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Considerations in the design and construction of reinforced soil structures using clay fills with special reference to Indonesia

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Abstract: Clay fills are frequently used in the construction of reinforced soil structures in Indonesia, especially the well-known "tanah merah", a residual soil derived from volcanic material. The investigation of clay fills for use as an engineered fill should include determination of index properties and strength, using appropriate methods to measure shear strength in terms of effective stress. The use of inappropriate test methods leads to strength values which are meaningless. During construction, compaction control is vital to achieve a fill of adequate properties, and consideration should be given to controlling compaction based on the achieved undrained shear strength and maximum air voids content, offering advantages over the traditional methods. After placement and compaction, the pore pressure in the fill will be negative, namely the fill will be in a state of suction. It is important that this suction is maintained, so that any drainage measures used should be detailed with the aim of preventing free water coming into contact with the clay for extended periods. Case studies illustrate more than 20 years of experience of building reinforced soil structures using clay fill in Indonesia.

1.0 Introduction

One of the important benefits of using polymer reinforcement in the construction of reinforced soil structures is that it permits the use of a wide range of possible fill material. Although the ideal fill might be a well graded angular sandy gravel, materials of this type are often scarce and expensive. In the context of Indonesia, such materials may not be present at all on some islands or in some locations, so that their import is extremely expensive, rendering the reinforced soil technique uneconomic based on such fills. The use of finer soils, available either at or near to a project site offers an alternative option to form the fill for these types of structure. However, as is normal in any geotechnical design, it is vital that the design parameters are properly established, and any limitations in the use of such fills are well understood. A common term used for fill material which does not meet "ideal" requirements according to some guidelines is "marginal fill". In the author's opinion this terminology is not appropriate, because it suggests that such material is only marginally suitable. However this is not the case, because a very wide range of soil types are suitable for use in reinforced soil structures, provided that their properties are well known and consequences in terms of performance in service are understood and taken into account in design and detailing.

The purpose of this paper is to provide some background information and guidance when considering the use of clay fills in reinforced soil structures. Are there any forms of structure where clay fills should be avoided? Probably in the case of load bearing bridge abutments, where the reinforced soil block carries the full dead and live loads from the bridge deck, clay fills should not be used. Likewise for very tall retaining walls which are close to vertical, such fills would require very careful consideration. However for typical highway retaining walls, clay fills may well provide a very suitable solution, and there is much experience in Indonesia of using such soils in highway structures. Facing angle is also relevant to these considerations. Reinforced soil steep slopes (this term generally applying to facing angles less than 70° to the horizontal with a "soft" or vegetated facing) are likely to be less sensitive to post-construction deformation during service, such that clay fill may be eminently suitable.

The contents of this paper are based mostly on published information, with the aim of summarising some important points which should be taken into account when planning, designing and constructing reinforced soil structures using clay fill. An important source of information is the work of Dr Laurie Wesley, and his many publications about Indonesian soils. Dr Wesley is well known to many Indonesian civil engineers, and his book on the "Fundamentals of Soil Mechanics for Sedimentary and Residual Soils" (Wesley, 2010a) should be standard reading for geotechnical engineers practising in Indonesia. This book has been translated into Indonesian as "Mekanika Tanah untuk tanah endapan & residu" (Wesley, 2011) bringing this source of information readily in reach of engineers in Indonesia. In addition the author published a paper on the "Practical use of clay fills in reinforced soil structures" (Dobie, 2011) in the 2011 HATTI Conference, which had the main aim of looking at the issue of pore pressures in clay fill. This paper summarises information on the nature and engineering properties of clay fills, their compaction and performance in service. The specific issue of drainage provisions with regards to such structures is discussed.

2.0 Description and properties of clay fills

2.1 *Sources of clay fill in Indonesia*

The vast archipelago of Indonesia is home to a very wide range of clay soils, from most of the main processes of formation, such as alluvial, residual and volcanic. There are also huge areas of peat and highly organic soils, which would not normally be considered suitable for use in reinforced soil structures. The properties of the clay soils are equally wide ranging. In some areas the clays are highly expansive, normally due to a significant content of the clay mineral montmorillonite. Figure 1 shows the surface of such a deposit near Surabaya in East Java. Clays of this type are best avoided for use in reinforced soil structures, because the large seasonal volume changes are likely to be unacceptable, and deep cracks formed during the dry season may permit water to enter deep into the fill once the wet season starts. The extensive deposits of Holocene alluvial clays are also unlikely to be suitable for use as an engineered fill, mainly due to their very high natural water content, which would require extensive reduction by drying out before use, and possible undesirable behaviour as a fill, in terms of volume change during service.

Figure 1: Surface of a highly expansive soil in Surabaya, East Java

One clay soil of major importance as an engineering fill with excellent properties is the deep red or brown residual soil derived from material of volcanic origin, known locally as "tanah merah" (meaning red soil). Figure 2 shows tanah merah being used to build reinforced soil retaining walls as part of the Jakarta Outer Ring Road project in 2003. Section 5.0 of this paper provides further information about this project and other structures built using tanah merah.

Figure 2: Using tanah merah in the construction of the Jakarta Outer Ring Road

Tanah merah is well known for having very good engineering properties, mainly due to the dominant clay mineral which is halloysite. The halloysite clay mineral is a form of kaolinite, in which the structure is tube shaped, rather than the normal plate-like shape of kaolinite, illite and montmorillonite. For further information, Wesley (2010a) provides a detailed discussion about the mineralogy of clay. This difference in the clay mineral shape of halloysite has important benefits in terms of engineering properties, making this clay a very good fill for use in reinforced soil structures. In this paper, the name "tanah merah" is only used to refer to these residual soils derived from material of volcanic origin, which are therefore only found on the islands where there has been extensive volcanic activity in the past, such as Java. It is the author's experience that engineers in Indonesia sometimes regard all clay soil of a brown or reddish colour to be tanah merah, even though they may not follow the definition above. This lack of distinction may lead to problems if the good engineering properties associated with tanah merah are applied to, say, a clay soil of alluvial origin. Therefore appropriate testing, both of index properties and shear strength remains very important. This is discussed in the following sections.

2.2 *Importance of index properties*

The site investigation of a clay deposit being considered as a fill for a reinforced soil structure should include determination of the normal index properties such and unit weight, water content and specific gravity, measured on samples taken from the proposed borrow source. Of major value to the assessment of suitability as a fill are the Atterberg limits, namely the plastic and liquid limits (W_P and W_L, or PL and LL). The Atterberg limits are best summarised on a plot of plasticity index ($PI = W_L - W_P$) versus liquid limit, known as a plasticity chart. Figure 3 shows a plasticity chart with Atterberg limit values for a number of clays, partly after Wesley (2006). The chart includes the "A" line, which is given by $PI = 0.73(W_L - 20)$. By way of comparison, the classifications for clay soils according to the AASHTO system are shown in red (A-4, A-5, etc). It should be noted that the differentiation between A-7-6 and A-7-5 soils is close to the "A" line, but is not the same. For fill suitability assessment, the "A" line should be used.

