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Discussions: Slopes and excavations

• QUESTION TO TATSUOKA

Q : P. Segrestin
(Terre Armee Internationale, France)

In the conclusion of his Keynote Lecture "Role of facing rigidity in soil reinforcing", Professor Tatsuoka acknowledges that, while wrapped around or gabion facings are quite acceptable for temporary or rural structures, the concrete panels used in most Terre Armée® walls have the advantage of being durable, aesthetically attractive and rigid enough to restrain the deformation of wall face. They adapt better to important permanent retaining walls.

However, Prof. Tatsuoka considers that for structures bearing heavy loads or bridge abutments, a full-height countinuous rigid facing is advisable —as in the so-called "stage construction method" or RRR system— to ensure their long term stability and maintain deformations within allowable limits.

Hence, it seems first necessary to remind that there are now 2800 Terre Armée® abutments in the world, supporting considerable bridge loads without any problem ; then to underline some major drawbacks in the RRR system.

It is claimed that this system accepts reinforcement lengths not exceeding 30 % of the wall's height (Tatsuoka 1991). However, it is easy to verify that a structure thus proportioned, even before being overloaded, is very close to sliding on its base. It stands only thanks to the embedment of the rigid facing into the ground as clearly confirmed by the loading test results shown on fig. 1. Opening a trench at the wall's toe would risk disastrous consequences.

It is also explained that the system's stability under surcharge accounts for two reasons :
1/ the rigid facing obliges potential failure lines to pass below its bottom whereas, it is said, with a discrete facing they would come out at mid-height.
2/ part of the backfill's weight and surcharge is transmitted by negative friction to the facing.

Obviously, point one would be better resolved if one merely added reinforcements where necessary,

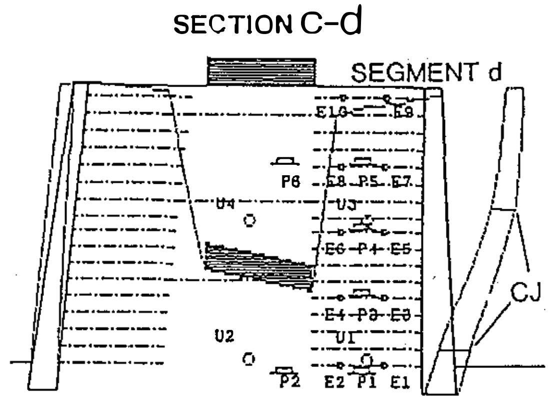

Fig. 1 - Deformation of JR n°1 test embankment by loading test (from Tatsuoka, op. cit.).

rather than shoulder the structure with a thick expensive facing.

Point two is confirmed by the bearing pressures measured by Prof. Tatsuoka under two models, one with rigid facing (type D), the other with discrete facing (type B), (fig. 2). This shows that the rigid facing does concentrate vertical loads, subjecting foundation soil to pressures 5 times greater than the model with discrete facing. Such an abutment does load the ground even more than a classical reinforced concrete abutment !

Thus, one understands that the RRR system with rigid facing no longer has the qualities which make reinforced earth structures attractive.

Employing reinforced earth implies in fact wanting to use it as a material in itself, not as an accessory. It means skilfully putting its resistance to advantage and its flexibility to use ; it means using it to spread the loads as widely and uniformly as possible and thus being able to even handle the case of grounds with poor bearing capacity.

The RRR system or "stage construction method" has lost these essential properties. Moreover, it requires a lot of manual work on site (temporary gabions, shuttering...) and lengthens construction time considerably.

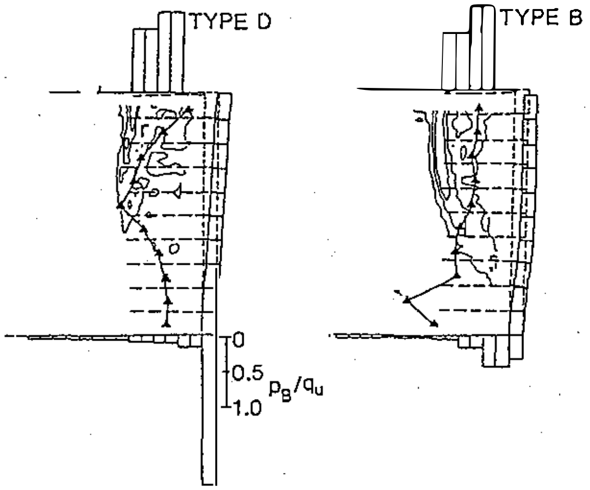

Fig. 2 - Pressure distribution at the bottom of model walls (from Tatsuoka, op. cit.).

REFERENCE

Tatsuoka, F., O. Murata, M. Tateyama. 1991. Permanent geosynthetic-reinforeced soil retaining walls used for railway embankments in Japan. *Denver, Colorado.*

A : F. Tatsuoka
(*University of Tokyo, Japan*)

In my keynote lecture, a geotextile-reinforced soil retaining wall (GRS-RW) system which uses a full-height continuous rigid facing is introduced. This method has been used by Japanese railway companies for reconstructing railway embankments for more than 6 km in total length. This GRS-RW method takes advantage of the structural role of the facing and is incorporated into design. In particular, the reaction on the bottom of facing is expected to work so as to increase the stability of wall. As Mr. Segrestin discussed, this design method is different from that of Terre Armee. He argued that this design method is not safe, because unexpected excavation in front of facing in the future may endanger the stability of wall.

