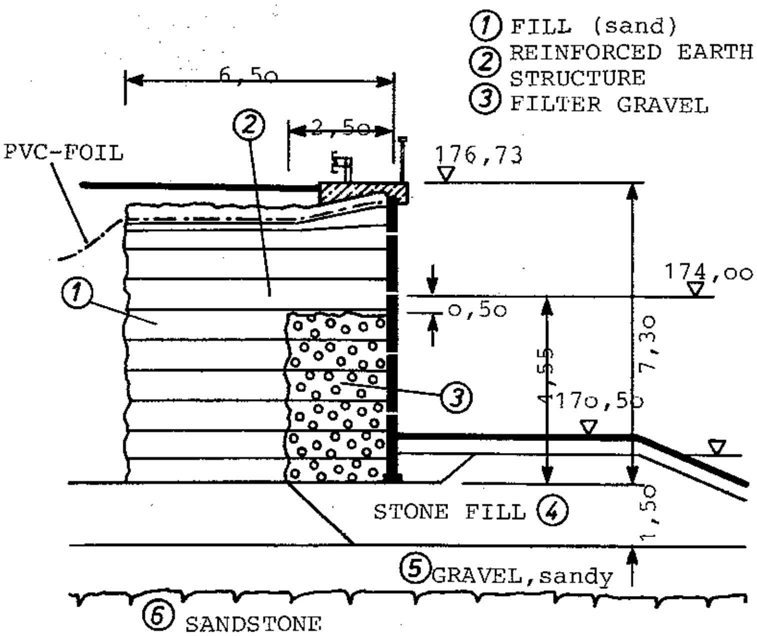


FIG.1: Cross section of Fremersdorf wall at km 43+o25

- a fine to medium sand (Fig. 1, ②) having about 13% of fines passing the 0,063mm sieve.

Both backfill materials were saturated with Saar water and tested according to German standardsfor pH-values and for the specific electrical resistance in Ohm times cm. The chosen backfill fulfilled the German standard [1] requirements for all soil mechanical and electro chemical aspects.

After placement of each lift of backfill density tests were performed in-situ.

Some of the chemical and physical properties obtained in the laboratory and in-situ are as follow:

- filter gravel:

specific gravity	γ_s =	26,5 kN/m^3
dry unit weight	γ_d:	15,2÷20,2 kN/m^3
relative density	D:	0,93÷0,98 [-]
coefficient of uniformity	U=	13,6 [-]
pH-value		7,6 [-]
specific electrical resistance		7120 Ω·cm

- sand

specific gravity	γ_s =	26,5 kN/m^3
dry unit weight	γ_d:	15,1÷19,7 kN/m^3
relative density	D:	0,93÷1,03
coefficient of uniformity	U=	6,0
pH-value		6,2
specific electrical resistance		11610 Ω·cm

Due to above density-properties an angle of internal friction of about 35° and a density of 18,5 KN/m^3 for both materials were assumed.

3 Test Section and Test Program

The test section was selected in a 6,0m wide region where the wall is approximately 7,30m high. The instrumentation used in monitoring static and dynamic stresses and deformations during construction and surcharge loading consisted of (see Fig. 2 and 3):

- 25 hydraulic pressure cells to measure vertical and lateral soil stresses (V1-11;H1-14); 100 strain gages on 10 selected reinforcing strips to measure tensile strains;
- 20 gage points on the concrete panels to determine deformations (W1-20);
- 10 dynamic pressure cells (BM1-10) and 9 dynamic transducers (SA1-9) to determine the dynamic response of the reinforced earth body.

