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Issues in the use of clay in reinforced earth construction

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ABSTRACT: The use of clay fill in the construction of reinforced earth walls involved substantial cost savings, and is being increasingly adopted by geotechnical engineers in New Zealand. This paper looks at some of the issues this raises, in the New Zealand context, especially the following:
(1) The appropriate soil strength parameters to be used in design, ie the choice of the peak value (ϕ'_p), the critical state or constant volume value (ϕ'_{cv}), or the residual value (ϕ'_r).
(2) The expected deformations which may occur at the wall facing, or elsewhere, either during construction, or after completion, because of the lower stiffness of clay compared to granular material.
(3) Possible adverse effects from excessive down-drag on the facing elements. The increased difference in stiffness between the facing and the clay fill could lead to excessive vertical load on the facing.
(4) Compaction difficulties and the possible development of pore pressures during construction.

1 INTRODUCTION

The advent of new reinforcing materials such as geotextiles and geogrids has made possible the use of clay fill for reinforced earth (R.E.) walls. The cost reductions associated with the use of clay has made its use increasingly attractive This paper looks at several issues which arise from the use of clay.

2 SOIL STRENGTH PARAMETERS

2.1 *General*

There are differing opinions among geotechnical engineers as to whether the peak, critical state, or residual strength should be used for the design of reinforced walls and slopes. For example, Zornberg et al (1998), describe model tests which show that failure is clearly governed by the peak value, and argue strongly (Zornberg et al. 2000) for its use in design. Leshchinsky (2000) proposes a hybrid design procedure in which the peak value is used to determine the critical slip surface, but the residual value is used in determining the grid anchor length. The current Netlon (1996) guide for the design of reinforced earth structures using geogrids suggests that the critical state parameter ϕ'_{cv} is the appropriate parameter. (The suffix cv denotes "constant volume"; the term "ϕ' critical", or ϕ'_{crit} is also used to designate the critical state angle, and is the same as ϕ'_{cv}). The Netlon guide also quotes the UK Dept. of Transport Advice Note HA 68/94 (1994), which recommends the use of the residual strength (ϕ'_r) for clays having Plasticity Index values over 25.

In New Zealand, the appearance of documents and codes for reinforced earth which recommend the use of ϕ'_{cv} has given rise to some mild unease and "puzzlement" amongst geotechnical engineers. Critical state soil mechanics has not made much impression on the practising geotechnical community in New Zealand, and reinforced earth design in the past, along with all other effective stress analysis, has been carried out using the familiar Mohr Coulomb peak parameters c' and ϕ'.

Together with the issue of the ϕ' parameter for use in design has been an interest in building reinforced earth walls using local clays, because of their ready availability. The choice of ϕ' parameter becomes more important with clays because the peak value is lower and the difference between peak, critical state, and residual values may be greater than for granular materials. A short research programme has therefore been carried out to investigate the parameter ϕ'_{cv} for some typical local soils (primarily of a cohesive nature), and try to establish its relationship with the standard "peak" Mohr Coulomb parameter ϕ'.

Table 1. Details of samples and test results.