This discussion point is already answered in my keynote lecture. I would like to add the following points, however:
1) I wonder why Mr. Segrestin is worried about this type of danger only for GRS-RWs. It should be reminded that usual permanent conventional retaining walls (gravity-type and cantilever-type) rely on the reactions at the bottom of wall structure, and it is common for them that excavation in front of wall is not allowed. Rather, I would like to ask Mr. Segrestin whether excavation in front of any Terre Armee wall is allowed. With this respect, Jewell (1992) that a question about the reliability of the bearing arrangements at the base of the face, and the susceptibility (or otherwise) of these to an unexpected excavation at the toe of the wall, presumably, is asked during the design of conventional gravity wall structures.

It seems to me that a continuous rigid wall wall used for the GRS-RW may be less endangered by excavation in front of wall than a discrete panels facing used for a Terre Armee wall, since the axial and longitudinal continuity of facing may help in maintaining the stability of wall.
For this type of GRS-RW, when the supporting ground is very weak, the deposit below the facing should be improved to some extent, for example by constructing cement-treated soil columns. This method has already been used at several sites including Biwajima, Nagoya and Rokujizo, Kyoto. This ground improvement increased the construction cost. It is noted, however, that even when Terre Armee walls had been constructed at these sites, the weak ground would have been improved in a similar way. This is because an excessive settlement would have led to a largely deformed wall face, which may endanger the stability of the facing even if the wall itself may not collapse.

Mr. Segrestin also argued whether this GRS-RW system is competitive with the Terre Armee system. My answer is "yes" based on the recent our experiences in Japan. Namely, this GRS-RW method won the completion with Terre Armee in several projects for reconstruction of railway embankments by the following advantages:
(a) A low-quality on-site soil which is not allowed to use as the backfill soil of a Terre Armee wall can be used for this system. The use of on-site soil as much as possible reduces the total construction cost very largely.
(b) For these sites, it was requested to limit to a minimum the amount of excavation in the slopes of existing railway embankment. If Terre Armee walls were constructed, in order to use relatively long reinforcement, as illustrated in Fig. 7.2 (b) of my keynote lecture, sheet piles should be installed and they should be anchored in the slopes so as to protect the existing embankments from possible large deformation which may endanger the safety of trains running next to the slope. No

doubt, these measures would have increased largely the construction cost and period.

(c) Discrete panels of the facing for a Terre Armee wall cannot be support relatively large concentrated load at the crest of the facing. Therefore, another independent foundation should be constructed to support, for example, an electric pole or a noise barrier wall. These structures can be constructed directly on the crest of, or immediately behind, a continuous rigid facing for the GRS-RW.

REFERENCE

Jewell,R.A. 1992. Links between the resting, modelling and design of reinforced soil, Keynote Lecture, Proc. of Earth Reinforcement Practice, IS-Kyushu '92.

• QUESTION TO MYLES

Q : M. Fukuoka
(*Science University of Tokyo, Japan*)

Q1 : Traditional nails have concrete cover but your fired nails have no cover. Are there any differences between the two from the mechanical point of effectiveness?
Q2 : Are the nails durable only with galvanized coating?
Q3 : How many nails are installed in a day?
Q4 : I would like to know the fired soil nails in practice.

A : B. Myles
(*Soil Nailing Limited, U.K.*)

The speed of installing nails by the launching technique is rapid; the approximate maximum rate is 15 nails per hour, that is one nail every 3 or 4 minutes. The maximum short term rate achieved is 4 nails every 6 minutes but it is not practical to keep the crew working at that speed so a realistic rate is in the region of about 15 per hour or about 70 to 100 nails perday. The nails are usually galvanized. A previous interesting paper showed the survival of the nails. I don't believe the small amount of grout on many of the nails is really a protection. The speed of the installation of the nails is such that the soil is moved slightly out of the way and we have dug up quite a lot of previously fired nails and found that the normal galvanizing of the nails is unaffected. Just to paint the nails is sufficient to find that the damage during installation is not affected by the speed of entry. I don't dispute Prof.Fukuoka's comments about whether the displacement is due to shear or bending; I think it is due to a combination of both. The fact that this type of nail is quite large in diameter compared to normal nails is important in their behavior. I am sure it will not take you many minutes to work out that by increasing the diameter from the normal 19-20mm to 40-45mm the bending and stiffness increase dramatically. What I believe we need to do with these nails is to orientate them more to take shear and bending. There will be some tension but the tension that you have in such a large bar only utilizes the capacity of the nails in the tensional form to a small extent and their primary resistance to movement is due to the bending or shear.

Q : E. Gartung
(*LGA, Nurnberg, Germany*)

Mr. Myles, I liked the beginning of your presentation when you mentioned geosynthetic nails. I can imagine situations where plastic nails are superior to steel nails. Could you please give us more information on geosynthetic nails made of polymer materials?