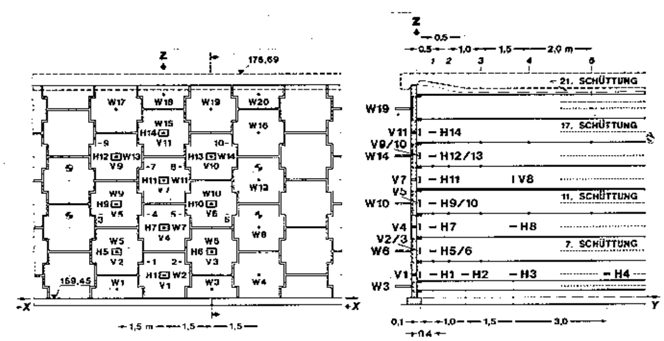


FIG.2: Test section with instrumentation for static measurements

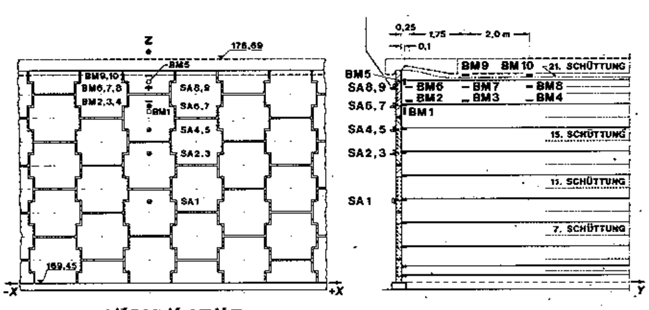


FIG.3: Test section with instrumentation for dynamic measurements

The test stages were as follow:
- measurements at construction stage up to the end of construction
- static measurements with a dead load (layers of concrete panels successively placed and replaced on top of the wall) up to a surcharge of 36 kN/m^2, which is about the German standard load for heavy trucks (see (Fig.4);
- static and dynamic measurements with a heavy loaded truck (335 kN) in line with the later heavy traffic at different cross sections; (Dynamic measurements at a speed of about 20,40 and 60 km/h);
- dynamic measurements with a vibratory roller (f=25 Hz, weight 90 kN) on top of the wall.

4 Results

4.1 Construction Stage

In the region where the test section was selected, there were two periods of highwater overflooding the whole site during the early stage of construction. In both cases the performance of the already erected parts of the structure was extraordinary good. Especially

FIG.4: View of Fremersdorf wall with test section and dead surcharge load

due to the stone fill and the filter gravel the placement of new lifts could go on soon after water table lowering. Similar good performance was shown by all measurement devices placed during construction, because waterproof methods were taken into account in advance. Thus from all the strain gages used only about 16% failed.

For the end of construction including the placement of the base course of Highway A8 the lateral earth pressure distribution is shown on Fig.5. The measured results are compared with the assumptions for active pressures and at rest earth pressures (dotted lines).

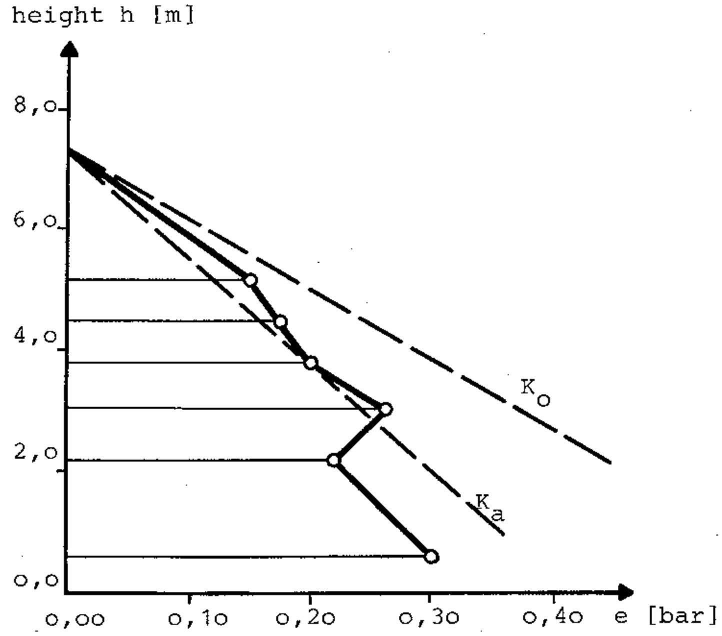


FIG.5: Lateral earth pressure distribution

The lateral earth pressure distribution measured is very similar to that predicted by Terzaghi for a wall rotating about its uppermost edge. The construction procedure obviously prevents the development of full active earth pressure for the upper part of the wall. These conclusion is verified by measurements of the horizontal displacements on top of the wall, which are about 2 to 3 mm (<0,0005·H) in magnitude thereby not allowing interparticle sliding of the fill which is necessary to