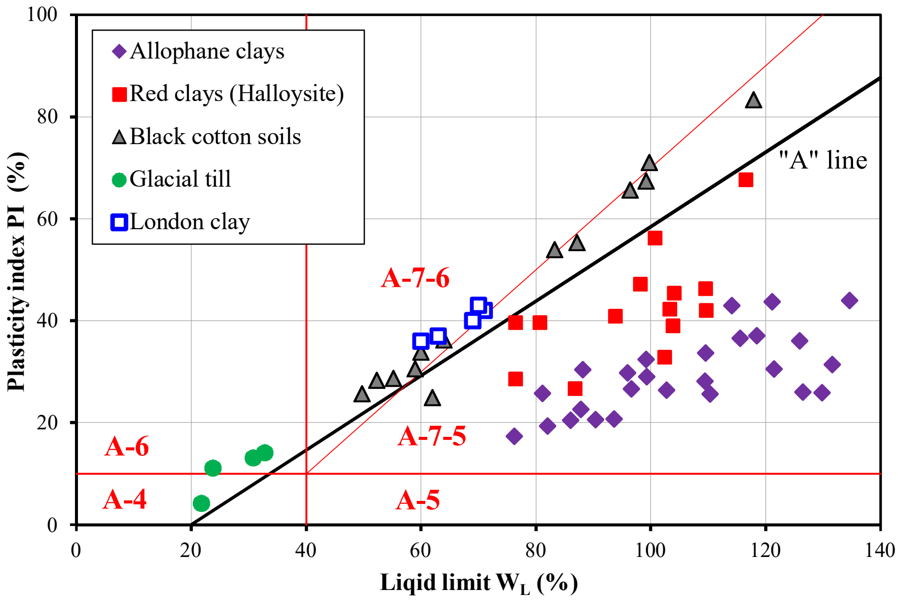

Figure 3: Plasticity chart for various clays which may be considered for use as fills in reinforced soil structures

When reviewing Atterberg limit data provided in site investigation reports, there are two important additional items of information which should be known.

- Firstly the % retained on the 0.425mm sieve, because only material smaller than 0.425mm (mid-way through the sand sizes) is used in the Atterberg limit tests. If this percentage is very small (or zero) then the Atterberg limit results may be taken as fully representative of the clay being investigated. However if the percentage retained is very large, then the fine fraction of the soil may only be of minor significance, and the Atterberg limits could lead to a false impression of suitability.
- Secondly the method of preparation, especially if wet sieving has been used to remove particles coarser than 0.425mm. This procedure results in the fine residue becoming a slurry which needs significant drying back to a state suitable for the Atterberg limit tests to be carried out.

With regard to the second point, the method of drying may be by air drying (takes a long time) or by oven drying (much quicker). However one problem is that either of these procedures can significantly affect the measured Atterberg limits, especially for clay minerals like halloysite and allophane. This has been investigated by a number of researchers, such as Rusli & Iqbal (1990), who measured the Atterberg limits of Malaysian clays based on "natural" (ie. without sieving and significant drying), air drying and oven drying. In general the measured Atterberg limits reduce from natural to air drying to oven drying. Similar data is presented by Wesley (2010b). Fortunately the reductions generally result in the plotted points on the plasticity chart moving almost parallel to the "A" line, but for some soil types, the distance moved can be very large, therefore giving a significantly different impression of the nature of the soil based on its Atterberg limits. This can be quite serious if the natural water is compared to the liquid limit in order to assess the sensitivity of a clay soil. If natural water content is much higher than the liquid limit, then this is a sign that the clay may be very sensitive, in which case a very large drop in undrained shear strength may occur on remoulding. Such behaviour would not be desirable for a clay material to be used as an engineered fill, almost certainly ruling out its potential use. However this could be a false impression if the liquid limit value was excessively reduced by the method of sample preparation.

The data on Figure 3 shows allophane clay, red clay (halloysite) which is tanah merah and black cotton soil which is high in montmorillonite. Also shown are two soils from United Kingdom. Glacial till is suitable for use in reinforced soil structures. As regards London Clay, this is well known as an expansive soil, however to the author's knowledge it has been used in reinforced soil structures in United Kingdom, but only in lower angle slopes. Figure 4 shows the plasticity chart for clay fills which are used in some of the examples which follow in the later sections of this paper.

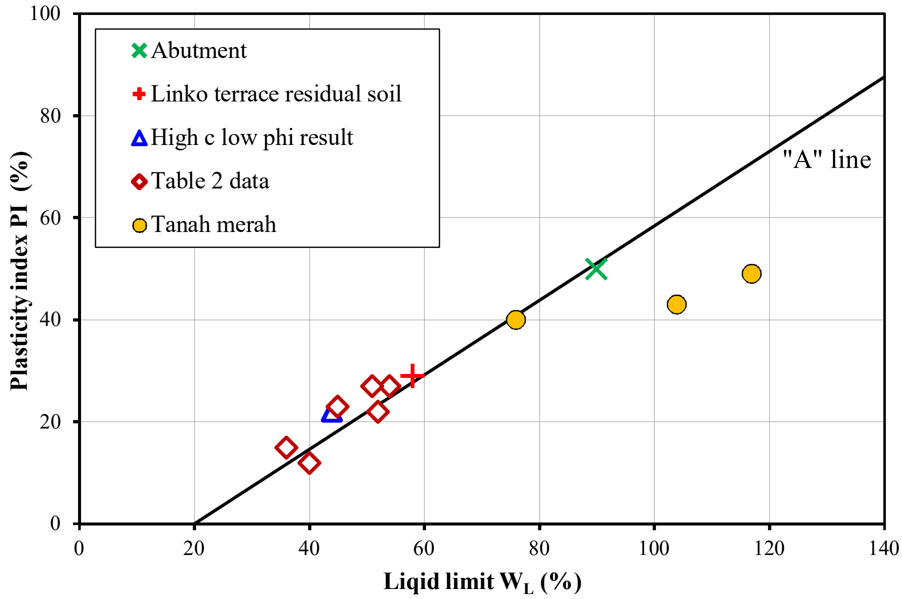

Figure 4: Plasticity chart for some of the clay soils mentioned in this paper

2.3 *Shear strength properties*

For the design of reinforced soil retaining walls and steep slopes, the soil strength parameters required for both the reinforced fill and retained fill are the strength parameters in terms of effective stress (c′ and ϕ′), sometimes referred to as the drained shear strength parameters. The "dash" after each term indicates that these parameters are in terms of effective stress. Table 1 summarises the main standard laboratory tests which are suitable to measure c′ and ϕ′ for clays, with a few comments about each test. Of these three, the consolidated drained triaxial test is rarely used.

Table 1. Standard laboratory shear strength tests suitable for clay fills

Test type	Comments with regards to testing clay fills
Shear box	Must follow full procedure of saturation, consolidation and shearing, with rate of shearing based on time to failure = approximately 12.7 × t_{100} from the consolidation stage. Only feasible with small shear boxes for clay fills.
CU triaxial	Best test to use, requires high quality "effective stress" laboratory due to the importance of measuring the pore water pressure accurately, and keeping the test system air-free. Provides information about the undrained behaviour.
CD triaxial	Shear stage takes longer than CU test due to requirement to keep pore pressure = 0. No information is obtained about the undrained behaviour.

Based on the author's experience of reviewing geotechnical site investigation reports for projects in Indonesia and elsewhere in Asia Pacific, the shear box test seems to be the method most commonly used for re-compacted clay fills. The shear box has the advantage of being relatively easy to use, and does not require all of the controls and sophisticated equipment required for CU (consolidated undrained) triaxial tests, in which the accurate measurement of pore pressure is vital in order to obtain a meaningful result.

The shear box also has the advantage that it is relatively simple to re-compact clay fill into the test apparatus. However when used with clay specimens, it is very important that the correct procedure of saturation, consolidation and shearing is carried out, with the rate of shearing determined from the consolidation stage of the test. This can result in very slow rates of shearing, but this is very important to ensure that any pore pressures generated during shear (either positive or negative) are dissipated as the shearing takes place. This is necessary because there is no way to measure pore pressure during shear in a shear box, so the only way to ensure that the test measures c′ and ϕ′ is to shear slowly so that pore water pressure can be assumed to be zero. In this case the applied normal total stress can also be assumed to be the normal effective stress. The effect of using the incorrect procedure is examined in Section 2.4 which follows.

Figure 5 shows the result of testing a re-compacted sandy clay in a small (60mm) shear box. Under the test conditions the rate of shearing is given as 0.0072 mm/min, which is very slow. For the 50 kPa pressure, which reaches 9mm displacement, this would have taken 9/0.0072 = 1250 minutes or 21 hours. So the total duration of this test on three specimens allowing for saturation and consolidation would have been more than one week. However the result obtained is satisfactory, and the values of c′ = 1 kPa and ϕ′ = 34° would seem to be representative of a well compacted sandy clay.