A : B. Myles

The initial work that we carried out on synthetics was using woven polyesters in tubes with polymer concrete centers and this proved successful. We have found that the problem is that this type of nail needs to be produced in a special plant. The excellent work done through the years by our colleagues at Terre Armee, France, established the reputation of galvanized steel in soil internationally. The difficulty we clearly have in trying to promote synthetics is that there are large steel plants that produce steel at a low-price. Galvanizing is also done at such a low cost that the bottom line for synthetics is between twice or three times the unit cost of steel nails. As long as the authorities are prepared to accept the galvanized steel nails for long-term use then you have to think about the commercial reality, therefore we sometimes do not promote the use of synthetic nails. I believe next year we may use synthetic nails at a contaminated site where the demand is for exceptional durability. However work that have been done to date says more about the commercial reality that is dominating the market at the present time.

• QUESTION TO VUCETIC

Q : H.I. Ling
(*University of Tokyo, Japan*)

"It is believed that the modelling of construction sequence would affect the performance of the nailed excavation. I would like to know how it was simulated in your centrifuge test."

Reference:

1)Tufenkjian, M.R. and Vucetic, M. (1992). "Seismic Stability of Soil Nailed Excavations," *Proc. IS Kyushu '92,* pp. 573-578.

A : M. Vucetic
(University of California, Los Angeles, U.S.A.)

Soil nailed excavations are affected by the construction sequence in terms of the redistribution of stresses and deformations of the soil. However, a step-by-step excavation and installation of model soil nails was not performed in the centrifuge testing, simply because the modeling of the correct soil nailing construction sequence, while the centrifuge is operational, would be technically extremely difficult. Such centrifuge testing should include a method of installing the soil nails in flight, similar to what is occasionally done in pile centrifuge testing. Even with such an improvement, the main obstacle would still be the simultaneous procedure of excavating in stages and removing the excavated soil in-flight.

The moist sand was built into the model box in alternating black and white layers. Each layer was then compacted. As the number of soil layers increased and the box began to fill up, rows of nails coated with sand were placed horizontally on top of the compacted sand at predetermined elevations. After placing the nails, the next layer of soil was sieved on top of them and compacted. This process continued until all of the nails had been placed and the box had been filled close to the top. In this way, the entire set of nails was buried within the soil during soil compaction. During this process, the accelerometers were also buried at appropriate elevations.

Evidently, such a procedure of installing the nails horizontally differs from the actual construction procedure, where the nails are installed in the drilled holes at a small angle and then grouted. However, the model building procedure employed here provides excellent contact between the nails and the soil, which is an important characteristic of the field construction procedure. It is also believed that the behavior and failure mechanism are not significantly affected by a small difference in the inclination.

To make the model soil deposit more uniform and to bring it to a state resembling the field conditions, two common aspects of geologic history were roughly simulated. The first is the effect of preconsolidation, and the second is densification caused by previous earthquakes. This was done by conducting separate centrifuge test prior to the final dynamic testing (see Vucetic et al., 1993). After this test, the model was removed from the centrifuge and prepared for the excavation of the vertical face. The entire vertical face of the excavation was carefully trimmed back to expose the tips of the buried nails. The facing of the soil nailed excavation model was made from a thin Plexiglass sheet. To mount it flush against the vertical excavation and to secure the nails firmly to it, small diameter holes were drilled into the Plexiglass sheet corresponding exactly to the location of each of the nails. Next, a thin layer of sand was glued onto the inside surface of the facing, and a small amount of glue was placed into each of the holes. The sandy side of the facing was then slid onto and over the exposed nails flush against the vertical face. With an additional application of glue, each nail was individually connected to the facing. These steps were performed to ensure proper frictional interaction between the facing and the soil, and to prevent the facing from sliding off the nails during testing. After the facing was properly mounted, the completed model was remounted onto the centrifuge ready for dynamic testing. For more details on the testing procedure, see Vucetic, et al. (1993).

The authors believe that such a procedure, although not perfect, is sufficiently good to study the mechanism of complete failure under strong ground shaking. Small to moderate differences in the distribution of stresses and strains in the soil nailed mass should affect the behavior of the system during low intensity shaking, but it should not significantly affect the ultimate failure mechanism.

Reference:

Vucetic, M., Tufenkjian, R.M., and Doroudian, M., (1992). "Dynamic Centrifuge Testing of Soil Nailed Excavations," accepted to *ASTM Geotechnical Testing Journal,* June 1993.

Q : N. Sabhahit
(Indian Institute of Technology, Kanpur, India)

"Is your model for nailed soil slope different from model used for reinforced earth wall? If yes, how do you justify?"

A : M. Vucetic

Yes, it is different. We did not build the soil nailed model from the bottom up, like in the case of reinforced earth structures. Instead, we applied the model preparation technique which provides perfect connection between the nails and the soil, and we prepared the soil as close to naturally uniform soil as possible. Both of these aspects are characteristics of the real soil nailed structures with grouted nails.

As described in my answer to the Dr. H.I. Ling's question, the technique of the model preparation did not correspond exactly to the real construction sequence of soil nailed excavations. However, as Dr. Gassler has pointed out already, small differences in the state of stresses and strains in such a model should not significantly affect the failure mechanism, which is associated with large stresses and deformation, and which was of primary concern in the study described in the paper.

• QUESTION TO HEYMANN

Q : E.Gartung
(LGA, Nurnberg, Germany)

Mr. Heymann, you evaluated the pull out resistance of rods. Did you find an influence of ratio between the rod diameter and the hole diameter? Did you find an influence of the method of installation of the nails on their pull out resistance?