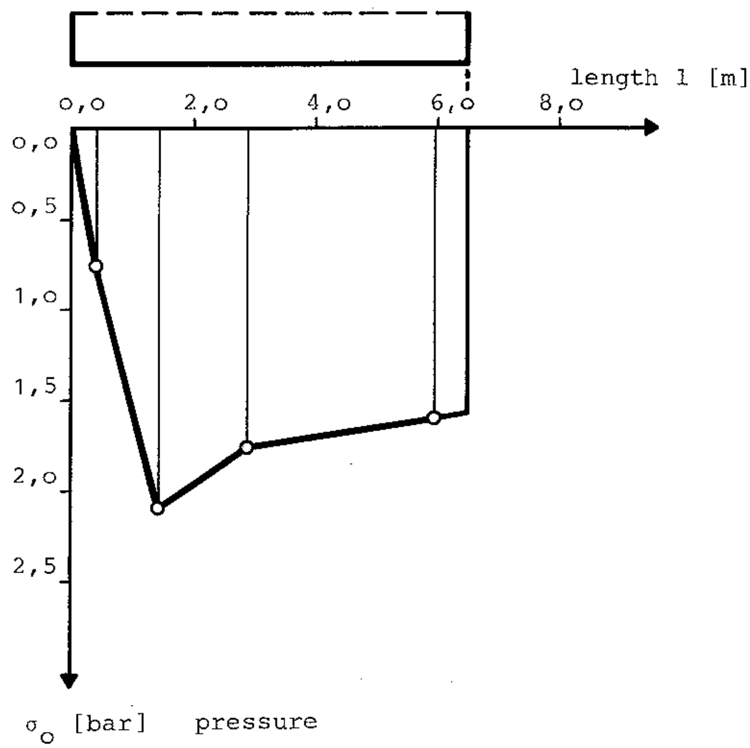


FIG.6: Pressures at base of RE-body

reduce the lateral earth pressures to a minimum. When utilizing the obtained data it seems obvious that the upper part of the wall should be designed for the at rest condition. For design practice using the active earth pressure assumption existing walls are nevertheless safe because adherence is in general more critical than tension at the top of the wall.

Fig.6 shows the σ_o -pressure distribution at the base of the wall for the end of construction stage. From the shape of this pressure distribution it is clearly understood that reinforced earth retaining walls are in general flexible structures. It can be shown that conventional stability analysed lead to similar pressure distributions, thus justifying the assumption that a reinforced earth body can be regarded as a monolitic bloc for external stability calculations.

The tension force distribution along reinforcements in different levels of the wall is plotted on Fig.7 for the strips 2,5,8,10 respectively and for the end of construction stage. Interestingly to note is the line of maximum tension forces (dotted line), where the zone of peak tension moves away certain distances from the facing. For the lower and uppermost strips these distances are approximately between 1/6 and 1/3 of the strip length where as for the middle two strips these distances are between 1/3 and 1/2 of the strip length. From the qualitive distribution of horizontal movements it can be shown that the facings at the base and at the top of the wall react stiffer, thereby allowing arching to occur, which again is verified from the lateral earth pressure distribution.

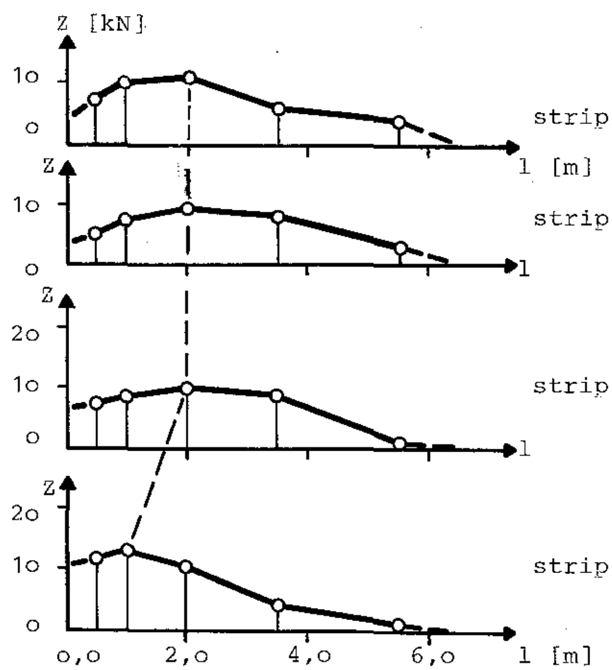


FIG.7: Tension force distribution along reinforcement for different strip levels

Comparing the tension obtained in the strips close to the facing with the measured forces due to lateral earth pressures on the facing equilibrium is obtained when assuming a parabolic earth pressure distribution between the neighbouring strips within an error of about 5 to 10%. This realistic distribution would than reduce earth pressures on the facing elements to about 2/3 of the assumed earth pressure distribution.

4.2 *Static Surcharge Loads*

The dead surcharge load was placed with 8 lifts of concrete facing panels on top of the wall to obtain the desired surcharge pressure of 36 kN/m^2 on an area of 3 times 6 m (Fig.4). Readings were taken every two lifts of placement and replacement of the panels. The change of earth pressure due to the maximum surcharge load is shown on Fig.8, compared with the theoretical assumption (dotted line) commonly used for design.

The magnitude of stresses and strains due to the surcharge load is in general about 1/10 of the magnitude of stresses and strains developed during the construction of the wall.