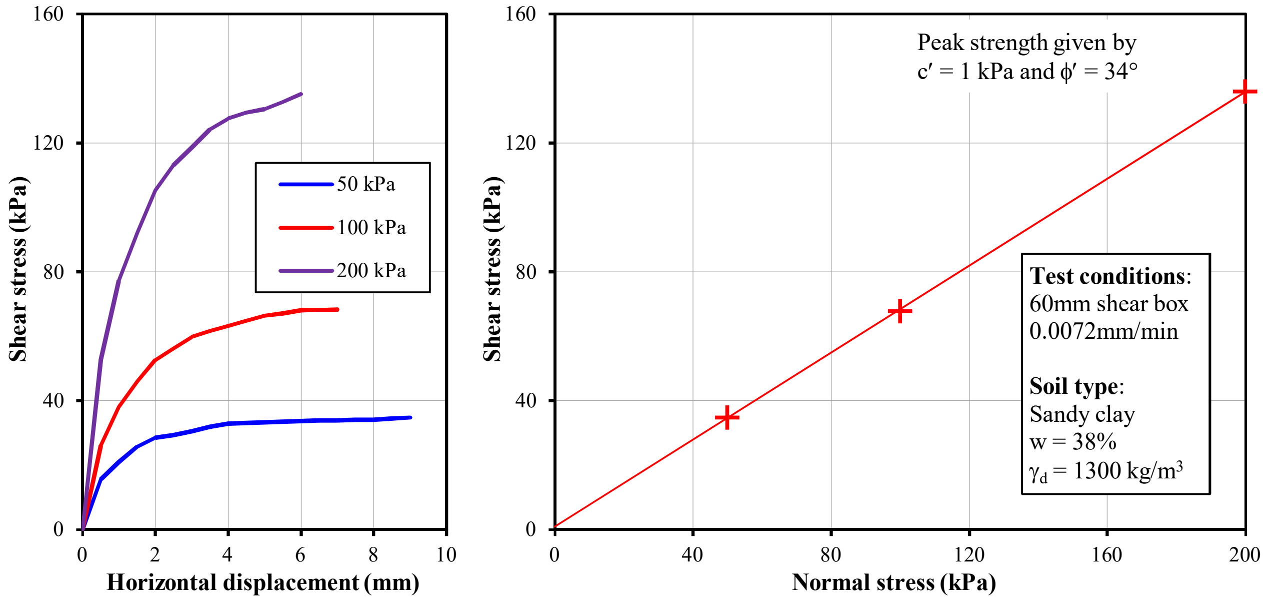

Figure 5: Consolidated slow shear box test carried out on sandy clay fill

CU (consolidated undrained) triaxial tests require a much higher level of sophistication in terms of laboratory equipment compared to the shear box. In most testing laboratories, the equipment is set up in a special "effective stress room". Procedures must include effective de-airing of all pressure lines connected to the triaxial cell. Transducers are used to measure pressures and stresses, with automated control and recording via a computer. In the CU test, although the shearing is "undrained", the rate of shearing must still be quite slow, because pore pressure changes due to shear are generated mainly in the middle of the test specimen, but they are measured at its ends. Therefore the rate of shearing must be sufficiently slow to ensure minimal gradient of excess pore pressure through the specimen. During a CU triaxial test, the known or measured values are cell pressure, deviator stress and pore water pressure, and these results are best summarised as stress paths, consisting of shear stress plotted against mean normal effective stress during each test.

Figure 6 shows the stress paths for a series of CU triaxial tests carried out on specimens of Linko Terrace residual soil from Taiwan, described as sandy silty clay, although sand content is minor. The data provided includes Atterberg limits which are plotted on Figure 4 and fall just above the "A" line. This test programme was carried out on the clay fill with three different preparation conditions (for this fill maximum dry unit weight (γ_{dmax}) = 15.0 kN/m^3 with optimum water content (w_{opt}) = 25.2%):

DRY: Dry of optimum at just over 90% of γ_{dmax} on the dry side (s_u of compacted specimen = 78 kPa)

OPT: At optimum so close to 100% of γ_{dmax} (s_u of compacted specimen = 137 kPa)

WET: Wet of optimum at just over 90% of γ_{dmax} on the wet side (s_u of compacted specimen = 19 kPa)

Based on these three methods of specimen preparation, the initial consistency of the compacted clay varies from soft for WET up to very stiff for OPT. However based on the stress paths shown on Figure 6, all failure behaviour fits closely to an envelope defined by c′ = 5 kPa and ϕ′ = 27°, despite the differences in the starting conditions of each set of specimens. DRY and WET have similar behaviour, which is not surprising because after the saturation and consolidation stages of the test, the specimens would have had similar density. The undrained shear behaviour of OPT is different. However it could be asked what is the benefit of better compaction if c′ and ϕ′ remain almost the same? The important difference is in the pore pressure generated during shear. The total stress paths are 45° lines starting from the same points as the effective stress paths (indicated as TSP by fine dotted lines), and the horizontal difference between the lines is the excess pore water pressure generated during undrained shear. Therefore for OPT, the excess pore water pressure is much less than for DRY and WET, and the failure shear stress reached is therefore considerably higher, by about 50%, which is a desirable behaviour for a clay fill.

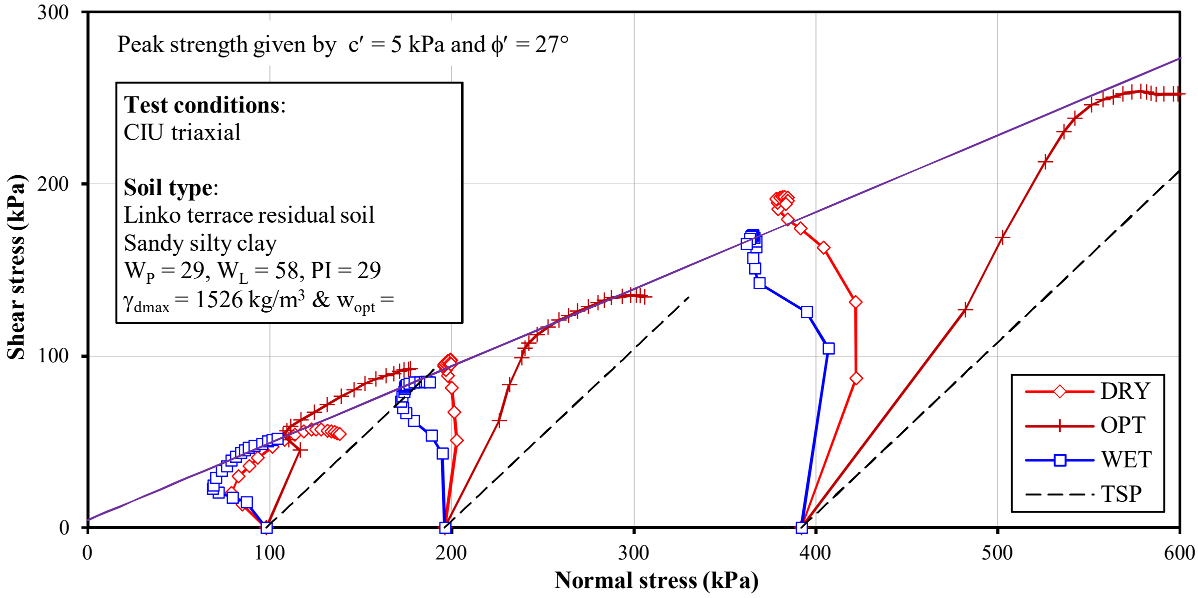

Figure 6: CU triaxial tests carried out on Linko Terrace residual soil with different preparation conditions

In the case that reliable measurements of ϕ′ for a clay fill are not available, or if verification of measured values is required, then there are many published empirical relationships between ϕ′ and index properties, in particular the Atterberg limits. One example of such a relationship is shown in Figure 7, taken from Figure 2.6 of the UK Highways Agency design guide for reinforced soil slopes, reference HA68/94 (Highways Agency, 1994). This guide was formally withdrawn in 2016, but remains a useful document providing guidance on the design of reinforced soil slopes for a wide variety of fill types, including clay fills. However when such relationships are used, it is important to be aware of the basis for the data plotted, and in this case the peak strength values are for glacial tills, whereas the residual strengths are for glacial tills and sedimentary clays. It should be noted that residual strength is the shear strength after a large displacement on the shear plane, and would not normally be relevant to the assessment of properties for a re-compacted clay fill for use in a reinforced soil structure. Figure 7 includes suggested values for the constant volume (CV) shear strength both from HA68/94 and BS 8002, the British Standard for the design of earth retaining structures (BSI, 2015). Generally ϕ'_{cv} may be considered as a lower bound value for re-compacted fill.

