Earth Reinforcement Practice, Ochiai, Hayashi & Otani (eds) © 1993 Balkema, Rotterdam, ISBN 90 5410 093 1

Discussions: Wall structures

• COMMENT

T. Matsui
(Osaka University, Japan)

My comment concerns about the effectiveness of finite element simulation to predict the reinforced soil behavior. I am first setting up a question "Can FEM be an effective tool to predict total behavior of reinforced structures ?"

In the last session of IS Kyushu Symposium in 1988, you may recall a heated discussion between Prof. Jones and Prof. Shen, on the effectiveness of finite element prediction of reinforced structures. At the end of that session, in my concluding remarks as the Chairman, I made such two comments on behavior prediction of reinforced structures that the use of computer simulation techniques could provide a basis for the development of reliable analytical methods, including more realistic mechanism, and also that some kind of FEM would have a possibility to give us a solution for the question on how to connect between combined element behavior and total behavior in reinforced structures.

In four years since then, I am very pleased to introduce you an example of Class-A prediction

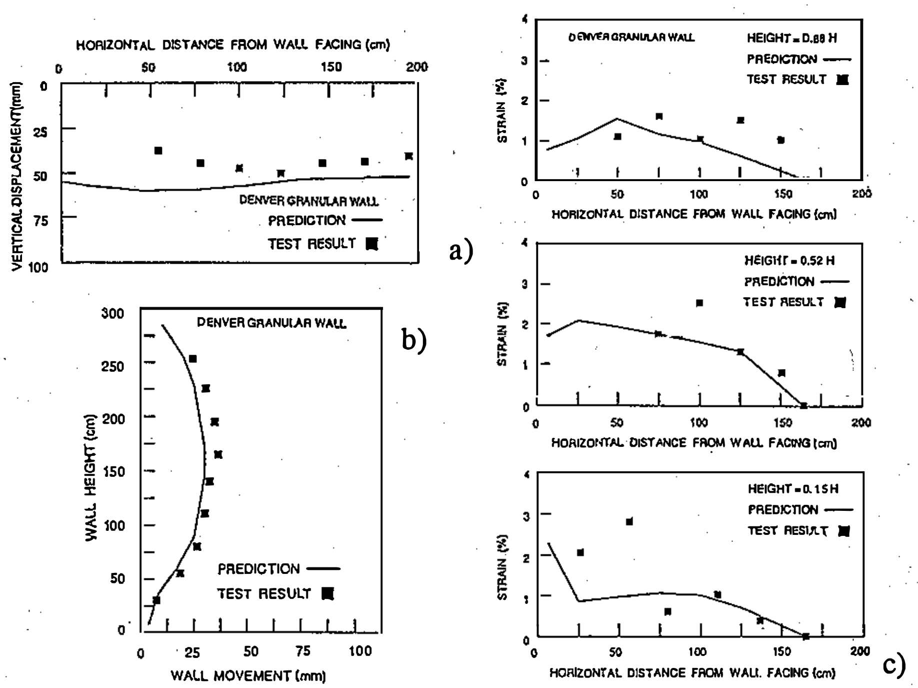

Fig.1 The comparisons between prediction and test results of a) the displacement of top fill surface, b) the movement of the wall facing and c) the axial strain distribution of reinforcement

of reinforced wall by FEM. In the last year, at the beginning of August, an International Prediction Symposium for the Class-A prediction of two geotextile - reinforced soil retaining walls with granular and cohesive backfills was held at the University of Colorado at Denver. As for the details of the Symposium, please refer to the literature (Wu, J. T. H.; Proc. of International Symposium on Geosynthetic-Reinforced Soil Retaining Walls, Denver, pp.31-42, 1992). Among 15 International predictors, the Class-A prediction made by our finite element system gave its excellent prediction. As for the details of our finite element system, please refer to our paper on page 403 in the Proceedings of this Symposium. I will show you some of the details of the prediction.

Figs. 1 a), b) and c) show the comparisons between prediction and test results of the displacement of top fill surface, the movement of the wall facing, and the axial strain distribution of the three reinforcements, respectively, at the 105 kPa surcharge loading for the granular wall. It can be seen in these figures that the excellent Class-A performance prediction was established in success. As for the Class-A failure prediction, the measured and predicted failure loads just agreed in 203 kPa, and the predicted failure mode of the reinforced wall ground also agreed reasonably with the observed one.

You should remind that these predictions were made before knowing the test results. In conclusion, the answer for my question made at the beginning of my comment is "YES".