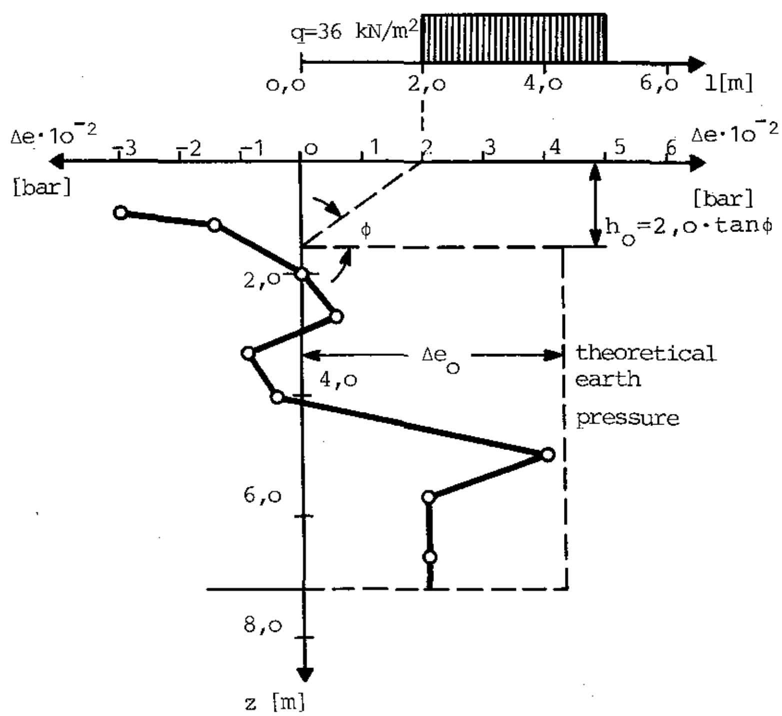


FIG.8: The change of earth pressure distribution due to surcharge load

For the two uppermost pressure cells (BM1 and BM5) which are in first place for the dynamic measurements static readings show a slight reduction of earth pressures in the upper portion of the wall. This tendency may be explained by looking at the lateral earth pressure distribution on Fig.5. Due to shear strains developed during loading local interparticle sliding of the fill can take place, thus reducing the state of stress in the upper region of the wall from an almost at rest condition to a close to active earth pressure situation.

The angle of load distribution was found from the measurements of the vertical earth pressures to be in the order of approximately 40°.

The change of tensile forces within the reinforcing strips is about 10% of the forces developed during construction with a qualitative similar distribution along the reinforcements showing a moderate movement from the zone of peak tension found during construction away into the reinforced earth structure and a slight decrease of maximum tension forces with depth. Close to the facing the change in tensile forces is almost zero for the uppermost strip rows which is in accordance with the change of earth pressure distribution shown on Fig.8. The horizontal movements of the panels due to surcharge load are in magnitude about 0,5mm. This small deformation leads together with movements of the construction stage to values close to 0,0005 times height H of the wall, which may lead to local interparticle sliding, thus confirming the earth pressure and tensile strain measurements obtained.

The heavy loaded truck (335 kN) was mainly placed for static measurements on different sections on the top of the wall to have comparable results with the dynamic measurements discussed thereafter.

4.3 Dynamic Surcharge Loads

After the static surcharge loading the loaded truck was used to produce dynamic changes of stresses and strains when passing the test section at a speed of 20,40,60 km/h within different lines on the base course. Preliminary studies showed almost velocity independent behaviour. Therefore the line close to the coping of the wall was chosen together with a speed of approximately 40 km/h for the main investigations.

From the present state of the study the following tendency can be drawn:

- the maximum amplitude of vertical deformations of the wall is about 30 μm in magnitude for the upper third (Fig.9);
- the amplitudes of horizontal deformations correspond qualitatively to the measurements of the static case (Fig. 9) in magnitude they are about one third of the vertical deformations for the upper third;
- the change of tensile stress within the strips due to dynamic loading is between 30 and 50% of the static loading (Fig.10);
- similar ratios between static and dynamic loading are found for the earth pressures in the upper part of the wall;

The second part of dynamic surcharge loads was a vibratory roller on top of the base course to study dynamic effects during compaction of the fill. The maximum vertical amplitude measured in this case at a frequency of 24,5 Hz was constantly around 220 μm, while the horizontal amplitude reached about 140 μm at the top increasing within 2m depth to about μ190m. During the test which lasted for every step about 5 min no unusual dynamic phenomena appeared.