A : G.Heymann
(University of Pretoria, South Africa)

All the test nails presented in the paper were of the drilled and grouted type. Furthermore, the diameter of the steel reinforcements were 20mm to 25mm and the diameter of the holes in the order of 100mm. From the above it is clear that the influence of installation method on the soil nail pull out resistance, as well as the influence of ratio between reinforcement diameter and hole diameter could not be investigated.

• QUESTION TO WON

Q : E. Gartung
(LGA, Nurnberg, Germany)

Mr. Won, in your case history all soil nails had the same length of 12 m. Would it not be preferable to vary the length of the nails to avoid a sharp boundary between reinforced and unreinforced soil? In the design we should probably try to create a smooth transition from reinforced to unreinforced soil by a variation in nail length.

A : G.W. Won
(Road and Traffic Authority, Australia)

In reply to Dr E Gartung's question concerning the length of the soil nails, all soil nails were manufactured from 20mm steel bar with a yield stress of 410 MPa and were 12 metres long. For practicality of manufacture all soil nails were made to the same length.

The reason for the 12 metre long soil nails was that a mininum development length of 4 metres was required behind critical slip circles when analysing both for the local stability of the soil nailed wall and the global stability of the Freeway embankment.

At the time of design, the actual shear bond capacity of the soil nails was not known. Reasonable estimates of the shear bond strength between the soil nail and compacted clay/sandstone fill in the embankment was determined from mechanical tests (eg grading, plasticity). Actual pull out test data confirmed that the shear strength capacity for the soil nails was between 20 to 25 KN per metre run of soil nail. Consequently a 4 metre development length behind critical slip circles was sufficient to generate a tension of about 80 KN which was the design axial working load for the soil nail

During soil nailing drilling and installation, sandstone bedrock was encountered at the cut/fill line at either end of the embankment and consequently some of the 12 metre long nails at either end of the soil nailed terraced structure were partly grouted into the sandstone. This would have contributed to anchorage stability of the structure.

• COMMENT

A. Zhusupbekov
(Technical University, Temirtau, C.I.S.)

As a result of experimental data a new system of soil bearing capacity increase was suggested due to changing stress and strain (horizontal tensor component). It is achieved by soil pressurizing by grooves with used special technology in former USSR (Golly et al.1983; Golly & Zhusupbekov, 1982). The idea of this reinforced system is to adjust bearing capacity of retaining walls by anchored them by excavations of underground works.

The suggested system (Figure 1) is applied in the following way. Groove 1 is rammed down the soil massif 2. It is supported by cable 3 bended blocks 4.

Another groove 1 with blocks 4 (bended by cable 3) sited of the horizontal line is set opposite each groove 1. Subload 5 is sited on winch platform 6. The number of blocks 4 correspond to the designed number of reinforcement on a vertical line. Ends of cable 3 are fixed on winch 7. As a result of cable 3 stretching by alternative winches 7 switching on there appears a slot in soil. After dropping cable 3 to the block 2 level, cable 3 is disjoined from winch 7 and is restreched by grooves. This system makes it possible to adjust bearing capacity of anchor timbering by retaining walls(Zhusupbekov, 1989). The mechanics of the interaction with massif of soil for new anchored retaining walls are different from traditional anchored retaining walls.

The corresponding results for laboratory tests(Figure 2) show that relative bearing capacity of anchored model of retaining wall assembling by the suggested system is 2 times more than that of the known system.

Thus, economic efficiency and expediency of the suggested system is evudent: it promotes increase bearing capacity of vertical anchor; anchor stretching control and mechanism of technology are simplified due to cheap mechanization. Thus, the results of this investigations confirm effect of horizontal stress. This new suggested system is especially effective

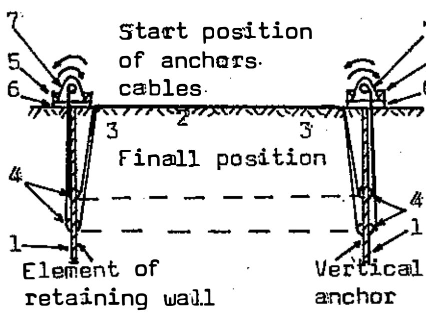

Fig. 1 Technology of anchored wall: 1-grooves; 2-soil; 3-cable; 4-blocks; 5-subload; 6-platform; 7 - winches

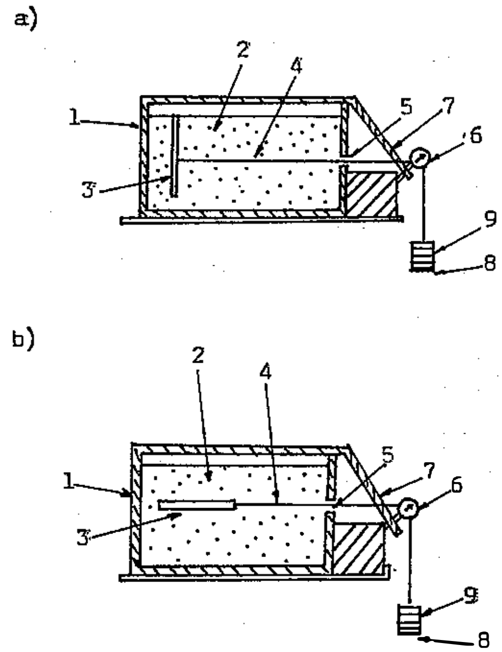

Fig. 2 Experimental installation for anchored models: 1-tray; 2- soil; 3-brass pipe; 4-steel string; 5-opening; 6-flexometer; 7-corbel; 8-plats; 9-loads

for anchored of retaining walls by constructing and underground building or works by excavations.