• COMMENT

H.I. Ling and F. Tatsuoka
(*University of Tokyo, Japan*)

This is to discuss about the finite element procedure employed by San and Matsui (1992) for predicting the behavior of the Denver walls. It was difficult for the readers to follow the said paper, in which the details of several important aspects of the finite element modeling of the problem were not provided (e.g., type of finite element used for modeling soil, geosynthetic and timber facing; boundary conditions; solution technique; among others). These were not found in the original paper (Matsui and San, 1992). It is of no doubt that depending on the modeling approach, the performance of the entire simulated wall would be greatly affected. Moreover, the theoretical background on the definition of collapse load was not provided. Therefore, it remains a 'black box' to the readers although the authors claimed that they have performed the best prediction, particularly the 'exact prediction' of the collapse load.

The finite element procedure has to be well verified against the closed-form solution before it can be applied for the actual analysis. Needless to mention about the difficulties existed in analyzing one of the most sophisticated soil-structure interaction problems, such as the reinforced soil structure, the finite element cannot capture precisely the collapse load even for the simplest geotechnical bench mark problem. With reference to the studies conducted by, for example, Sloan (1981), it was known that there is a tendency for finite element to greatly overestimate the collapse load in Prandtl's solution, which assumes a smooth rigid footing resting on a weightless and purely cohesive soil. The result obtained could be so much dependent on the numerical aspects of the finite element.

Moreover, in the field problem, determination of the collapse load can be even more difficult due to the numerical instability in the finite element procedure following the development of shear bands in the soil (e.g., Siddiquee, 1991). This is particularly true for the problem with a dense or compacted soil, such as the reinforced soil retaining wall, which is dilative and exhibits a significant strain softening behavior after peak load (Tatsuoka et al., 1991).

The discussers felt that the authors' close prediction of the collapse load could be due to a balance between <u>overestimation</u> and <u>underestimation</u>. That is, overestimation could have resulted from, for instance, the nature of solution procedure and lower order finite element, while the underestimation could have resulted from not evaluating the wall friction and the drop gate used to retain the surcharge sand, among others. The possible improper modeling of the wall facing might have also influenced the results of analysis. The rigidity of the facing has a great effect on the performance of the wall (Tatsuoka, 1992), and has to be properly simulated in an analysis. If the 4-node quadrilateral elements have been used, the bending rigidity of a wall could have been underestimated.

Therefore, a good agreement in some aspects of the performance between an experiment and an analysis for a particular case history does not necessary imply that particular finite element procedure can be equally applicable to other cases unless the whole reinforced soil system has been fully modeled and verified. The discussers would also like to emphasize that more effort is still needed before finite element procedure can be reliably used for predicting the performance of reinforced soil structures.

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• COMMENT

C.J.F.P. Jones
(*University of Newcastle upon Tyne, U.K.*)

Rowe and Ho (1992) in their review of the behaviour of reinforced soil walls comment on the design methods used for reinforced soil structures. The methods identified are largely associated with the design of structures and embankments using proprietary reinforcements and do not cover the methods used in the general design of reinforced soil in parts of Europe, the United States and the United Kingdom. These methods can be classified as those based upon the coherent gravity hypothesis or the tieback hypothesis, Jones (1985). The coherent gravity method is an empirical technique which has been described by Mitchell and Villet (1987) and Minestere des Transports (1979). It was developed to cater for structures reinforced with steel strip (inextensible) reinforcements. The tieback method was developed by the U.K. Department of Transport (1978) (Memorandum BE3/78) and is based upon limit equilibrium methods. It is independent of the reinforcement material and is used with both inextensible and extensible reinforcement and with anchors. The tieback method described in Memorandum BE3/78 was revised in 1986 and is due for further major revision in 1993.

The most recent innovation in the design of reinforced soil structures in the U.K. has been the introduction of a Draft British Standard BS8006 introduced in 1991. The document has been adopted as a draft Australian Code of Practice. The British Standard (BS8006) is written in a limit state format and covers all forms of reinforced soil identified as Internally Stabilised systems with the exception of soil dowels, reticulated micro piles and special materials, figure 1. Hybrid systems such as tailed gabions are included. Any form of reinforcement is acceptable including proprietary materials. The Code covers vertical structures, embankments, cuttings and foundation problems.

The U.K. Department of Transport Design Memorandum BE3/78 covering the design of walls and bridge abutments is also being revised. Where possible this revised Memorandum relies upon the British Standard Code BS8006 but has specific limitations associated with Government Funded Structures. In particular, the analytical model used is the tieback method rather than the coherent gravity method. The revised memorandum covers reinforced soil and anchored earth (but not soil nailing), and is applicable to any form of reinforcement including anchors and grids formed from both metallic materials or geosynthetics. The revised design memorandum accepts the use of waste fills such as pulverised fuel ash.

A particular development of the new design memorandum is the use of Limit State concept. The two limit states considered are the Ultimate Limit State and the Serviceability Limit State which are defined as;

Ultimate Limit State at which a collapse mechanism forms in the ground or in the retaining structure, or when movements of the retaining structure lead to severe structural damage in other parts of the structure or in nearby structures or services.

