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Discussions: Embankments

• QUESTION TO GOURC

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

My Question relates to Prof. Gourc's keynote lecture. I would like to congratulate it is indeed a very good lecture. One thing that interested me was those geogrid reinforced cells that he used in one of the projects to reinforce the base of the embankment. And I would like just to have the information from it and was it effective to reduce the lateral deflection or displacement of the embankment and what was the magnitude of those displacement and then secondly was it found to be cost effective.

A: J.P. Gourc
(*Uniersity of Grenoble, France*)

Unfortunately I have not got some of the details about the application but I think that it will be necessary for next time. I think it will be just possible to say now we will obtain with this technique a good mattress with a good rigidity. It is the reinforcement of the best quality of the embankment of course, but with big amount of geo-cell. I have also some question as to this kind of research. My questions are first how to design this kind of reinforcement, how to take into account the value of showing resistance with this kind of reinforcement in a global or overall equilibrium method. That is the first question for me and the second is we have to compare this kind of reinforcement with another conventional one with sheets and in the case it is possible to compare the efficiency of the two techniques with the same value of the area of geogrids. But in the present time I don't have the answer to this question, if some person in this floor can get answer to this question, it is very interesting because the paper I read on this question furnish only some details on the construction but not on the results.

Q : M. Bolton
(*Cambridge University, U.K.*)

Thank you, would anybody like to take Prof. Gourc's invitation and give us more informations on the success of this technique.

A : J.T.H Wu
(*University of Colorado, Denver, U.S.A.*)

I believe that Prof. Duncan, BPI stands on analytical studies as well as field tests on these geogrid mattresses. My general impression is that it is not cost effective.

Q : M. Bolton

Perhaps Mr. J. Paul might to say something.

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

The answer to the first question is that research work carried out in the USA showed that the cellular structure was approximately 70% more effective than the equivalent area of reinforcement in horizontal layers.

The system has also proven, on many occasions, to be extremely cost effective. While the construction of the Geocell may appear to be labour intensive a small team of 3 men can erect up to 500m²/day which is usually faster than the filling operation.

The design method has been published (The use of slip line fields to assess improvement in bearing capacity of soft ground given by a cellular foundation mattress, installed at the base of an embankment, C.G. Jenner, D.I. Bush and R.H. Bassett. Internat. Geotech. Symp. on Theory and Practice of Earth Reinforcement, Fukuoka, Japan, 5-7 October 1988) but a research project is currently taking place which may lead to refinements of the method.

• QUESTION TO GENSUKE

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

Q1 : Fig. 3 (page 236 in the proceedings, Volume I) shows the 2-block failure mechanism-earthquake loading conditions. The wall body with its own weight W is subjected to the inertia force (a/g)W. Then, the wall body moves forward direction. The backfill is stretched horizontally, and the horizontal component of earth pressure decreases accordingly. The right figure illustrates Ea is positive. I would like to know why Ea is positive.
Q2 : We measured accelerations of retaining walls during earthquake. Horizontal components of accelerations are not the same at the bottom and at the top. One example, bottom 100 gals and top 200 gals, retaining wall 5 m high. What the design value of acceleration do you suggest to use?

A : D.D. Gensuke
(Deutsche Montan Technologie-IWB, Germany)

Thank you very much Professor Fukuoka for this most interesting comment. You hit two important points: the possibility of an increase or a decrease of earth pressure of the unreinforced block of the failure model during an earthquake and the question about the distribution of the horizontal earthquake acceleration (a) over the height of the wall. As to the second aspect we have to admit that we only used a simplified approach and thus did not include the change of the horizontal earthquake acceleration with the height. This certainly should be considered for future research on this topic. In our paper we intended to demonstrate the probabilistic safety concept for the case of a reinforced wall subjected to earthquake loading. It was just a first step that certainly calls for improvement. As to the first aspect, the possibility of a decreasing earth pressure, we admit that a decreasing earth pressure was not considered. We do think, however, that for design purpose the most unfavorable case is the relevant one. Thus, by only considering an increasing earth pressure we are on the safe side. We hope that this will answer you questions and thank you once again for the valuable discussion you kicked off.

• COMMENT

J.T.H. Wu
(University of Colorado, Denver, U.S.A.)

The interface element formulated based on the method of stiffness has been widely used in finite element analysis of soil-structure interaction problems to allow for relative movement between dis-similar materials (such as between backfill and reinforcement and between backfill and facing). I heard a number of presentations this morning in which this type of interface element was used, and I expect to hear a few more applications before the end of the Symposium. I am compelled to point out two serious problems associated with this method of simulation.