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1) A.Golly, V.Shatunov and A.Zhusupbekov (1983). Foundation bearing capacity increase by horizontal stress change, Proc., of Leningrad Civil Engineering Institute, Sankt-Petersburg, CIS, pp. 40 - 46.

2) A.Golly and A.Zhusupbekov(1982). System of retaining wall assembling in soil. Author Certificate No.903471, Invention Bulletin No.5 of USSR.

3) A.Zhusupbekov (1989). Bearing capacity of season - freezing and thawing soils, Proc., 10-th International Conference on Ocean Eng., Lulea, Sweden, pp. 953-961.

• COMMENT

M.C.R. Davies
(*University of Wales, Cardiff, U.K.*)
R.A. Jewell
(*Geosyntec Consultants, Belgium*)

When soil reinforcement (or a soil nail) is positioned across a potential slip surface in the ground, the strain and deformation in the shearing soil mobilises axial stresses (either tension or compression, depending on the orientation of the bar) and bending moments in the reinforcement. Sophisticated testing in large scale, laboratory shear apparatus has been carried out in recent years to investigate this interaction between soil and the reinforcement it contains. Two major series of experiments were completed by the research groups at Oxford University and at the University of Wales, Cardiff.

The purpose of this discussion is to present data from recent tests carried out at Cardiff, which have shed more light on, as well as confirming, previous observations. The practical consequences of these findings are then discussed.

The direct shear apparatus at Oxford tests a 1m x 1m x 1m sample, and a normal load is applied to the sample perpendicular to the central shear plane, a conventional configuration for direct shear testing (Pedley et al, 1990, and Pedley, 1990). A drawback of the test, however, is that only a relatively short length of the reinforcement is available to mobilise bond on either side of the shear plane, and this limits the magnitude of axial force that can be developed. Although, much greater axial forces can be mobilised when end anchors are added to the reinforcement.

The large shear apparatus at Cardiff was designed to address this problem and to allow reinforcement bars up to 2.8m length to be tested, crossing a 1.5m x 1.5m shear plane (Barr et al, 1991 and Jacobs, 1993). In order to model the stresses when a soil nail is placed towards the top a steep slope, beneath horizontal ground, the overall stress in the direction of shear is controlled using an airbag arrangement. This choice of boundary condition (with no direct control on the stress applied normal to the shear surface) makes the test difficult to interpret simply in terms of the measured boundary forces. However, the development of stresses in the reinforcement is recorded from strain gauge measurements to give the mobilised axial force and bending moment.

The results for one series of tests in the Cardiff apparatus on an unreinforced sample, and with the sample reinforced by three, circular steel bars of diameter, D = 25mm, placed perpendic-ularly across the shear plane, are shown in Figure 1. The plane bar was used directly in one reinforced test ("smooth" bar) with the test repeated using the same bar but with a layer of sand glued to the bar surface ("rough" bar).

The measured volume change in the samples indicated that the peak rate of dilation was achieved at a shear displacement of the order 40mm, coinciding closely to the point of maximum shear resistance measured in the unreinforced test (Figure 1). The interpretation for the unreinforced test gives a peak angle of friction of 42 degrees (Jacobs, 1993).

The measured axial force and maximum bending moment in the reinforced tests are shown plotted in Figures 2 and 3. The maximum bending moment, and hence the maximum shear force in the bar, increases progressively with shear displacement in the sample. Indeed, the increase in bending moment is limited by the formation of plastic hinges in the bar, which occurred at a shear displacement of the order 120mm in both tests.

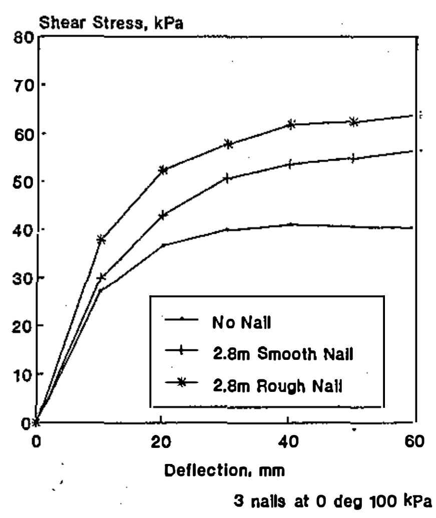

Fig. 1 Variation of nail interface

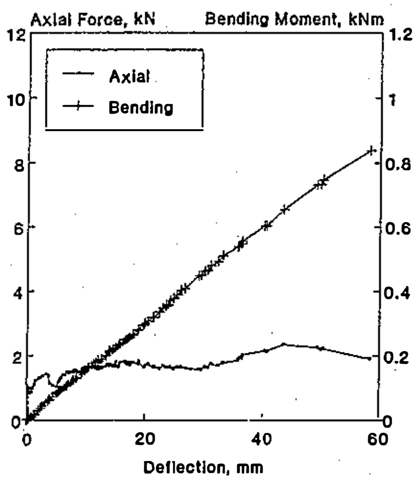

Fig. 2 Development of axial force and bending moment in "smooth" nail

In comparison, the axial force in the bars develops much more rapidly, reaching a peak at a shear displacement of about 5mm and 20mm for the smooth and rough bars, respectively (Figures 2 and 3). Axial pullout of the rough bar is shown clearly by the reduction in axial force which occurs due to strain softening in the sand as the relative displacement between the bar and the surrounding soil increases (Figure 3). Pullout also occurs with the smooth bar, but at a shear displacement of around 5mm. The friction mobilised by direct slippage between the soil particles and the smooth steel surface of the bar remains approximately constant as pullout progresses.