Serviceability Limit State at which movements of the retaining structure affect the appearance or efficient use of the structure or nearby structures or services which rely on it.

The limit states are identified in terms of Limit Modes. Six Limit Modes are considered, figure 2. Of particular interest is Limit Mode 6 covering deformation which is used to check the

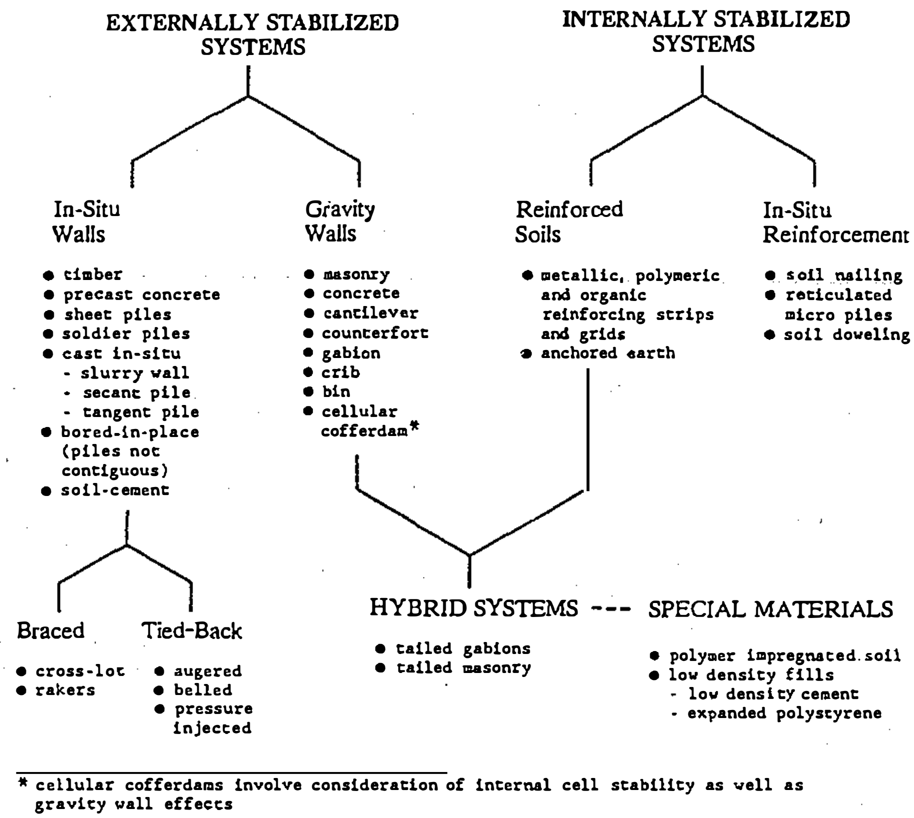

Fig.1 Classification system for Earth Retaining Structures (After O'Rourke & Jones 1990)