5 Conclusions

The conclusions reached after studying the preliminous obtained results of the measured project are as follow:

- maximum earth pressures and maximum tensile strains in the reinforcing strips are developed during the construction period
- the construction sequence plays an important role for the final state of stress within the reinforced earth structure
- the lateral earth pressure distribution measured is very similar to that predicted for a conventionel wall rotated about its uppermost edge
- the tension force distribution along the strips shows zones of peak tension moving away from the facing less in the upper and lower regions of the wall and more in the middle
- lateral earth pressures on the wall facing elements are reduced to about 67% assuming a parabolic distribution between neighbouring strips
- negligible deformations of the wall facing were observed
- surcharge loads comparable to the German standard for heavy traffic lead to an increase of soil stresses and tensile strains in the strips of only about 10%

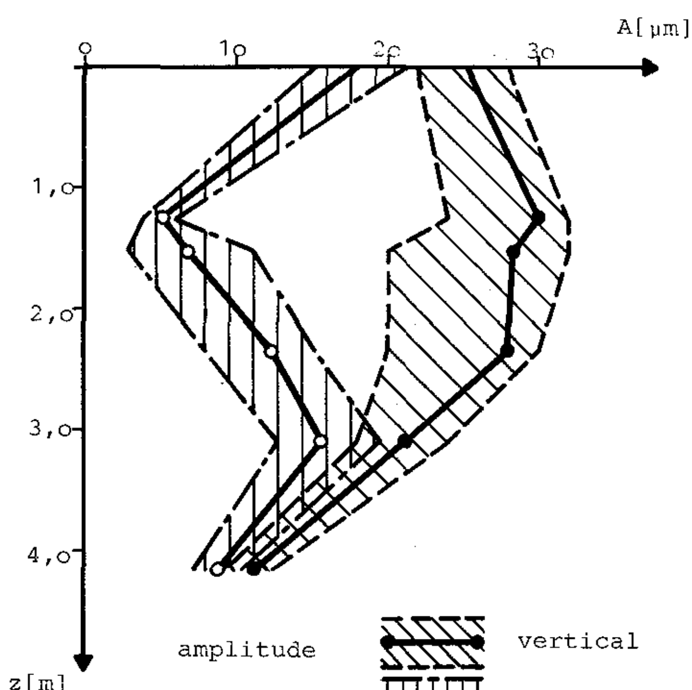


FIG.9: Amplitude spectrum of deformations along upper part of the wall

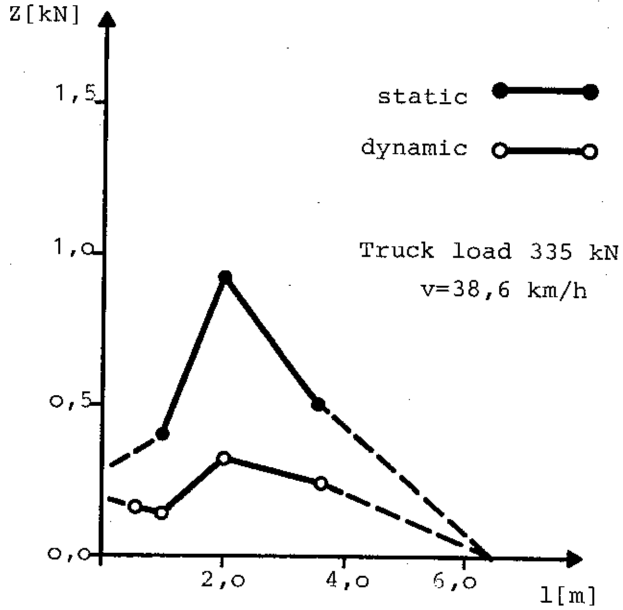


FIG.10: Tension force distribution along strip 10 for dynamic and static surcharge

- surcharge loads reduce a possible state of stress in an at rest condition to an almost active earth pressure distribution in the upper region of the wall
- during all dynamic tests no unusual dynamic phenomena appeared

The experience gained from this study is the good overall performance of the reinforced earth retaining wall at Fremersdorf during construction with highwater periods and during static and dynamic surcharge loading with working loads similar to those under normal traffic. The design according to the German standards [1] is on the safe side even if it does not predict truely the behaviour of reinforced earth.

6 References

[1] Vorläufige Richtlinien für die Anwendung des Bauverfahrens 'Bewehrte Erde', Ausgabe Januar 1977, aufgestellt Bundesanstalt für Strassenwesen, Köln, Herausgeber Bundesminister für Verkehr, Abt. Strassenwesen.

Achevé d'imprimer
sur les presses
de l'imprimerie Rosay, 94300 Vincennes,
le 15 octobre 1980.
Dépôt légal : 3e trimestre 1980.