Figure 7 also includes data from three of the cases presented in this paper, with the Linko Terrace strength and index properties taken from Figure 6. Both the Linko Terrace and the Abutment soils are residual soils, plotting either on the top edge or well above the data for the glacial and sedimentary soils in Figure 7. Data points are also shown for tanah merah. This suggests that the origin of the soil may also be of relevance to the assessment of ϕ′, so that an alternative approach might provide more representative values. Figure 3 indicates how different types of clay plot at different distances above or below the “A” line, which is examined in Figure 8 in relation to ϕ′, based on Wesley (2006, 2010).

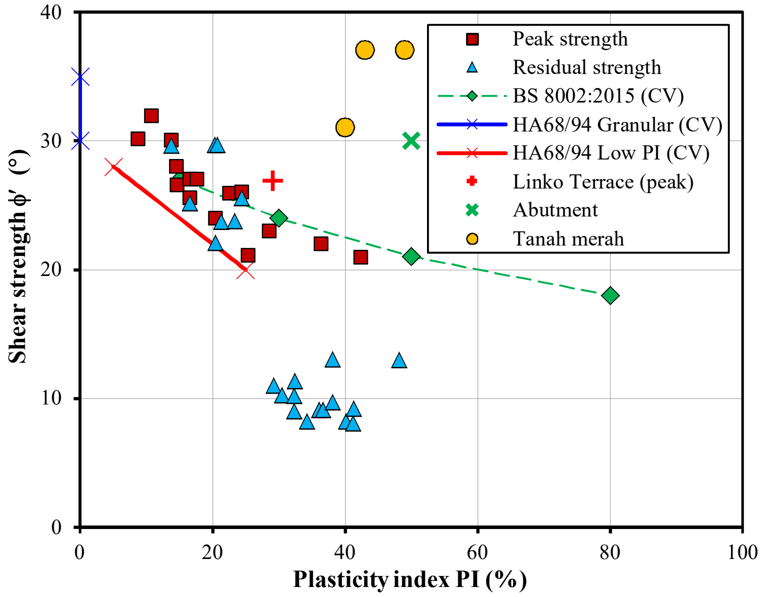

Figure 7: Assessing ϕ′ for clays based on plasticity index from HA68/94 (Highways Agency, 1994)

Figure 8 shows a relationship between ϕ′ and the distance above or below the "A" line based on the plasticity chart, after Wesley (2006, 2010a). This provides an elegant resulting relationship with a strong correlation, demonstrating clearly that as soils plot increasingly higher above the "A" line their engineering properties deteriorate, whereas if they plot increasingly lower below the "A" line their engineering properties improve. Data for the Linko Terrace residual soil, the Abutment residual soil and London Clay (sedimentary) have been added to the original plot, and fit in with the general trend very closely.

Many published correlations between ϕ′ and index properties for clays do not provide information about the soil formation process or clay mineralogy, and scatter can be very great. It is therefore important that investigations are carried out to understand the source of such correlations so that an informed decision may be made about applicability. Of course, if in doubt, then shear strength tests should be carried out using appropriate methods. The next section will examine the result of using the wrong test methods to measure ϕ′.

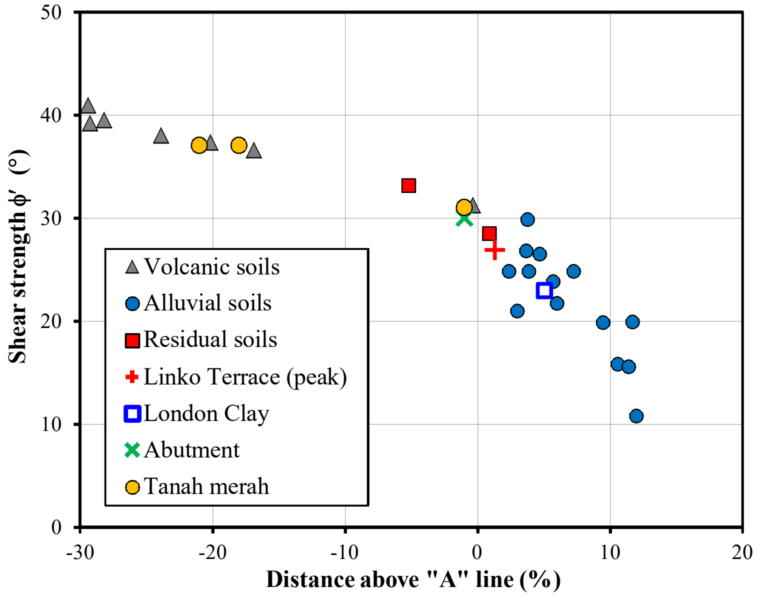

Figure 8: Assessing ϕ′ for clays based on distance above or below the "A" line (after Wesley, 2006 & 2010a)

2.4 *The "high c low ϕ" problem*

Over many years of reviewing data from geotechnical reports prepared to support the design of reinforced soil structures in the Asia Pacific region and further afield, the author has frequently observed an issue with reported shear strength data for clay soils due to the use of fast shear box tests. This issue occurs so frequently that it has been dubbed the "high c low ϕ" problem, with the dashes, indicating strength properties in terms of effective stress, deliberately omitted. Test standards in some countries permit such forms of testing, for example in China there are shear box test standards for both unconsolidated fast shear tests on clay and consolidated fast shear tests on clay. Both forms of testing provide meaningless results, somewhere in between undrained and drained, and the resulting strength parameters cannot be used as the basis for the design of reinforced soil structures.

Table 2 tabulates soil properties from a proposed source of clay fill for a reinforced soil project in Indonesia. Water content is generally close to the plastic limit, and the clay is firm to stiff on sampling. The Atterberg limits are shown on Figure 4, and plot just above the "A" line mostly, suggesting that this clay is not tanah merah. The clay would probably be suitable as a fill for a reinforced soil structure, but then values of shear strength are required for design. The table includes c and ϕ, but the referenced standard is for an unconsolidated undrained (fast) shear box test. The results given in the table are of the type "high c low ϕ", and are not suitable for use in design based on effective stress, as required for the design of a reinforced soil structure. Therefore this data should be disregarded, and an assessment is probably best based on correlations with Atterberg limits. The position of the plasticity data on Figure 4 combined with the relationship on Figure 8 suggest ϕ′ slightly less than 30°, with c′ = 0 which is a common assumption and recommendation for re-compacted clay fill.

Table 2. Laboratory test data for a clay fill source in Indonesia

Test	SNI Standard	A	B	C	D	E	F
w_n (%)	03-1965-1990	21.1	36.6	20.6	23.5	21.5	23.9
W_P	03-1966-1990	28	27	24	30	22	21
W_L	03-1967-1990	40	54	51	52	45	36
PI		12	27	27	22	23	15
% < 0.425mm	03-1968-1990	97.7	98.5	98.7	99.2	98.7	99.1
s_u (kPa)	03-3638-1994	86.8	100.6	54.0	133.9	95.2	93.7
c (kPa)	03-3420-1994	49.1	34.3	45.1	57.9	43.2	44.1
ϕ (°)		24.9	25.5	16.2	21.2	18.5	20.5

Figure 9 shows the results from an unconsolidated fast shear box test on a clayey silt from the Mediterranean area. The Atterberg limits are similar to the Table 2 data, and the location on the plasticity chart is also therefore similar on Figure 4. Likewise the reported c and ϕ values (again without "dash") show the same high c low ϕ characteristics as in Table 2.