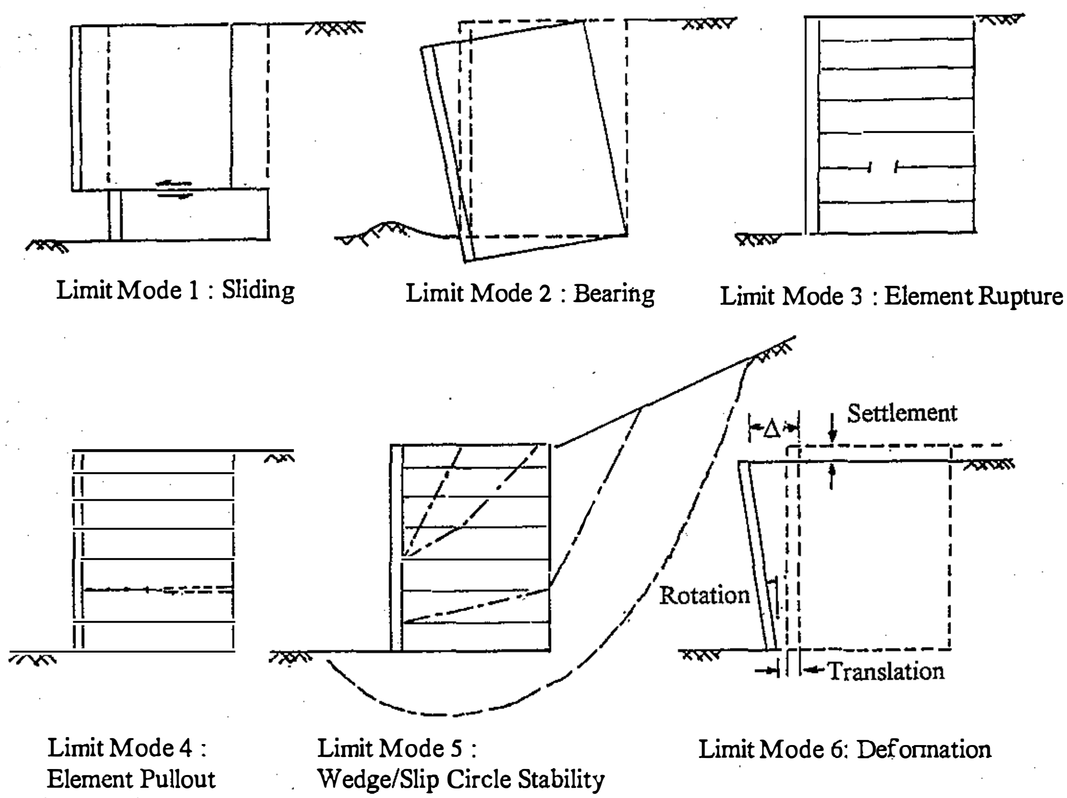

Fig.2 Limit Modes of failure

serviceability of any structure and which is also used to determine the stress-state applicable in the analysis. The analytical model used in the new design memorandum is sensitive to the form of reinforcement used in that an extensible reinforcement will lead to greater structural deflection during and post construction than an inextensible (stiff) reinforcement. The use of stiff reinforcement can result in additional stress being attracted to the reinforcement, a point implicity acknowledged in the coherent gravity hypothesis where the Ko stress state is used in the analysis.

In the revised U.K. Department of Transport design memorandum the correct stress-state of the soil for use in the analysis is obtained by adopting the following analytical sequence;

i. The geometry of the structure is chosen.

ii. The stress-state of the soil, K_{des}, is assumed equal to Ka (the active condition).

iii. Limit Modes 3 and 4 are checked and the quantity of reinforcement needed to satisfy these conditions identified.

iv. Limit Mode 6 is checked.

$$\text{If } \Delta \geq 0.0001\text{ H}$$

the selection of K_{des} = Ka is considered to be justified and the design proceeds with consideration of the remaining Limit Modes, figure 2.

$$\text{If } \Delta < 0.0001\text{ H}$$

K_{des} = Ka and consideration of Limit Modes 3 and 4 is repeated with K_{des} = Ko (at rest pressure).

The new Department of Transport strategy is unique in providing a general analytical method which acknowledges the concept of strain compatibility, the importance of reinforcement stiffness and the role of structural deformation, Jewell (1992), O'Rourke and Jones (1990).

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• QUESTION TO YAMAGAMI

Q : F. Tatsuoka
(University of Tokyo, Japan)

The authors reported a very interesting case history of a tall reinforced earth wall constructed by the French metal strip-reinforced soil retaining wall system (Terre Armee). Indeed, this paper provides a very valuable data set for further improvement of the design method of reinforced soil retaining wall systems.

Needless to say, Terre Armee assumes that metal strips function as **tensile** reinforcement as the other types of reinforced soil structures. In the FEM analysis described in the paper, the backfill soil reinforced with metal strips is modelled as a homogeneous orthotropically anisotropic material. Therefore, the equivalent Young's moduli in the vertical and horizontal directions E_1 and E_2 in Eq. (1) should be different. I consider that if sufficiently large tensile forces are activated in the reinforcement, for a given mass of tensile-reinforced soil, E_1 in the horizontal direction, in which reinforcement layers are placed, should be much larger than E_2 in the unreinforced vertical direction, or at the worst, E_1 is not much smaller than E_2.

Despite the above, Table 1 shows that the ratios **n**= E_1/E_2 which were back-analysed from the observed behaviour of the actual reinforced wall be much less than 1.0; i.e., as small as 0.0473 for No. 1 case of analysis. So my questions are as follows:

(1) For the result of the FEM analysis, I would like to know whether in the major part of the reinforced zone, strains in the horizontal directions are **tensile** or compressive. If they are compressive, the value of **n** slightly less than 1.0 may be possible. In this case, however, the reinforcement does not function as "tensile reinforcement". If they are **tensile**, the reinforcement functions as "tensile reinforcement" as assumed in the conventional design of Terre Armee. I wonder, however, why the back-analysed values of **n** are so low.

(2) If they were measured, I would like to know whether the measured global horizontal strains in the reinforced zone were **tensile.**

It is my understanding that any tensile-reinforced soil structure should be designed so that strains in the reinforcement in principle be **tensile**. This point should be ensured particularly when a vertical reinforced soil retaining wall has a relatively high slope behind the reinforced zone, since the slope may activate relatively high earth pressure on the back face of the reinforced zone which may compress the reinforcement in its lengthwise direction (i.e., the horizontal direction). In that case, the tensile reinforcement will not help directly in stabilizing the slope. From this point of view, it must have been a wise decision to give up the employment of the other cases 1, 2 and 3 for the actual construction, which have more steep slopes behind the reinforced soil retaining wall.