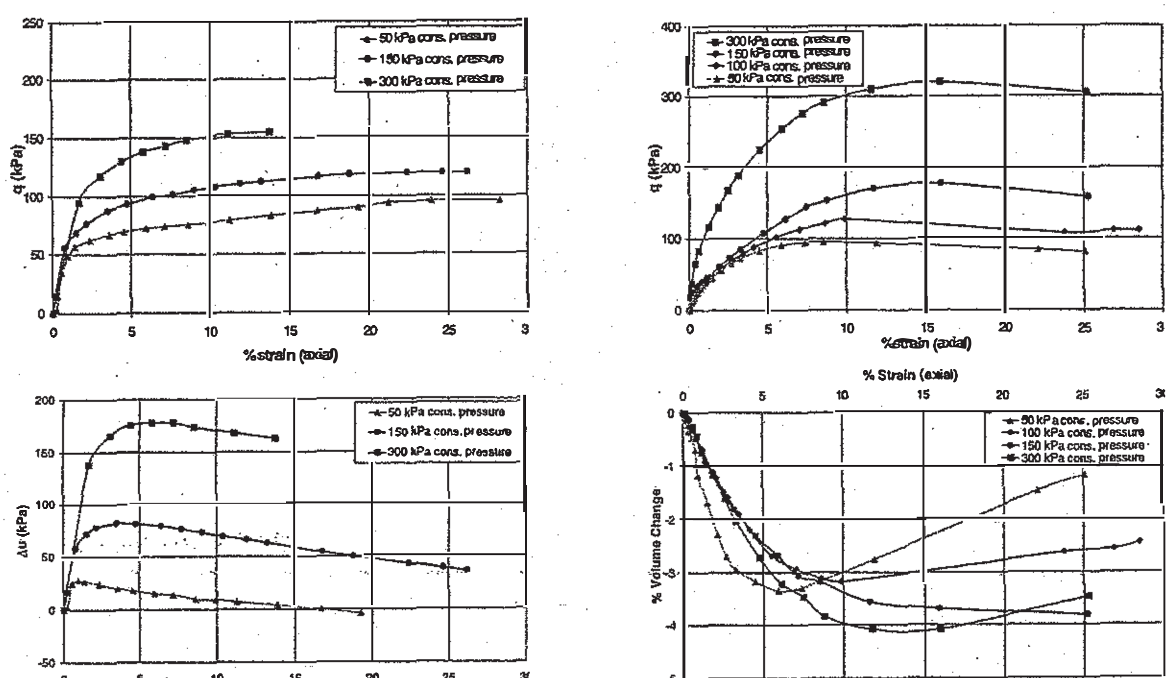
SAMPLE DETAILS	Atterberg Limits L.L.	Atterberg Limits P.L	Atterberg Limits P.I.	Natural water content %	SHEAR STRENGTH Parameter	Peak	Large strain "end" values	Residual
1. CLAY; moderate plasticity, pale grey	65	32	33	49.5	c' (kPa)	20	0	0
					ϕ' (deg.)	28.2	30.5	15.0
2. CLAY; high plasticity, dark grey.	84	27	57	45.5	c' (kPa)	11.2	0	0
					ϕ'(deg.)	26.8	25.1	10*
3. SILTY SAND; yellow	-	-	N.P	26.2	c' (kPa)	14.5	0	0
					ϕ'(deg.)	35.6	33.3	27.0
4. SANDY CLAY; with some coarse material.	104	60	44	61.5	c' (kPa)	22	0	0
					ϕ'(deg.)	35.8	36.8	34.1

2.2 *Experimental investigation*

Four samples were obtained from local sites; all were residual soils. Three came from weathered sandstone, and the fourth from weathered volcanic ash. Classification and compaction tests were first carried out, followed by triaxial testing. Consolidated undrained and drained triaxial tests were carried out on each material; they were continued to large axial strains - about 30% in the hope or expectation that this would lead to critical state behaviour and define the critical state parameter ϕ'_{cv}. In addition to the triaxial tests, ring shear tests were carried out on three of the samples to determine the residual strength friction angle ϕ'_r.

The results of the tests are described fully by Wesley and Davidson (2000), and only a summary is given here. Sample details and results are found in Table 1, and typical triaxial results shown in Figures 1 and 2. Figure 1 shows both the consolidated undrained and drained tests for Sample 1. In the consolidated undrained tests the deviator stress increases rapidly up to strains of 2 to 3%, and then undergoes a very slight but steady increase up to the maximum strain of 20 to 30%. The pore pressure initially rises and then shows a steady decrease. In the drained tests, the deviator stress rises less steeply and reaches a peak at strains between about 10 and 15 % and then shows a slight but steady decrease. The volume initially shows a significant decrease, followed generally by a slight but steady increase.

As neither deviator stress nor pore pressure (or volume change) have levelled off to constant values at the end of these tests, the critical state has clearly not been reached, and ϕ'_{cv} has not been established.



Consolidated undrained tests

Drained tests

Figure. 1 Typical triaxial test results (from Sample 1).

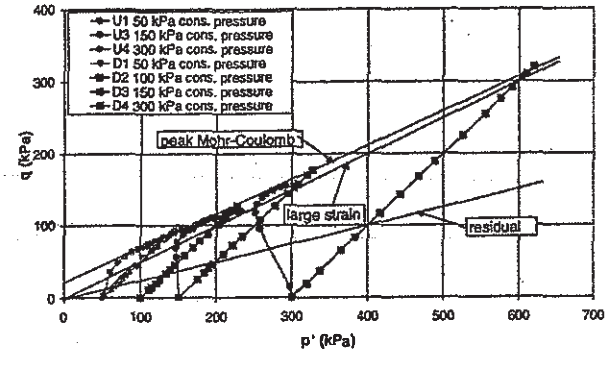


Figure 2. Typical peak, "large strain", and residual shear strength parameters (from Sample 1).