In both reinforced tests, a stage of limiting shear strength is reached at a shear displacement of the order of 40mm (Figure 1), corresponding to the peak strength in the unreinforced sample. It is reasonable to take this magnitude of shear displacement as a practical design limit state, beyond which unserviceable behaviour would be likely due to excessive ground distortion.

Peak axial force is mobilised at an earlier stage of the test, at a shear displacement of the order 20mm in the test with the rough bar. The measured values of maximum axial force and maximum shear force (deduced from the bending moments) in the reinforcement of these stages of the tests are summarised in Table 1.

To allow comparison with results from other tests, and theoretical analysis, it is helpful to compare the maximum axial and shear forces mobilised in the reinforcement with the fully plastic axial capacity of the bar, $T_p = \pi D^2 \sigma_y/4 = 295$ kN where $\sigma_y = 600$ kN/m^2 for the bright steel bars (Table 1).

The results in Table 1 are similar in to those published previously from the tests in other shear apparatus. They indicate that the full axial (or pullout) force in the reinforcement is mobilised at an early stage of shearing, when very little shear force is mobilised in the reinforcement. Shown dramatically by the data in Table 1, is the importance of bond between the reinforcement and the soil; the rough bar mobilises of the order 6 times more axial force than the smooth bar. However, even for the rough bar, the maximum force mobilised is only 3.8% of the full axial capacity of the bar. As well as lengthening the bar to mobilise a greater proportion of the available axial capacity, the bar may be grouted into the ground, ensuring both a "rough" contact and a larger surface area of contact.

To mobilise shear force in the bar requires significant shear displacement. The absolute limiting shear force that was mobilised in the bars, once plastic hinges had developed at a shear displacement of the order 120mm, was $(T_s)_{max} = 7.1$

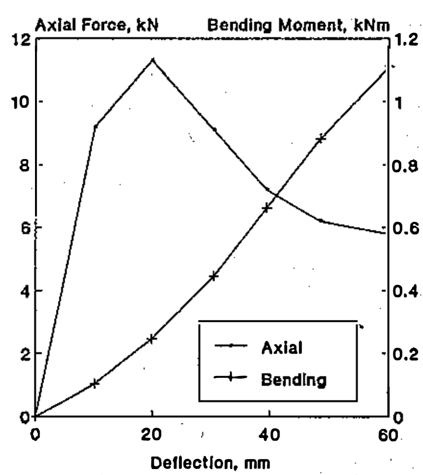

Fig. 3 Development of Axial force and bending moment in "rough" nail

Table 1. Comparison of forces measured in tests on "smooth" and "rough" soil nails at a shear displacement 20mm and 40mm.

Reinforcement case	"smooth"	"smooth"	"rough"	"rough"
Shear displacement (mm)	20	40	20	40
Axial force, T_{ax} (kN)	1.9	2.3	11.2	7.2
Axial force ratio, T_{ax}/T_p	0.006	0.008	0.038	0.024
Maximum moment (kN/m)	.28	.61	.24	.65
Maximum shear force, T_s (kN)	1.4	3.1	1.2	3.3
Shear force ratio, T_s/T_p	0.005	0.011	0.004	0.011

kN, or 2.4% of the axial capacity of the bar. This relatively poor utilisation in shear of the capacity of a solid circular bar is a direct function of the action of bending in the bar. Further, only a proportion of this maximum shear force can be mobilised at shear displacements corresponding to a serviceable structure. Typical values at "failure" in the reinforced soil are in the range $T_s/T_p \leq 3\%$. The measured maximum shear force in these tests corresponds closely with those measured by others.

The plastic analysis by Jewell and Pedley (1992) indicates that the maximum shear force that can be mobilised in a circular reinforcement bar is,

$$T_s/T_p = 0.85 / (l_s/D)$$

where l_s is the distance between the points of maximum bending moment in the reinforcement bar (on either side of the shear plane in the soil), and D is the bar diameter. The typical ratio l_s/D measured in direct shear tests on reinforced soil is in the range 15 to 30, which suggests a limiting maximum shear force in the range T_s/T_p = 3% to 6%. Only about one third to one half of this maximum shear force would be expected to have been mobilised at "failure" in the reinforced soil.

Previous shear testing gives results that fall within this range, Pedley et al, (1990). The results from the Cardiff tests summarised in Table 1 also fall within the range, as do the data represented by Hayashi et el., (1992) to this conference (a measured limiting shear force T_s/T_p < 4%, Hayashi, 1992).

These and other published data on soil nailing lead to the following practical conclusions.