A : T. Yamagami
(*University of Tokushima, Japan*)

The current structure is peculiar in that it consists of a huge embankment with a complex of three stage reinforced earth walls having a maximum overall height of 38m as shown in Fig.-1. Therefore the scale of the reinforced earth walls , in particular the length of the strips have been determined by a particular stability analysis outlined in Fig.-2, in addition to the conventional method. Moreover the reinforced earth walls have been idealized as an orthotropic, homogeneous elastic body in the F.E. numerical analysis. It is thus no longer relevant to look at this structure or the numerical results from an ordinary point of view. Fig.-9 shows the behavior of a homogeneous, simple substance exclusively. In this regard, I do not think it is significant to discuss stresses or strains in the strips based on Fig.-9.

Compressive and tensile stresses are mixed in the numerical results, though the former is seen to be predominant. All the minor principal stresses are in tension especially near the bottom of the first stage of the walls.

Finally, in the F.E. analysis we have not made any assumption except that the reinforced earth walls are orthotropically elastic.

• COMMENT

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

Discussion on the paper by Yeo,K.C., Andrawes,K.Z. and Saad,M.A. (1992): "The use of a compressible boundary layer in reinforced soil structures," Proc. of Earth Reinforcement Practice, IS-Kyushu '92, Vol.1, pp.449-462.

It is shown in the paper that for reinforced soil retaining walls, the application of a compressible material at the back face of a propped facing can reduce the lateral earth pressures on the back face of the facing <u>during the construction stage</u>. This effect may be "beneficial for situations where it is desirable to minimize the loading on adjacent structural elements (e.g., basement walls, bridge abutments, etc) " (Rowe and Ho, 1992).

The use of a compressible layer would not be particularly beneficial, however, when a facing should free-stand when completed (i.e., not supported by any external measures), which is the case for most vertical reinforced soil retaining walls. It seems that this consideration is supported by the test result presented in the paper. Namely, the timber walls were constructed while reinforcements were not connected to the facing propped externally. After the full-height of wall was completed, before the propping of facing was removed, the facing was connected to the reinforcements (otherwise, the facing could not stand safely). As shown in Fig. 4c, when this connection was made presumably firmly, the earth pressure at the back face, which had been kept to a small value by the use of a compressible layer during the construction stage, increased largely inevitably. In Fig. 4c, indeed, for all the three reinforcement layers, along the reinforcement length, the tensile force became the largest, or close to it, as approaching to the facing. This relative large connection force means the corresponding increase in the earth pressure on the back face of facing. It is the discussor's view that this situation is desirable for the stability and less deformation of the wall, since the increase in the earth pressure means the increase in the confining pressure, which in turn increases the stiffness and strength of soils adjacent to the facing. Then, it seems that the use of a compressible layer is beneficial in reducing the earth pressure on the back pressure only for the construction stage, but it is not particularly after the walls are completed.

As discussed in the discussor's keynote lecture for this symposium (Tatsuoka, 1992), the discussor considers that relatively large earth pressures on the back face of facing, which could be as large as the earth pressure at rest, can be supported safely by a relatively light continuous rigid facing, when the facing is tightened to the reinforcement layers. This situation is like that a relatively thin continuous beam supported at many points with a short span can sustain relatively large spread load.

In addition, external load applied close to the facing on the crest of backfill may collapse a compressible layer further. I wonder whether this may develop undesirable deformation of the backfill. The several different methods using a full-height rigid facing for a geotextile-reinforced soil retaining wall is discussed in detail in Tatsuoka (1992).

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• QUESTION TO RIMOLDI

Q : S. Kaniraj
(Indian Institute of Technology, Delhi, India)

Rimoldi and Cambiaghi in their paper entitled "The use of geogrids in road application in Italy" have presented three case histories. In the first case history they explain the design of multi layer reinforcements for an embankment on soft clay. Equation 1 of the paper is used to determine the number of reinforcement layers and the reinforcement force in each layer. However, with only one equation and more than one unknown, it is possible to determine these values only if some unknowns are assigned predetermined values.

The reinforcement forces are calculated for a global factor of safety in eq.1. There is no need to multiply these values again by the global factor of safety in Eq.2. And, to determine the required tensile modulus the working force rather than the peak force should be used in Eq.3. The design according to Eqs. 2 and 3 of the paper will result in a very conservative design of the reinforcement.

A : P. Rimoldi
(TENAX SpA, Italy)

If we put I=1 in Eq.(1), we obtain the special case of the embankment shown in the paper. In this case with one layer the problem is solved. In general we have to arrive to the solution by a trial and error procedure in order to obtain the final design. If we need more than one layer of reinforcement, then we set a preliminary layout by experience, we make preliminary calculations and check for the global stability with this equation.