In the method of stiffness, the behavior of the interface between dis-similar materials is typically represented by the stiffness associated with two sets of springs--one in the direction normal to the interface (k_n) and the other in the tangent direction (k_s). The first serious problem is regarding determination of the values of k_n and k_s. Since the formulation typically simulate the interface as an assembly of "segments" (with zero initial thickness), the value of k_s shall be a function of the relative displacement between two contact segments, and is commonly determined by direct shear tests or pullout tests. However, as shown in Figure 1, the slopes of the curve (i.e., k_s) is clearly a function of the specimen size, among other variables. This implies that correct values of k_s must be deduced from an interface shear test of which the specimen size be equal to the length of the interface segment in the finite element discretization. This problem renders the determination of the interface properties impractical, especially when the finite element discretization of the interface is not uniform.

The other serious problem with the method of stiffness is related to numerical difficulties. Since the correct interface state is not known (whether it will remain in contact, slip, or separate) before a load increment is applied. An iterative procedure is required. This is generally

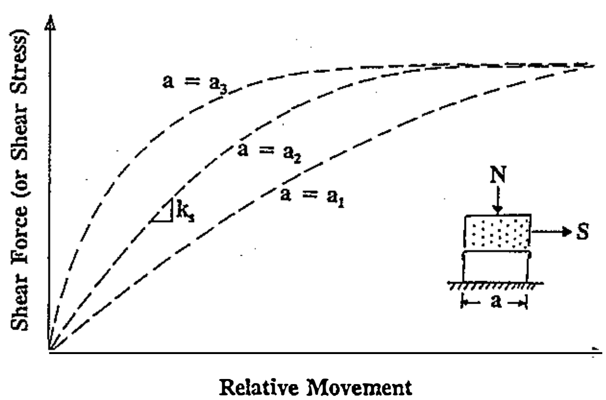

Figure 1. Interface Shear Test Results with Different Speicimen Sizes

accomplished by assuming that the contacting segments will remain in contact after a prescribed load application in the first iteration. The assumption of remaining in contact is performed by assuming "large" value of k_n in the analysis (when the normal stress is compressive). If the value of k_n is not large enough, penetration of the contacting nodes will occur--which would be kinematically inadmissible. On the other hand, if the value of k_n is very large (say, 10^{10} lb/in), the significant digits of the "penetration" may be truncated, hence the resulting normal stress at the interface may be seriously in error. Keep in mind that a correct value of the normal stress is needed to determine whether interface slippage will indeed occur.

My suggestion for simulation of interface behavior in finite element analysis is as follows:

1. In the cases where relative movement at the interface is judged unlikely or unimportant, the analysis should be performed with fixed interface condition, i.e., interfacc clcmcnt should not be used.

2. If relative movement at the interface is deemed critical, yet the only available interface element in the finite element code is based on the method of stiffness, analyses with two extreme interface conditions should be performed--one with fixed interface and the other with frictionless interface (i.e., with zero friction angle or with the value of k_s being zero). The two solutions will provide an upper and a lower bound between which the actual interface condition will lie.

3. If relative movement at the interface is considered critical and if one is really serious about simulating the correct interface behavior, one should consider using the interface element proposed by Katona, et al, 1976 and Helwany and Wu, 1987.

REFERENCES

1] M. G. Katona, J. M. Smith, R. S. Odello, and J. R. Allgood (1976). CANDE - A Modern Approach for Structural Design and Analysis of Buried Culverts. Report No. FHWA-RD-77-5, Naval Civil Engineering Laboratory.

2] B. M. Helwany and J. T. H. Wu (1987). Numerical Simulation of Soil-Geotextile Interaction in Pullout Tests. Geosynthetic Research Report No. GR-87-03, Department of Civil Engineering, University of Colorado at Denver.

• QUESTION TO OIKAWA

Q : M.R. Madhav
(Indian Institute of Technology, Kanpur, India)

Reinforcement provided at the base of an embankment has several functions. In case of embankments on soft soil, the reinforcement prevents lateral displacements of the embankmnet and foundation soils. Consequently, the bearing capacity of the soft soil and the stability of the embankment are increased significantly. If the foundation soil is soft or very soft, it needs to be improved by sand, granular, lime or DJM piles. In case of a reinforced embankment on improved ground, the function of the reinforcement at the base of the embankment is to redistribute the embankment load to achieve uniform settlements.