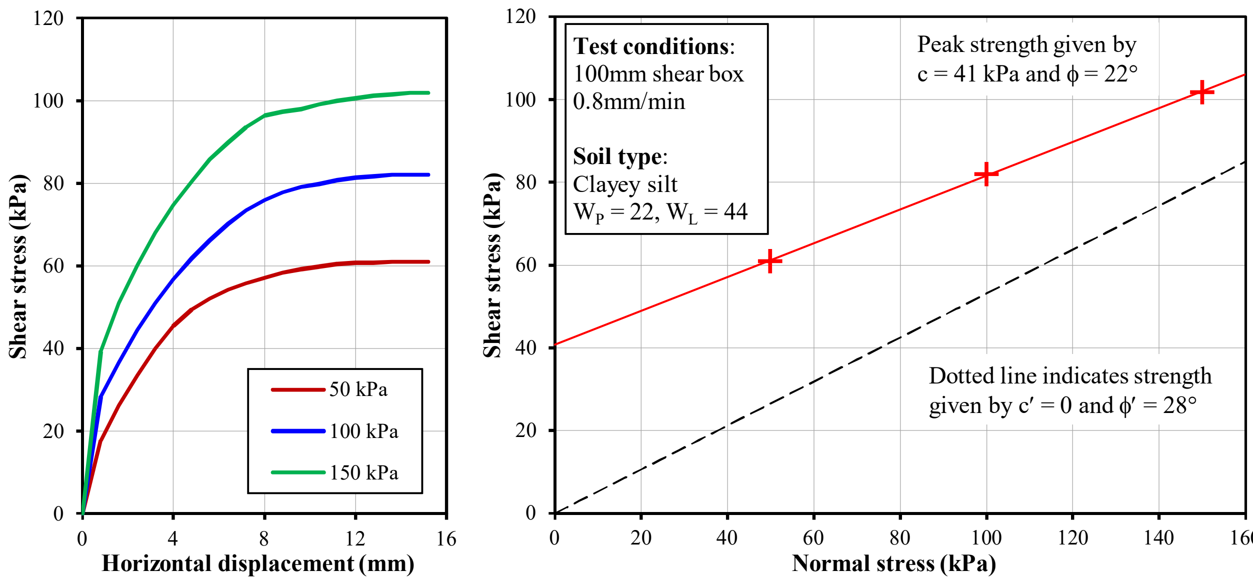

Figure 9: Unconsolidated fast shear box test result on re-compacted clayey silt

The results on Figure 9 provide an opportunity to discuss why the unconsolidated fast shear box test procedure cannot give values of c′ and ϕ′. Assuming that the sample is compacted into the shear box, which is the case here, and that the material is reasonably stiff, which would be appropriate for a clay fill, then the pore water pressure in the specimen will be negative, ie in suction. Also there will be air in the test specimen. Therefore when the normal stress is applied as part of the test procedure, this is a total stress, so if pore pressure in the sample is negative, then the effective normal stress is higher than the total normal stress applied. During fast shear, the pore water pressure will change compared to the initial state, and the suction might increase or decrease, depending on the soil density. This means that when shear failure is reached in the test, the applied normal stress is a total stress, and because the pore water pressure or suction is not known, then the effective stress is also not known. A line has been drawn on Figure 9 for c′ = 0 and ϕ′ = 28°, which might be considered reasonable for a soil of this description. At the level of the 50 kPa plotted normal stress, the horizontal distance to the dotted assumed "effective stress" line is a little over 60 kPa, so this would be the suction in the specimen at failure. However it cannot be measured, so the shear box test can only be reported in terms of total stress.

The aim of the consolidated slow shear test procedure described in Table 1 is to bring the measured total stress data as close as possible to the dotted line in Figure 9, namely to keep excess pore water pressure or suction as close to zero as possible.

3.0 Compaction control

As with any engineered earth structure, control of compaction of the fill used for reinforced soil structures is very important. This is especially the case with clay fills, which in general are more difficult to compact than granular soils. The traditional method of controlling compaction is based on carrying out laboratory compaction (Proctor) tests to measure maximum dry density and optimum water content of the material proposed to be used as a fill, and then permitting the fill to be placed within certain limits of these optimum values. On-site control is performed by measuring the density and water content of the compacted layers, and then comparing to the limits established by the compaction tests to check compliance.

Wesley (2006, 2010a, 2010b) describes an alternative approach to controlling the compaction of clay fills developed in New Zealand, and the remainder of this section is based entirely on the publications of Dr Wesley. Figure 10 shows the traditional compaction curve for a clay fill, namely a plot of dry density versus water content. However, as part of the test procedure, it is also possible to measure the undrained shear strength of the compacted clay specimens, in this case both by unconfined compression tests and by vane tests, which give slightly different results. This additional data measured during the compaction test is very helpful in understanding the likely behaviour of the compacted clay, and such studies should be carried out as a matter of standard procedure when investigating potential sources of clay fill, especially if there is little experience of using them in reinforced soil structures.

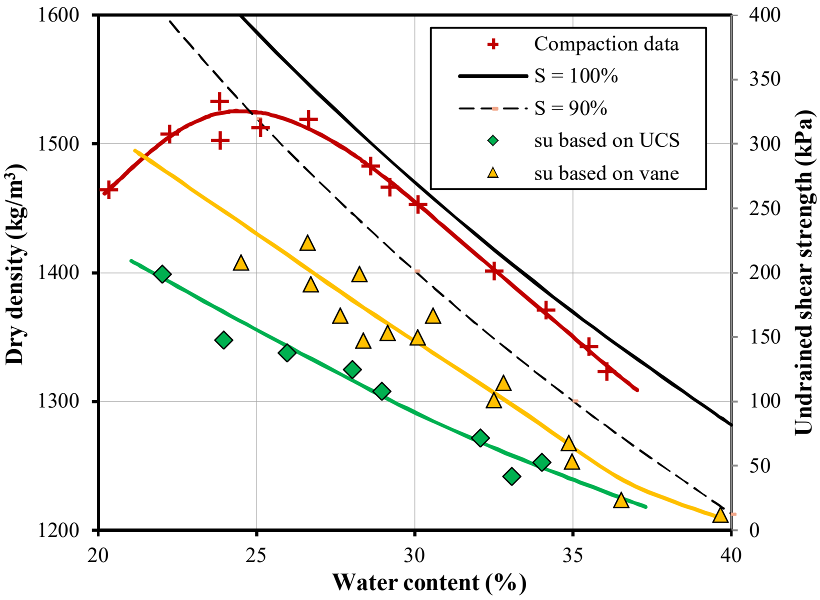

Figure 10: Compaction data for clay fill after Wesley (2006, 2010a & 2010b)

In Figure 10 the values of undrained shear strength from the two tests are somewhat different. The important point from this figure is that the undrained shear strength at the optimum water content is about 230 kPa from the hand vane and 160 kPa from the unconfined tests. The undrained shear strength of all soils of moderate plasticity is in this range when compacted at standard Proctor water content. Therefore by specifying an undrained shear strength of not less than 150kPa, it can be ensured that the soil is not compacted wetter than optimum water content. In order to ensure that the clay is not compacted too dry, an upper limit is put on the air voids in the soil, normally 8% or 10%. If the soil is too dry, it becomes impossible to compact it to give air voids below these limits. Suitable limits for these two control parameters are as follows:

- Undrained shear strength: not less than 150 kPa (average of 10 tests) with a minimum single value of 120 kPa.
- Air voids: for most normal soils not greater than 8%, but for volcanic ash (allophane) clays not greater than 12%

The two methods of compaction control produce a very similar range of values of dry density and water content. The undrained shear strength and air voids method has the major advantage that the specification does not change with changes in soil type. It has been found to be very satisfactory in producing firm high-quality fills. The value of optimum water content may vary over a wide range, but the specification in terms of shear strength and air voids remains unchanged. Furthermore the use of a hand vane or penetrometer is a very quick way to measure undrained shear strength on site, but it is still necessary to determine in-situ water content, density and specific gravity in order to check the air voids content. However if the clay source tends to be on the wet side, then the air voids requirement should be met reasonably easily.