This is perhaps not surprising, as there is real doubt as to whether the critical state can be created in clays, especially in standard triaxial tests. Once the peak deviator stress has been reached, clays tend to develop specific failure planes; movement is then concentrated on these planes and deformation is no longer uniform. Instead of moving towards the critical state, the strength may simply decline from peak towards the residual value. Fig. 2 shows the same results plotted as stress paths on a q/p′ plot where:

$$q=\frac{\sigma_1-\sigma_3}{2} \qquad p'=\frac{\sigma_1'+\sigma_3'}{2}$$

"Best fit" lines have been fitted to the results as shown, giving peak Mohr-Coulomb parameters. In addition lines have been drawn through the end points of the stress paths to define the "end" or "large strain" values of the parameters. Table 1 shows peak, "end", and residual strength parameters from all the samples. It is evident that the post failure decrease in strength is quite small, and in none of the tests does it approach the residual value, except for the volcanic ash sample. This material shows the typical behaviour of allophane clays, with a high residual strength not far below the peak value.

2.3 *Discussion*

The difficulty of measuring the critical state parameter ϕ'_{cv} for clays from conventional triaxial tests is clearly a strong pragmatic reason for not using it in design. However this is not just an issue of pragmatism; the difficulty of creating the critical state in clays suggests that it is not a useful theoretical concept for such materials. If the material fails in triaxial tests by passing from the peak strength progressively towards the residual strength then the same can be expected in field situations. There are thus both pragmatic and theoretical reasons for not using ϕ'_{cv}.

At the same time, it should be noted that the difference between the peak strength and that at large strains is not great so the question of which of these two parameters to use is not really a major issue. A pragmatic approach could be taken and either the peak ϕ' value be adopted with the c′ value neglected, or the "end" ϕ' value at a strain of about 30% be adopted. These values can be expected to be very similar.

As mentioned earlier, some codes recommend that when clay is used in reinforced earth, the residual strength should be used in design. This seems a grossly over-conservative approach, without any theoretical justification. There are no other situations in geotechnical engineering where the residual strength is used on an intact material, and there is no reason for it to be used with RE walls.

It is rather difficult to see why the issue of using anything other than the peak strength has arisen with RE walls. Safety factors are used in their design, and the deformations they undergo are generally small, so there is no reason to think the soil will be stressed beyond its peak strength. Also, deformations occur in an overall manner, so there is little likelihood of displacement on specific planes leading toward the residual strength.

3 DEFORMATIONS WITH CLAY FILL

An argument sometimes raised against the use of clay fill is the likelihood that it will lead to excessive deformations. A theoretical study of this issue has been undertaken using finite element analysis of a hypothetical wall. The wall was 6m high with typical properties and spacing of the geogrid reinforcement. The analysis was carried out for both a "soft" facing with similar modulus to the soil, and a stiff facing with modulus similar to that of concrete. In each case a range of soil modulus values was investigated. The results are summarised in Figures 3 and 4.

For clay fill the Young's Modulus (E_s) is likely to be in the range of 10 to 20 MPa, while for a well compacted granular fill it is likely to be in the range of 30 to 50 MPa, or possibly higher. Figure 4 (soft facing) shows that the deformation with clay fill is likely to be about double that for a granular fill. Figure 5 (stiff facing) suggests that the clay fill may result in over twice the deformation for a granular fill. However with the stiff facing the deformations are so small that they are unlikely to be of any consequence. Even with the soft facing the deformations are small and would only be a cause of concern in special situations. The reinforced walls currently popular in New Zealand are being built using segmental block facings. The most common type of blocks are known as "Keystone" blocks. Facings of this type will have high vertical stiffness but low bending stiffness, so that the deformations to be expected will be between those shown in Figs 3 and 4. A number of R.E. walls using clay fill and geogrid reinforcement have been built in New Zealand in recent years. The clay has been of residual origin, including both volcanic and sedimentary (siltstone/sandstone) parent rock. Wall heights have been from 4m to about 12m. As far as the author is aware, the walls are performing satisfactorily, and deformations have been within expected, acceptable limits.