1. The absolute magnitude of shear force, T_s, that can be mobilised in a soil nail is limited to a small proportion of the axial capacity of the bar, T_p. The test data and theoretical analysis indicates a maximum likely limit of the order T_s/T_p < 10%, or less. To mobilise shear force in reinforcement requires relatively large shear displacements which might not be compatible with a serviceable structure. Thus it is unlikely that in practical soil nailing cases (steel bars up to about 40mm diameter) that a shear force greater than about $T_s/T_p \approx 5\%$ could be assumed in a limit equilibrium design analysis.

2. The full axial capacity of a soil nail could be mobilised in axial tension, in the limit T_{ax}/T_p<100%. Axial tension is mobilised with relatively little shear deformation in the soil, and the contact between the soil and the reinforcement may be maximised by grouting the bar into the soil.

Anchoring devices may also be used to boost the magnitude of axial force mobilised in the reinforcement.

3. Thus, in structures where only small displacements can be tolerated during construction, such as steep or vertical excavations formed by soil nailing, the mobilised shear force in soil nails may at best be expected to make only a second order contribution to stability, a point emphasised by Gassler (1992) at this conference. The use of bars grouted into the soil is likely to be most effective in these applications.

4. Where greater soil displacement may be tolerated, and where the shear displacement in the soil is more localised across pre-existing shear surfaces, then the use of ungrouted steel bar reinforcement may be appropriate. While only a relatively small proportion of the axial capacity of each nail may be mobilised, a greater number of nails of relatively shorter length may be used, taking advantage of the rapid nail installation of ungrouted nails that can be achieved by new equipment (for example, Myles and Bridle, 1992).

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• COMMENT

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Tatsuoka (1992) comments in his Keynote paper on the benefits of the use of rigid facings for reinforced soil walls, identifying the inherent additional structural stability of these structures when compared with structures built using elemental facing systems.

The structural advantage obtained from the use of rigid facings is clearly illustrated by the revised U.K. Department of Transport design memorandum for vertical reinforced soil walls and bridge abutments. This revised design method is based upon the previous Department of Transport (1978) Design Memorandum BE3/78 drafted to consider limit state design concepts presented in the form of Limit Modes, figure 1. Limit Mode 5 covers mechanisms of failure which in the usual reinforced soil structure can pass through any elevation in the structure. In the case of structures erected with rigid facings, the potential failure planes are reduced, and must pass below the toe of the structure. Experience shows that retaining walls including reinforced soil walls, seldom fail on a plane passing through the toe. The usual critical failure plane passes through the face of the structure at a point 1/3 above the toe, and in the case of structures formed from element facing units and masonary structures occurs

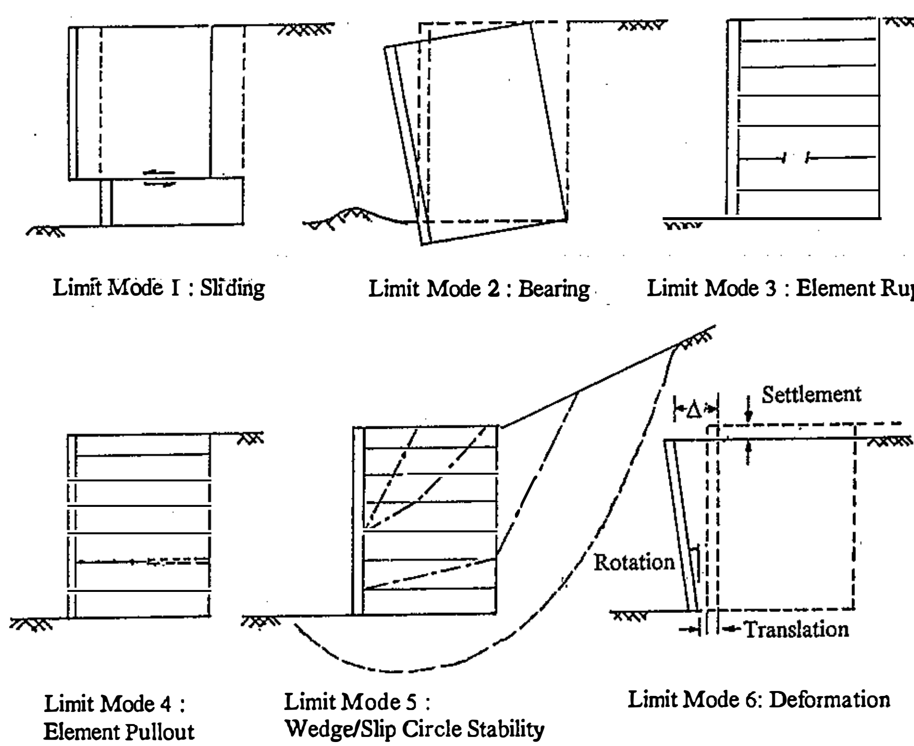


Fig. 1 Limit Modes of failure of reinforced soil structures

when the facing distorts to a point where mechanical stability is lost at which point failure is inevitable and usually very rapid, Lee et al (1993).

For a number of years the non-proprietary system used to construct vertical reinforced soil structures in the U.K. has been based upon the use of a rigid facing formed from steel H sections and concrete planks (king post and panel construction), figure 2, Jones et al (1990). This form of construction can be used with any form of reinforcement and any type of fill. It has proved to be competitive for use in the construction of retaining walls and bridge abutments.