4.0 Performance in service

4.1 *Pore pressures in compacted clay fill*

Once completed it is important to appreciate how a compacted clay fill will perform in the medium to long term, during the service life of the structure. An important, and often misunderstood, feature of clay fill is the pore water pressure which is likely to existing after completion of the structure. Dobie (2011) examines this issue in detail, presenting a large amount of measured data on the topic. Put simply, a mass of well compacted clay fill of typical highway structure height, compacted to the specification suggested in Section 3.0, will be in a state of suction after completion, that is the pore water pressures will be negative. This is a very important and desirable feature of a compacted clay fill, and has a major influence on the design and detailing of drainage measures incorporated into the structure. Drainage measures are discussed in Section 4.3.

Starting with a lump of clay fill excavated and delivered to the construction site, and assuming it is suitable for use in the fill, its initial state of stress can be assessed from Figure 11 (Dobie, 2011). Total stress is sensibly zero at this stage, so that the undrained shear strength of the clay must be created by suction within the clay, as indicated in Figure 11.

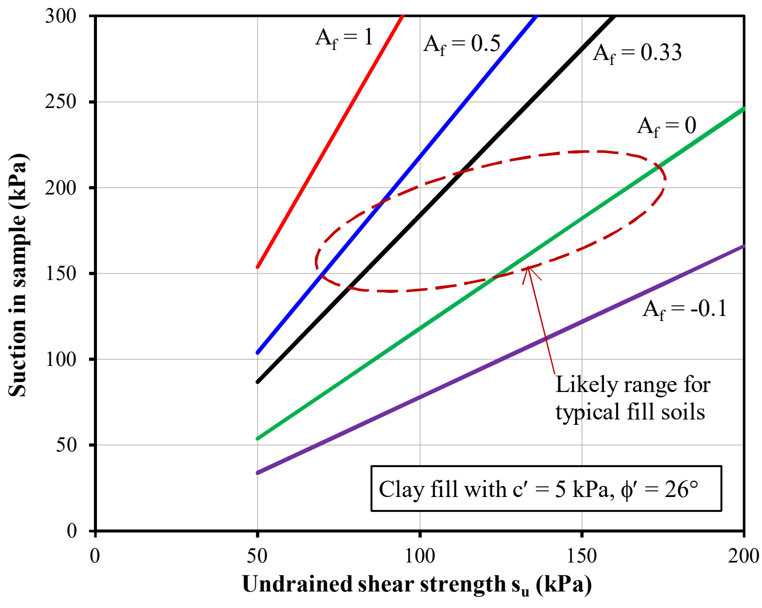

Figure 11: Relationship of suction in an unconfined soil sample to its undrained shear strength (Dobie, 2011)

The derivation of Figure 11 assumes that the clay is saturated, but in reality as suction increases, air dissolved in the pore water will tend to come out of solution, so that the clay becomes partially saturated, which is likely to have the effect of reducing suctions compared to Figure 11, however suctions will still be high. As the fill is spread and compacted, the high suctions will remain. There is a notion often mentioned that compaction induces high pore pressure. This may well be the case for the brief moment as the roller is directly above an element of clay in the layer being compacted, but once the roller moves on, the clay will relax and try to expand. This expansion will be resisted by suction developed in the pore spaces. By the principle of effective stress, in order for the surface of a compacted clay to have sufficient strength to bear the weight of a roller or compaction machine, the pore water pressure must be large and negative to create the required effective stress, and, therefore, sufficient undrained shear strength.

As filling continues and the total stress builds up above the initial layer at the base of the fill, this undrained increase in total stress will result in a lessening of the negative pore water pressure at the base of the fill. Eventually, if the fill becomes sufficiently high, the pore water pressure at the base will become positive. For normal earthworks specifications (ie. target undrained shear strength of 150 kPa), this height could be anything from 10m to 15m. This process is illustrated very clearly in Figures 12 and 13, taken from a paper published in the 1978 ICE Conference on Clay Fills, by Farrar (1978), who presents pore pressure data from a 12m high highway embankment constructed using compacted London Clay. A simple section is shown in Figure 12.

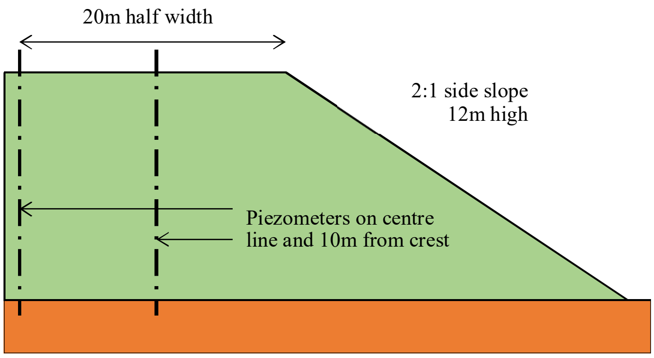

Figure 12: Section through London Clay highway embankment after Farrar (1978)

The reported properties of the clay fill are: $W_P = 24$, $W_L = 73$, unit weight = 20 kN/m3 and water content on placement was 2 to 4% over optimum. Undrained shear strength was not reported, but the clay fill is described as being wetter than desired, so it might be expected that s_u would have been on the low side. The fill was constructed over an 18 month period, and pore water pressures in the fill were measured at end-of-construction, after 2 years and after 4 years. A detailed profile of these measurements is shown in Figure 13.

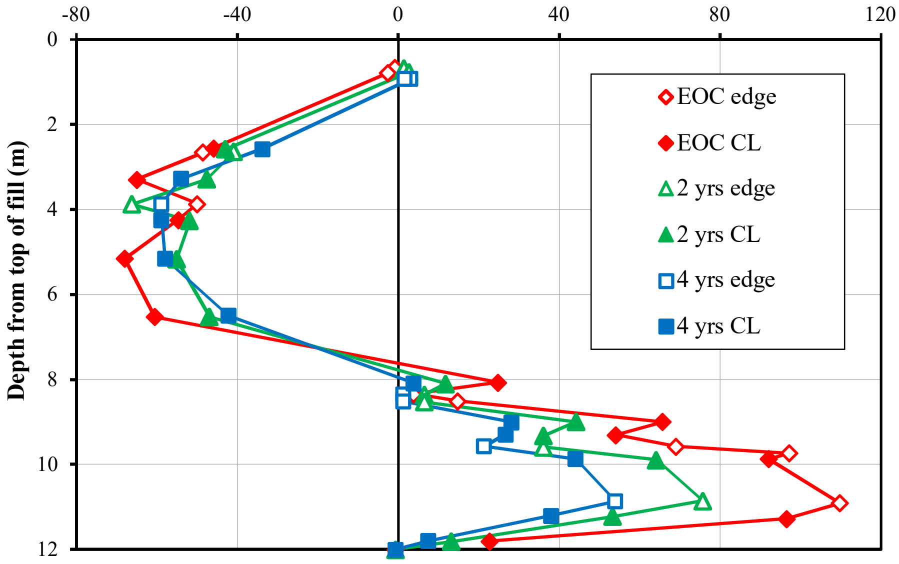

Figure 13: Profile of pore pressure versus depth in London Clay highway embankment

This demonstrates the principles discussed above very clearly, with suction in the upper 8m of the fill, and positive pore water pressures below this level. Suction at the top of the fill has been affected by contact with surface water, yet the distribution of suction measured over four years is sensibly constant. However below the 8m depth, the excess pore water pressures are seen to be dissipating, albeit slowly, presumably towards a basal drainage layer.

4.2 *Deformation in service*

It is well known that all reinforced soil structures will deform outwards laterally as the reinforcement builds up load. This is an inevitable process in the development of an equilibrium condition in the structure. In the case of granular fills, the majority of this lateral deformation occurs during construction and soon after construction is complete, in response to the increase in gravity loads and earth pressure. During the remaining service life, outward deformations are generally relatively small, consisting of a small amount of forward tilting about the base, see Dobie & McCombie (2015).