The Factor of Safety is applied to the tensile reinforcement, because it is a general method in reinforced soil design, where usually you don't refer to the ultimate strength of the reinforcement. We have to decrease the peak force by partial Factor of Safety and one of these is the global Factor of Safety which takes into account the general level of safety that we want. We can apply the Factor of Safety to the soil characteristics (Φ', C') or to the geogrid strength. To the Author's opinion, in this case it is appropriate to apply the Factor of Safety to the geogrid strength.

Usually some typical properties of the geogrid reinforcement are measured in laboratory performing tensile tests. Then we have to decrease the peak strength to obtain the allowable strength. One may say that this is quite conservative. But the project shown in our paper was a special one, with a very low-strength foundation. We did plate loading tests because we needed to perform in situ tests for checking our design. We wanted a modulus of subgrade reaction of 15 MPa (measured with the 300 mm diameter loading plate) and we obtained 17÷18 MPa, just a little above the requirement. Therefore the procedure explained in the paper is reasonably conservative, but not too conservative.

• QUESTION TO SEGRESTIN

Q : J. Paul
(*Netlon Limited, U.K.*)

It is stated that the design was carried out to the requirements of BE78. This is an extremely conservative design method and face movements as shown in the paper have never been seen in practice. Where there very unusual loading conditions, and if not, how can the author explain why the computer analysis is so far away from what is seen in practice?

A : P. Segrestin
(*Terre Armee Internationale, France*)

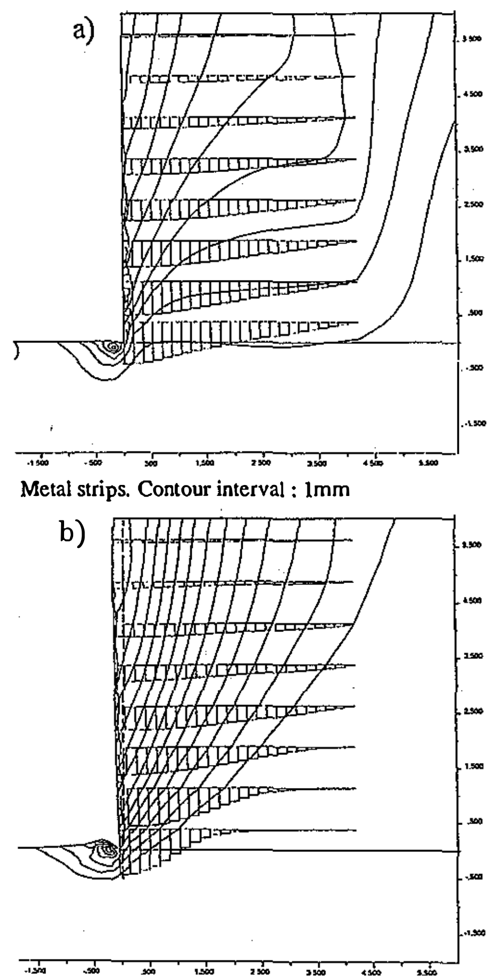
"Your FEM analysis with extensible reinforcements shows very large deformations, much larger than observed on actual structures. Did you consider any unusual loading conditions for your models ? Do you think that your conclusions can be valid without calibrating your models with actual field conditions ?"

The finite element studies were carried out with Professor Smith on models of very simple walls, without any surcharge, though "built" in stages. The models had various types of reinforcements, all designed according to the guidelines of the proprietary systems and in conformity with the design rules in force in Great Britain (BE 3/78). These rules can be considered relatively more conservative than in most other countries, which means that they lead to using more reinforcements... therefore to lesser deformations.

We do not claim that our models reflect the perfect reality (no more than others...). They do not, for instance, model the effects of compaction, no more than construction methods or "knacks" allowing to compensate in advance, on the site, a large part of the deformations resulting from the elongation of the reinforcements. The aim of our study was therefore not to predict precise deformations, but at the very most to estimate those that might have to be compensated. Its purpose was above all to compare the mechanisms following which these different types of structures behave.

We indicated in our paper that, while the qualitative observations about these behaviours were judged interesting, the numerical results of the finite elements pertaining to tensile loads in the extensible strips (hence their deformations) were deemed doubtful. We attributed this to the possible excessive rigidity of the FEM mesh.

This then lead us to resort to models based on



Metal strips. Contour interval : 1mm

Extensible straps. Contour interval : 10mm

Fig. 1. FLAC study. Horizontal displacement contours.

"finite differences", using the FLAC program developed by Itasca. During the oral presentation we gave the outlines of the results obtained since the paper was written.

This time the values of the tensile loads appear quite plausible : regardless of the type of reinforcement, whether extensible or inextensible, the maximum loads are rather similar at a same level and in keeping with what is expected. On the other hand the variation of the loads along a same layer may have a different appearance, as can be seen on fig. 1.