In the case study reported by Oikawa et al(1992) to this symposium, embankment No.2 is built on fully sand pile treated soil. The reinforcement forces are small indicating the second type of function mentioned. The embankment appears to be functioning as a shear beam and bends concave upwards. As a result the tensile forces are larger in the lower layer than in the upper layer of reinforcement. In contrast, embankment No.2 is built with only the central portion beneath the embankment reinforced and stiffenned by sand piles. It appears to have bent concave downwards thus generating higher tensile forces in the upper layer of reinforcement than in the lower layer. The results presented seem to be consistent.

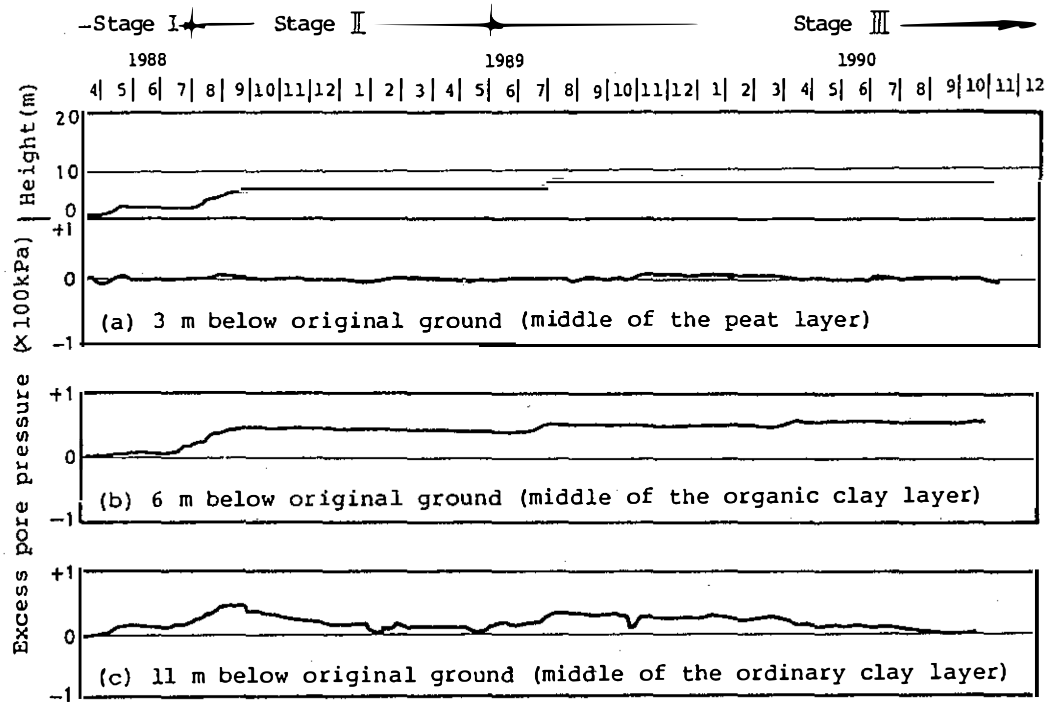

Fig. 1 Excess pore pressure variations under the right beam

A : H. Oikawa
(*Akita University, Japan*)

The excess hydraulic pressure measured at three points under the right berm of the embankment No.5 are shown in Fig 1. The followings can be seen in the figure: (1) the excess hydraulic pressure at 3 m below the original ground, where is the middle of the peat layer, dissipated almost instantaneously and indicated almost zero during the whole period of the execution; (2) the excess hydraulic pressure at 6 m below the original ground, where is the middle of the organic clay layer, never dissipated; and (3) the excess hydraulic pressure at 11 m below the original ground, where is the middle of the ordinary clay layer, developed quickly during construction and dissipated quickly during a rest of filling. Almost same trends were observed in the layers under the central fill and under the left berm. Referring to the behaviour of excess hydraulic pressure within the organic clay layer, two possible reasons may be considered: the first is the sealing effect of the upper peat layer and lower clay layer; that is, the rapid dissipation of excess hydraulic pressures and consolidation that occurred within the peat layer and within the ordinary clay layer was accompanied by a decrease in permeability and, thus, the dissipation of excess hydraulic pressure from the organic clay layer was inhibited; the second is that the compressive-type failure occurred within the organic clay layer had reduced the rate of dissipation of excess hydraulic pressure from its layer. However, no definite answer with respect to this matter could not be obtained yet.

By the way, referring to the higher mobilization of the upper net reinforcement comparing with the lower one (Fig.11 in the Proceedings), I can't explain the reason. The interpretation of this phenomenon requires further investigations.