In the case of reinforced soil construction using clay fill, the same principle applies, however there may also be deformations created by changes in water content of the mass of clay fill which forms the reinforced fill and backfill. This may be seen clearly by examining the post-construction deformation of the abutment wall shown in Figure 14, built in New Zealand using weathered Waitemata Clay, being a residual soil derived from the weathering of sandstone.

Figure 14: Bridge abutment in New Zealand built using weathered Waitemata Clay residual soil

Figure 15 shows a cross section through the abutment, and provides some of the main soil properties and design features. The clay fill is of relatively high plasticity plotting just below the "A" line on Figure 4.

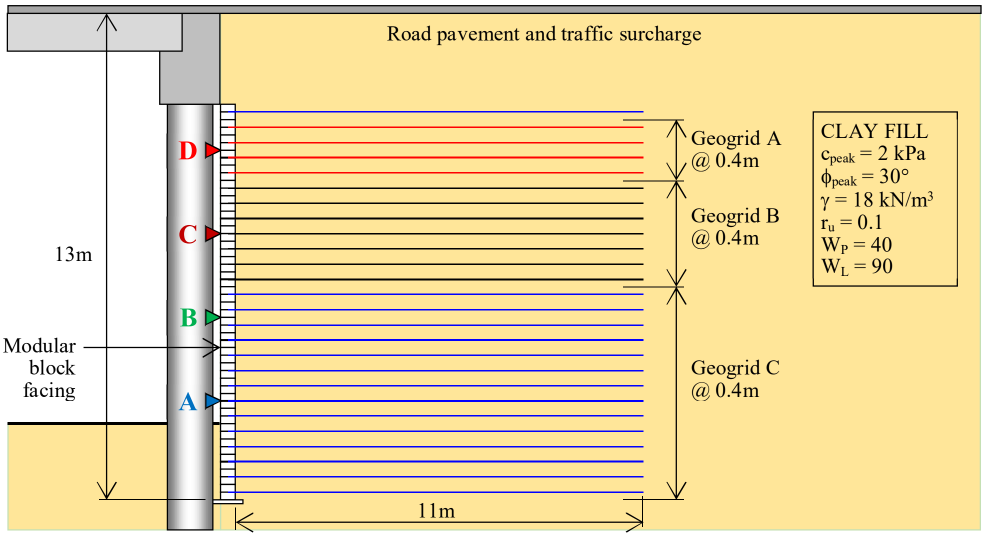

Figure 15: Cross section through the abutment shown in Figure 14

Survey targets were established on the face of the retaining wall in order to observe post-construction deformation, with D at the top and A at the base. The outward deformation behaviour over a period of more than 500 days is shown in Figure 16. It can be seen that the nature of this outward movement does not match the description given above for the case of granular fill, namely mostly occurring during or soon after construction. In this case there is a suggestion that some of the deformation may be related to wet and dry seasons, with wet seasons providing a source of water which allowed the clay to swell, taking into the account the discussion in the previous section about the likely pore water suctions which would be present after construction. It can be seen clearly that deformation starts rapidly, then slows, and then accelerates again. It can also be seen that targets B, C and D move outwards by much the same amount, so this is not tilting, but a general forward deformation. Target A is very close to the external ground level which probably provides restraint. This deformation reached equilibrium and the abutment has provided satisfactory service for many years.

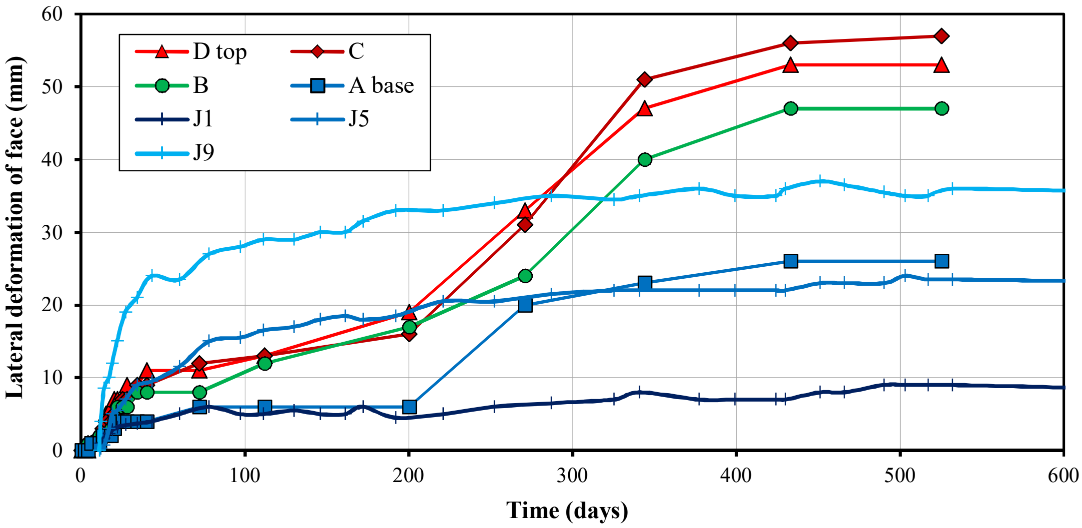

Figure 16: Post-construction outward deformation of the abutment (with comparison to granular fill – see below)

POST-PUBLICATION ADDITION: at the time of presentation of this paper on 3rd October 2017, additional information was added to Figure 16 to provide a comparison between the outward deformation behaviour of a clay fill with that of a granular fill. The behaviour indicated as J1, J5 and J9 comes from a trial wall built in Japan in 1995 published by Nakajima et al (1996) with further longer-term data after 8 years of measurements by Onodera et al (2003). The wall was 8m high, as shown in Figure 16a, constructed with sand fill and was relatively lightly reinforced. The points J1, J5 and J9 are targets on the facing located at base of wall, mid-height and top of wall respectively. The pattern of deformation behaviour measured from completion of loading indicates forward tilting of the wall, which is quite different to the pattern of behaviour seen from the clay fill abutment wall.

Figure 16a: Japanese trial wall 8m high with sand fill (Nakajima et al, 1996)

4.3 *Drainage measures*

An important series of construction details for any reinforced soil structure are the drainage measures and features used to minimise the effects of both run-off from rain and possible internal ground water flows. This is discussed in terms of two common forms of construction:

- Back-to-back retaining walls built to support highways or railways (Figure 17)
- Retaining walls or steep slopes built up against an existing hillside, where groundwater flows may be present (Figure 18)

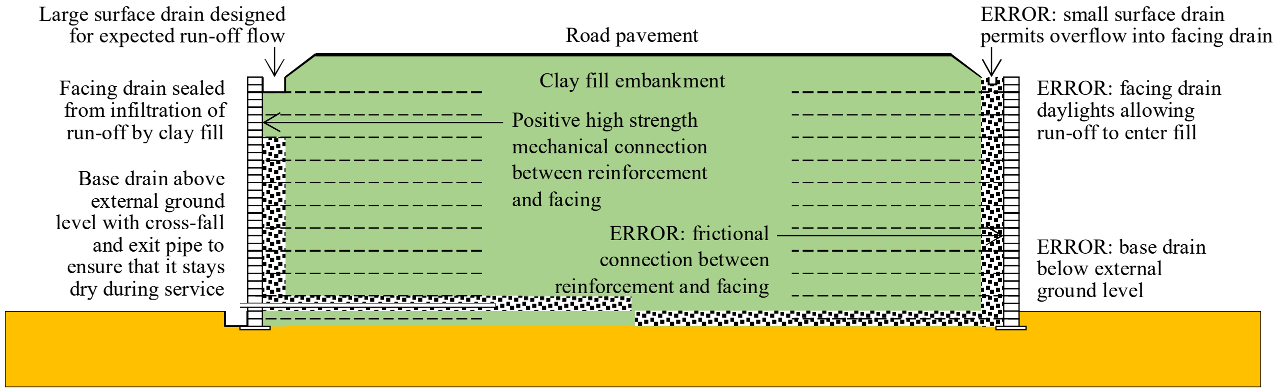

Figure 17: Typical back-to-back highway embankment with drainage features: preferred on the left, to be avoided on the right

One important feature of well compacted clays to take into account is the fact that they are effectively impermeable. The permeation of water into such materials is very slow indeed, so that free water needs to be in contact with the surface of a compacted clay for a very long time to have any significant effect. Clearly if the clay is in a state of suction, which is likely to be the case following the arguments and evidence presented in Section 4.1, then this will encourage the permeation of free water into the clay mass. It is therefore important that drainage features are detailed in such a way as to prevent long term contact between free water and the surface of the compacted clay fill, including, and importantly, at the base of the fill. It is an important principle that run-off from rainfall should be removed from the structure by adequate surface water drainage, of sufficient capacity to handle expected peak flows, which can be very high in tropical countries such as Indonesia. One final point to remember, especially when considering internal drainage, is that drains arranged to let water out of a structure can also let water in under some circumstances.