Of course, since extensible reinforcements elongate more than metal strips, the outcoming deformations at the facing are more significant. In this example the facing deflection, as a ratio of its height, is 0.13 % for metal strips and 1.90 % (15 times more) for extensible reinforcements. As mentioned above, this concerns the deformations to be compensated during construction, as far as possible.

These values correspond to extensible reinforcements having stiffnesses of 0.45 to 1.80 MN/m, depending on layers. With regard to this, it should be noted that the stiffness values used in our models are those found in the manufacturers' technical sheets. Should they consider that these values are not representative of the actual behaviour of their products, for instance in a confined environment, it is up to them to provide other figures, with the necessary substantiation.

• QUESTION TO SMITH & SEGRESTIN

Q : J.G. Collin
(Tensar Earth Technologies, U.S.A.)

Finite Element Analysis as we have heard from Dr. Jewell is a powerful tool when used correctly. However, for meaningful results it is imperative that one correctly model the boundary conditions. In the authors analysis the wall facing was modelled with the same properties as the soil. This is unrealistic for reinforced soil walls with perhaps the exception of a wrapped face wall. The unrealistic lateral displacements predicted from the analysis 92.5mm for an eight(8) meter high wall clearly indicate modelling problems.

However, the authors draw vast conclusion from this analysis. To draw conclusions about the actual field performance of inextensible and/or extensible reinforcements from uncalibrated finite element analysis can be very misleading as the FEM results may have no correlation to reality.

A : I.M. Smith
(University of Manchester, U.K.)

Several types of facing were analysed using thin elements adjacent to the wall to capture interface effects. While the nature of the facing is significant it is not "critical" in displaying the essential differences between inextensibly and extensibly reinforced walls. In this short presentation a "soft" facing was assumed so as to permit a clear comparison between the two types of system unclouded by (difficult to quantify) facing effects.

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

The "soft" type of facing used in this study is similar to the one used (among others) in a previous extensive parametric F.E.M. study carried out by T.A.I. in 1982-83 (100 models of walls and bridge abutments). This model had lead to very satisfactory results, compared to the measurements obtained from the monitoring of actual Reinforced Earth® structures, faced with flexible semi-elliptical steel units.

One advantage of the soft type of facing, in a comparative study, is that the results are not affected by parasitical issues, such as the "panel-effects" brought to the fore on fig. 1. They can therefore be analysed more easily.

Fig. 1 also shows on the other hand that there are no fundamental differences between the behaviour of structures with "soft" or "discrete" facings, as far as essential things such as maximum tensile loads are concerned.

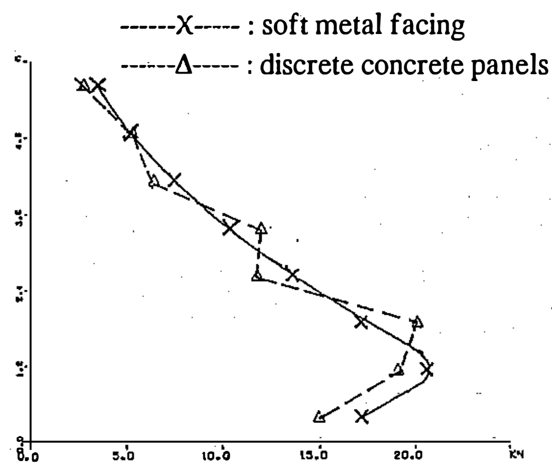


Fig.1. F.E.M. study, TAI 1982/83 - 6.0m high Reinforced Earth wall with 4.2m long steel strips. Maximum tensile load as a function of depth.

• COMMENT

C.J.F.P.Jones
(*University of Newcastle upon Tyne, U.K.*)

The main problem that the designer faces when attempting to use the finite element method in the design of a reinforced soil structure is that he or she has no knowledge of the fill material that will be used in the actual structure. As a result, any finite element analysis has to be based upon assumed material properties. The chance of the analysis being realistic are slim.

If details of the fill material to be used are known, as was the case in the Denver wall experiment, the use of the finite element method can be justified and can be shown to be a very powerful aid to the designer.

• COMMENT

R.A. Jewell
(*Geosyntec Consultants, Belgium*)

1 Introduction

Stability and deformation should both be considered in the limit analysis design of reinforced soil walls, particularly as deformation can be a critical aspect of the behaviour for polymer reinforcement materials. This creates a need for simple but rational analyses of wall deformation that are amenable for use in design.

While the finite element method is a suitable analysis for the calculation of wall displacements, it is still too complex a method for general use in design. The uncertainty at the preliminary design stage as to the exact soil fill that will be used, the type and properties of the reinforcement that will be selected, even the exact geometry of the structure, and the magnitude and location of any external loading that might be applied, all call for a relatively simple calculation method that can give a reasonable prediction of the order of wall displacement that should be expected.