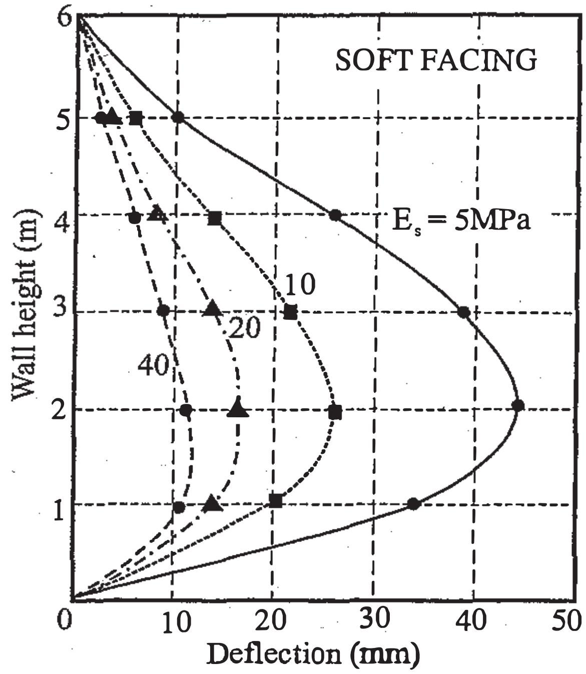


Figure 3. Influence of soil modulus on deflection,for a "soft" facing.

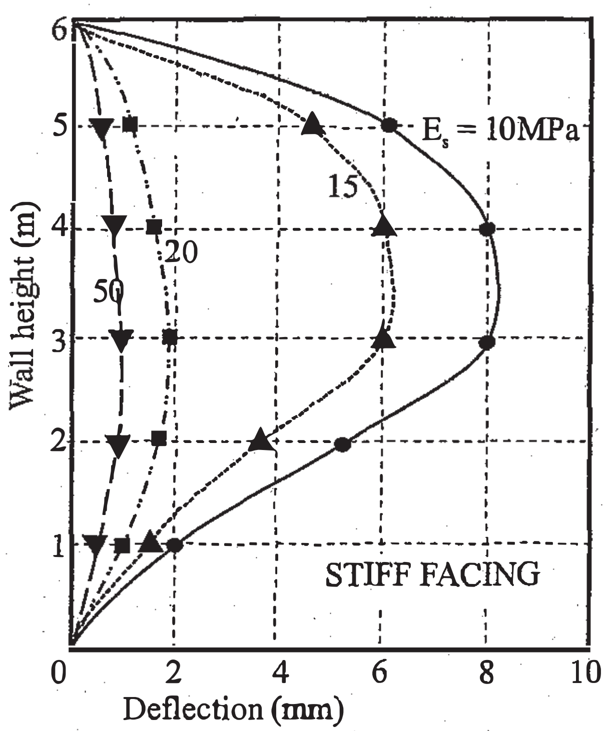


Figure 4. Influence of soil modulus on deflection,for a stiff (concrete) facing.

4 DOWNDRAG ON FACING ELEMENTS

The author became interested in this issue because of observations made of a R.E. wall recently constructed in New Zealand using clay fill and "Keystone" segmental facing. Near the base of the wall there are cracks in a number of the Keystone blocks. These are not affecting the performance of the wall, but they are clearly undesirable, if only for aesthetic reasons. The wall is about 12m high, and built of the clay from which Sample 1 (see Table 1) was taken. The different stiffness of the clay fill and the concrete facing means that the facing will attract higher stresses than its gravity weight and the soil in the immediate vicinity of the wall will have less stress than its gravity weight.

This effect has also been investigated as part of the finite element study. A wall facing 20cm thick with an E value typical of concrete (25,000 MPa) was adopted. Soil modulus values of 10 and 100 MPa, were selected, representative of a fairly plastic clay fill, and a very high quality, dense, granular fill. The results of the analysis are summarised in Figures 5 and 6.