In the case of the back-to-back wall depicted in Figure 17, the only source of water is likely to be run-off from rainfall on the top surface. These flows should be channelled away from the retaining walls by adequate surface water drains, and not allowed to come into long term contact with the surface of the compacted clay. At typical highway embankment heights of 6m to 8m, the entire thickness of the clay is likely to be is a state of suction after construction, and ideally it should be kept this way. For walls of this type, there are two common drainage features which are frequently detailed: a base drainage layer and a drainage layer behind the facing.

The drainage layer behind the facing has become a common feature in the detailing of modular block retaining walls, although it is not normally used behind panel facings. However it must be asked: what is the purpose of this drainage layer? The clay fill is in a state of suction, and there are no internal flows to be intercepted. This drain could be omitted altogether, however most designers prefer to retain this feature. If this facing drain is used, then it is very important that it is not continued upwards to daylight at the top, especially if a surface water drain is located there, as depicted on the right-hand side of Figure 17. If this is done, then water overflowing the surface drain can readily run down the facing drain, coming into contact with and softening the clay fill. The detail on the left-hand side is preferred. It is normal to separate the clay fill from the gravel of the drain using a non-woven geotextile.

As regards the base drain, again the question should be asked: what is its purpose? The fill is likely to be in a state of suction and there are no internal water flows to be intercepted at the base. One reason for the base drain is to act as a capillary break, which might be important if there is a high water table in the foundation soil, providing a source of water to be “sucked” up into the clay fill. However if this is the case, then it is very important that the drain is genuinely free-draining (uniform gravel) and is kept empty of water. These base drains are often placed directly over the foundation level, which is generally below external ground level if normal embedment recommendations are followed, as depicted on the right-hand side. There is then a problem of removing any water trapped in the drainage layer. It is better to detail the base drain as shown on the left-hand side, preferably with a gradient towards the exit point, and an exit pipe to encourage any water to flow to an external drain at the toe of the wall. However care should be taken if the foundation soils are compressible and may settle, which will be greater in the centre, and might reverse any outward gradient, causing water to pond in the middle. In this situation a base drain is probably best avoided.

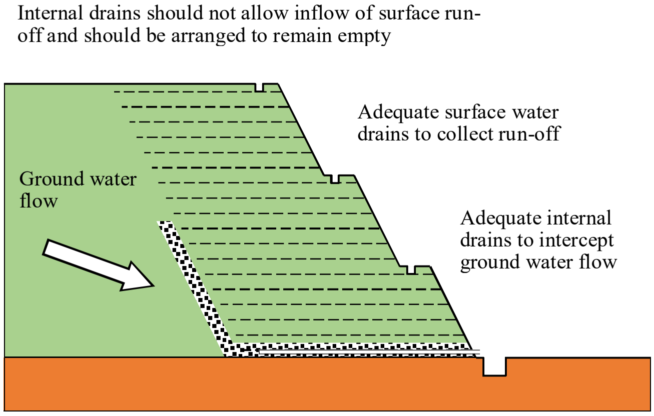

Figure 18: Drainage features for a reinforced soil slope built against an existing hillside

The case of a reinforced soil steep slope built against an existing hillside is depicted in Figure 18. It is again important that adequate surface water drains are detailed to collect surface water run-off. If internal ground water flows are expected, then they should be intercepted using an internal gravel drain as shown, but it is very important that such a drain does not daylight at the ground surface, which would permit surface water run off to come into contact with the compacted clay mass. Details such as outward gradient and exit pipes should be the same as discussed above.

5.0 Case histories of reinforced soil structures in Indonesia built using clay fill

5.1 *Approach embankment to railway bridge over the Citayam River (1997)*

This 94m long 70° degree steep slope was built in 1997 as an approach embankment to a railway bridge over the Citayam River as shown in Figure 19, a short way south of Jakarta in Java. Maximum wall height is 7.5m at the abutment, and the slope was designed to carry a live load from railway loading, as well as a seismic ground acceleration of 0.25g. The reinforced fill used was tanah merah, with a design shear strength of $\phi' = 32°$ based on appropriate testing.

Figure 19: Reinforced soil steep slope carrying railway loading built using tanah merah

5.2 *Jakarta Outer Ring Road (2001 to 2012)*

The development of the Jakarta Outer Ring Road took over 10 years to complete, but at many locations the retaining structures were built using reinforced soil techniques with tanah merah fill. One such wall can be seen in Figure 2 during construction, and the same wall is complete in Figure 20. In total some 5 km of retaining walls were built using these techniques, frequently as bridge approaches with varying heights up to 10m.

Figure 20: One of the many reinforced soil retaining walls built using clay as part of the Jakarta Outer Ring Road

5.3 *Bridge approach structure in Tangerang (2016)*

After more than 20 years of experience building reinforced soil structures using the modular block facing system with tanah merah backfill, the technique has become well known, and Figure 21 shows a typical construction in Tangerang, in this case forming an approach embankment to a bridge.

Figure 21: Bridge approach structure built in Tangerang using tanah merah backfill

6.0 Conclusions

Clay fills have been used to construct a large number of reinforced soil retaining walls and steep slopes in Indonesia. In fact the use of clay fill or other fine fills may be more common than using granular fill. Although the properties of some clay fills, such as tanah merah, are well known, a thorough geotechnical investigation should be carried out to determine index and strength properties, which are often better than assumed based on having no data. For re-compacted tanah merah it is common to design using ϕ' of around 27°, but if appropriate testing is carried out, this could well be assessed to be somewhat higher, as in the case of the steep slope described in Section 5.1.

As regards compaction control, it is likely that in most projects, the traditional limits are applied to the optimum water content and maximum dry density. However the method outlined in Section 3.0, based on minimum undrained shear strength and maximum air voids provides a very satisfactory alternative approach.

It is very important that the pore water pressure distribution in a re-compacted clay fill is well understood, and that for typical highway structures, the pore water pressures are likely to be negative, such that the clay is in a state of suction. This has important consequences with regards to the likely behaviour of the fill during service, as well as the drainage measures which might be detailed for the structure. If drainage details permit free water to come into contact with the clay mass for extended periods of time, then significant swelling may occur with subsequent outward deformation and loss of undrained shear strength taking place, both of which are undesirable.

However the economic benefits of being able to use readily available clay fill compared to expensive imported fills are likely to be significant, which is confirmed by the widespread use of clay fills in Indonesia.

Acknowledgements

Many of the papers and publications of Dr Laurie Wesley form an important resource for geotechnical engineers in Indonesia, and this paper has made extensive use of some of these references. I was fortunate to meet Dr Wesley for the first time about 25 years ago in Auckland, and have kept in close contact ever since. His visits to Indonesia continue to this day, allowing engineers in Indonesia to benefit at first hand from his wisdom and knowledge of the soils of Indonesia. I would also like to acknowledge the help of colleagues at PT Multibangun Rekatama Patria who provided the information about the projects described in Section 5.

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