The system presents two advantages. The use of the H section and concrete planks produces a facing, with *flexural rigidity*, thereby, providing the most stable structural form identified by Tatsuoka (1992), but with the added advantage of providing this rigidity/stability at all stages, including during construction. Secondly, the facing can be left as a king post and panel structure or provided with an additional face treatment to enhance the aesthetics of the construction. In industrial conditions, in a

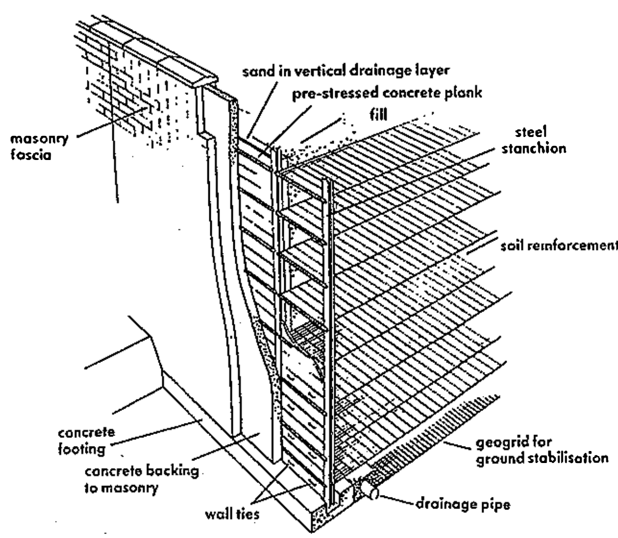


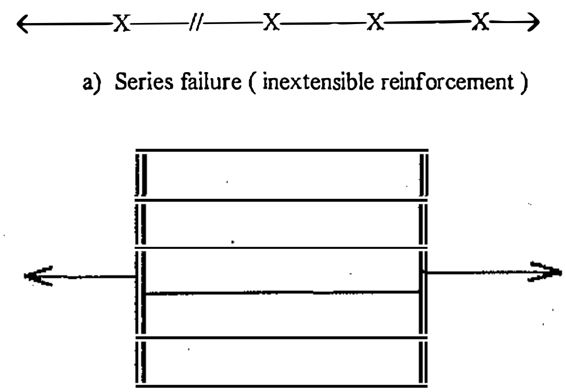
Fig.2 Use of H pile and panel facing (masonry facia is optional)

contemporary environment or with temporary structures, the appearance of a king post and panel structure may itself be acceptable. In the case of permenent structures in sensitive locations, an architectural treatment is usually appropriate. Significantly, the use of the H pile and panel method permits the construction of *economic* facings to the highest architectural standards, Jones (1992).

As the H pile and panel system is non-proprietary, any form of reinforcement can be selected by the designer. Connection of the reinforcement to the H pile and panel facing system can follow a range of methods. The most appropriate has been found to be the sliding connection which allows for settlement of the soil fill without causing additional stresses to the facing/reinforcement connections whilst providing a full strength structural connection. This provides the most stable structure whilst retaining all the structural benefits associated with flexural rigidity.

The use of rigid facings for the construction of reinforced soil walls provides no penalties in respect of construction. In the case of the H pile and panel method, it is usual practice to erect the H piles at the start of the construction sequence and to hold these in place with simple props. Propping of the facing is simple and requires very low propping forces, Jones (1985). In practice the prop is usually provided at 2/3 height and removed when the filling passes the half height of the wall. Early removal of the prop results in a small horizontal rotation of the wall face as the reinforcement strains, this results in mobilisation of the shear strength of the fill and reduces post construction movements. Where propping from in front of the structure is not practical, holding the facing back from behind the structure is the alternative accepted procedure.

One point not covered by Tatsuoka (1992) in his Keynote paper is the potential increased stability provided by the use of extensible reinforcement in a reinforced soil structure. Laboratory studies using metallic reinforcements have shown that the failure of model reinforced soil walls can be sudden if the failure mechanism is due to the rupture of metallic reinforcement, Limit Mode 3, figure 1, Bolton and Pang (1982). With metallic reinforcement, rupture of one reinforcement leads to rapid load sheading and potential overstress of adjacent reinforcements leading directly to further rupture which in turn leads to more load sheading and hence structural instability. The use of extensible reinforcement,



a) Series failure (inextensible reinforcement)

b) Parallel failure (extensible reinforcement)

Fig. 3 Anology of reinforced soil fail mechanisms

able to creep, may be inunune to this mechanism with stress redistribution being accomplished without reinforcement rupture, Jabcr (1991). An electrical analogy can be used to describe the different potential failure methods of inextensible and extensible reinforcements. The rupture and subsequent rapid failure with metallic reinforcement can be identified as being a *series system* failure where failure of one element leads directly and immediately to failure of the whole. Extensible reinforcement can be identified as being equivalent to a *parallel system*, in which total failure occurs only when all the reinforcements fail simultaneously, figure 3. If any reinforcement is overstressed, creep will occur leading to *limited* load sheading but not reinforcement rupture.

It can be concluded that the most stable reinforced soil structural form uses a rigid facing and is reinforced with a geosynthetic reinforcement.

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