Figure 5 shows the results as plots of vertical stress over the height of the wall at three locations, namely in the wall itself, 10cm and 10m from the wall. This illustrates very clearly the way in which the wall attracts stresses much higher than its gravity weight. The "gravity" stress at the base of the wall should be about 150 kPa; instead it is over 800 kPa. Immediately adjacent to the wall the stress is almost zero, and 10m away it is the gravity value (6m of soil at 20 kN/m^3 = 120 kPa). The soil immediately adjacent to the wall is thus largely supported by the geogrids and its weight is transferred to the concrete facing.

Figure 6 shows the vertical stresses on horizontal planes; curves are given for the base and mid-height of the wall. This again illustrates the way in which the wall attracts high stress and the soil immediately adjacent to the wall has a very low stress. The stress has virtually reached the gravity weight value at a distance of 4m. This was the length of the geogrids assumed for the analysis, although the results in Figures 5 and 6 are probably not greatly influenced by the length of the reinforcement. It should be noted also that the results are not much influenced by soil modulus values from 10 to 100MPa; it is only when the modulus approaches that of concrete that the picture presented in Figures 5 and 6 will start to change.

Figure 5 shows a maximum vertical stress in the facing of about 800 kPa. This is for a wall of height 6m. For a wall of the height in which the cracking was observed (12m) the stress would be expected to be about double this figure. Although this is many times greater than the stress due to the gravity weight of the blocks, it is only a small fraction of the expected crushing strength of the Keystone blocks,

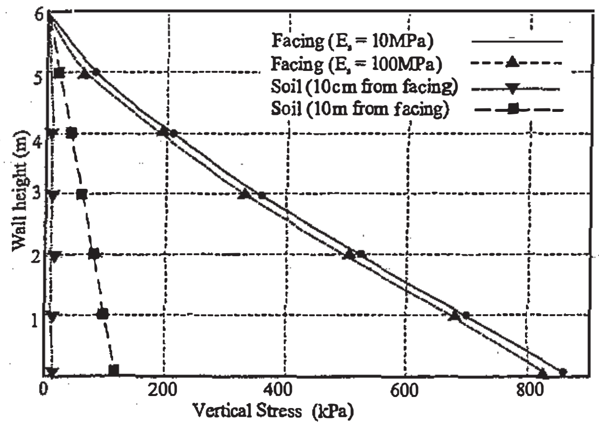


Figure 5. Vertical stresses in the facing and the soil.

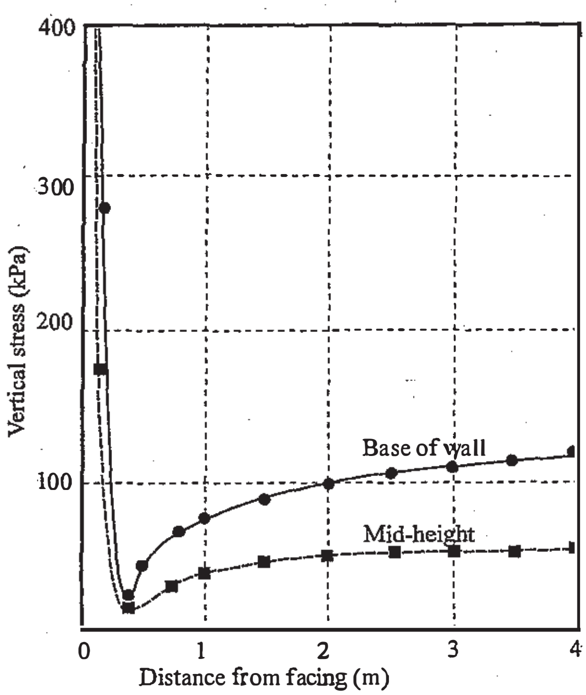


Figure 6. Vertical stresses at the base and mid-height of the wall.

which is in the vicinity of 20 MPa. Hence the additional stress in itself is not sufficient to account for the cracking observed in the blocks. It is probable that differential settlement due to non-uniform foundation conditions has induced bending stresses along the wall, and that a combination of these stresses and the additional vertical load has caused the cracking.