This discussion draws attention to one such method that is available, and which was introduced to the previous conference in Kyushu (Jewell, 1988). Application of the method is illustrated by predicting the horizontal wall displacements for various typical wall examples that have been studied using the finite element method and reported to this conference (Kaliakin and Xi, 1992; Rowe and Ho, 1992; Smith and Segrestin, 1992).

2 Back-analysis

An equilibrium analysis of the reinforced soil wall is used to determine the magnitude and variation of force along each reinforcement layer. The load-extension properties of the reinforcement material are then used to calculate the horizontal displacement of the face of the wall (Jewell, 1987). The results for any reinforcement spacing arrangement are summarised in simple non-dimensional charts.

The same analysis gives the incremental horizontal movement that results from the application of uniform vertical surcharge to the wall, or from creep extension of the reinforcement due to sustained load.

The method has been developed further to provide solutions for the prediction of vertical settlement (as well as horizontal displacement), by associating the Mohr circle of plastic incremental strain with the state of stress in the reinforced soil wall (Jewell and Milligan, 1989). A simple chart for the maximum value of horizontal displacement, δ_{max}, for uniformly

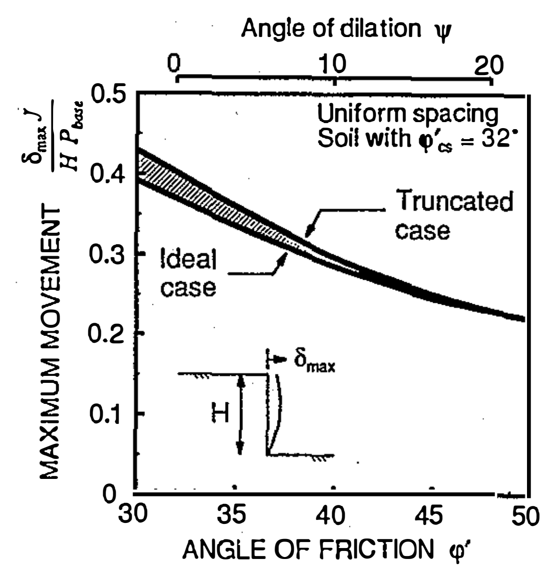


Figure 1

Table 1. Results of back-analysis and comparison with FE analysis.

Parameter	units	Kaliakin & Xi	Rowe & Ho	Smith & Segrestin
H	m	10	6	7.5
s_v	m	1	1	0.75
J	kN/m	600	2000	800 [1]
γ	kN/mm^3	18	20	20
ϕ'	°	35	35	36
$\frac{\delta_{max}J}{HP_{base}}$		0.35	0.35	0.35
P_{base}	kN/m	48.6	32	29
δ_{max} (from chart)	mm	280	34	95
δ_{max} (FE analysis)	mm	275	32	92.5

Notes: 1 Analysis for 2x110 kN/m layers

spaced reinforcement arrangements was derived and is shown in Figure 1 (Jewell, 1990). This chart is used for the back-analyses below. The analysis requires one number to be calculated and one number to be read from the chart.

The parameters for the analysis are the wall height, H, any uniform vertical surcharge on the wall, q_v, the angle of friction, ϕ', and the unit weight, γ, of the soil, and, the uniform reinforcement spacing, s_v, and the secant stiffness for the reinforcement, $J_{t,T}$, relevant to the time under sustained load at the relevant ambient temperature. To allow for creep extension, the analysis should be completed twice using the secant stiffness of the reinforcement relevant to the beginning and end of the period of sustained load (often between the end of construction and the end of the design life of the wall). The creep displacement is the difference between the two.

The number to be calculated is a nominal force for a reinforcement layer at the base of the wall,

$$P_{base} = s_v(\gamma H + q_v)\frac{(1-\sin\phi')}{(1+\sin\phi')} \quad \ldots(1)$$

The number found from the chart (Figure 1) is the non-dimensional measure of maximum horizontal displacement, δ_{max}/H, which is a function of the maximum extension in the reinforcement, P_{base}/J, and the angle of friction of the fill, ϕ'.

3 Results

The results for the maximum horizontal deformation in the walls found using the chart in Figure 1 are summarised in Table 1. The calculated maximum displacement is seen to be in close agreement in all three cases with the displacement found by the respective Authors using finite element analysis.

Rowe and Ho (1992) repeated the analysis for their wall example, but assuming an angle of friction in the soil, 40° and 45°. This reduced the calculated maximum wall displacement to, $27mm$ and $20mm$, respectively; the corresponding values found from Figure 1 are, $23mm$ and $15mm$, respectively.

4 Conclusions

The aim of this discussion has been to illustrate that a simple method of analysis for the deformation of reinforced soil walls can provide a rational basis for the prediction of wall deformation. This has been illustrated in the discussion by comparisons with numerical analyses for the walls.

The analysis allows quick hand calculations for assessing likely wall displacements and is well suited to practical use in design.

5 References

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