5 POTENTIAL FOR PORE PRESSURE DEVELOPMENT DURING CONSTRUCTION

Most clays in New Zealand exist in their natural state at water contents considerably higher than their optimum water content. This means that unless they can be dried significantly prior to compaction there is a danger of significant pore pressures developing during construction. A research programme is currently underway to investigate this situation and hopefully to provide guidelines to local engineers on how to avoid such an occurrence. Because of the variability of local residual soils, earth works are normally controlled by means of undrained shear strength (S_u) and air voids (a_v), (Pickens, 1980). A lower limit on S_u, usually about 150kPa, prevents the soil being placed too wet, and an upper limit on a_v, usually about 6%, prevents the soil being placed too dry. The concept is illustrated in Figure 7.

It is generally believed that within these limits, the likelihood of significant pore pressures developing during construction is small, but no systematic studies have been undertaken to verify this. Current research is aimed at rectifying this situation. Samples are being prepared at a range of water contents; they are then compacted at varying densities to give samples with differing air voids and shear strength. Pore pressure response is then measured in a triaxial cell. Figure 8 shows typical results for one particular water content.

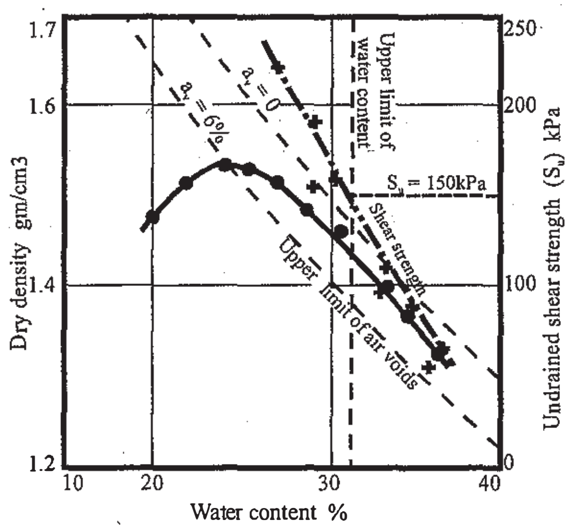


Figure 7. Strength and air voids compaction control.

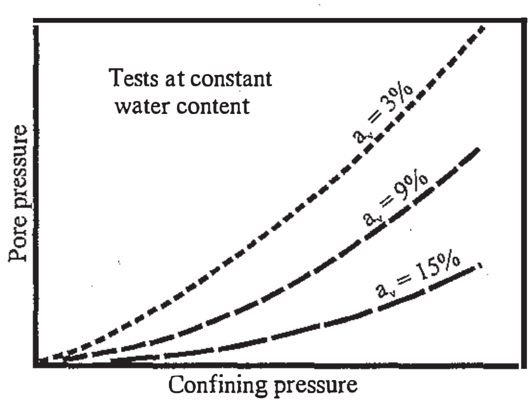


Figure 8. Influence of air voids on pore pressure.

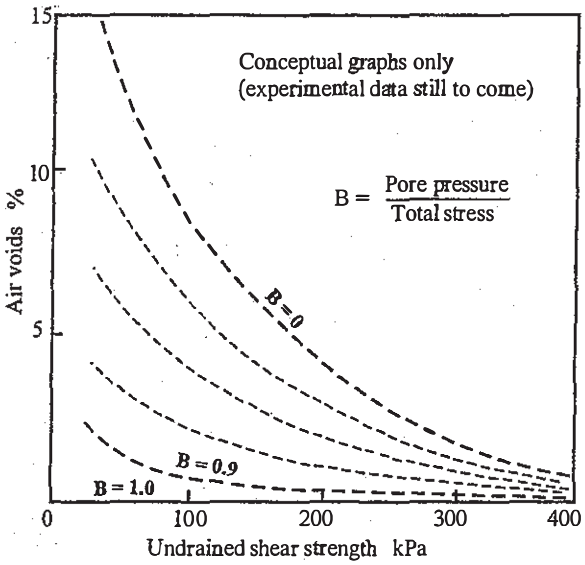


Figure 9. Pore pressure parameter B related to undrained shear strength and air voids.

The expected relationship between the pore pressure parameter B and the air voids and shear strength is shown conceptually in Figure 9. At the time of writing this paper insufficient data is available to establish the precise form the relationship should take, and whether it is independent of soil type.

6 CONCLUSION

None of the factors investigated and discussed in this paper present any real obstacles to the use of clay fill in geogrid reinforced earth walls. Only the need to keep deformations to very tight tolerances, or inability to dry the clay to an appropriate water content, could rule out the use of clay.

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