



Serviceability limit state check in reinforced soil design L'état limite en service pour la conception des murs en sol renforcé

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ABSTRACT Published design methods for reinforced soil structures concentrate almost entirely on analysis of the ultimate limit state. Most design guides give general requirements that settlements and deformations should not lead to a serviceability limit state, but little guidance is given as to how such assessments should be made. This paper describes a method of analysis based on the use of multiple two-part wedge mechanisms to predict a load distribution for each layer of reinforcement. This is then combined with information from isochronous load-strain curves for the reinforcement, in order to predict the likely distribution of post-construction strain. BS 8006-1:2010 provides guidance on post-construction strain limits, which are then compared to the predictions from the two-part wedge analysis. This provides an additional verification of the design layout established by the ultimate limit state check. The method is illustrated by examining the behaviour of an 8m high trial reinforced soil retaining wall built in Japan in 1995, and monitored for 8 years. Comparison of the actual wall performance with predictions made using the two-part wedge method gives good agreement.

RÉSUMÉ Les méthodes de conception publiées pour les structures en sol renforcé se concentrent sur l'analyse de l'état limite ultime. La plupart des guides de conception donnent des exigences générales sur les tassements des fondations et les déformations qui ne doivent pas conduire à un état limite en service, mais peu d'indications sont données sur la façon dont ces évaluations devraient être effectuées. Cet article décrit une méthode d'analyse basée sur l'utilisation de plusieurs mécanismes de deux blocs qui permet de prévoir une répartition des charges pour chaque couche de renforcement. Il est ensuite combiné avec les courbes de charge-déformation isochrones pour le renforcement, afin de prédire la répartition probable des déformations après la construction. BS 8006-1: 2010 fournit des indications sur les limites de déformation post-construction, qui sont ensuite comparées aux prédictions de l'analyse. Ceci fournit une vérification supplémentaire de la conception établie par la vérification de l'état limite ultime. La méthode est illustrée en examinant le comportement d'un mur de soutènement expérimental de 8m de haut en sol renforcé, construit au Japon en 1995, et un monitoring pendant 8 ans. La comparaison de la performance du mur réel avec les prévisions faites en utilisant la méthode de blocs en deux parties donne un bon accord.

1 INTRODUCTION

Design methods for reinforced soil structures published in most design guides today concentrate mainly on preventing failure through an ultimate limit state (ULS), using limiting equilibrium analysis with either a lumped safety factor or partial load and material factors to provide a margin against failure. Most design guides give general requirements that settlements and deformations should not lead to a serviceability limit state (SLS), but little guidance is given as to how such assessments should be made. In this situation, some of the potential serviceability limit

states may well be addressed by applying unnecessarily high ULS safety factors, possibly creating over-conservative designs.

This paper describes a method of analysis used to predict post-construction creep strain of polymer reinforcement in reinforced soil retaining walls. The method is based on the use of multiple two-part wedge mechanisms to predict a load distribution for each layer of reinforcement. This load distribution is then combined with information from isochronous load-strain curves for the reinforcement, in order to predict the likely distribution of post-construction strain.

BS 8006-1:2010 is a limit state code for the design of reinforced soil retaining walls which provides detailed guidance on construction tolerances and serviceability limits in Section 6.5.5, including limits on internal creep strain of polymer reinforcement. Figure 43 of BS 8008-1:2010 illustrates how the SLS base strength (T_{CS}) is assessed based on isochronous load-strain curves which are derived from creep test data, and this is reproduced as Figure 1 below. T_{CS} is the load which limits post-construction strain to 1% for retaining walls and 0.5% for bridge abutments.

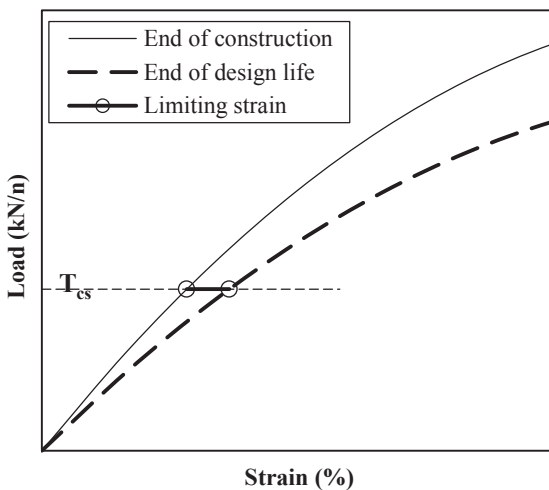


Figure 1. Assessment of SLS base strength (BS8006, Figure 43).

Table 1 provides a full summary of all parameters necessary to define T_{CS} . It should be noted that specific programmes of creep testing at low loads are required to establish the isochronous curves as depicted on Figure 1.

Table 1. Parameters used to define T_{CS} according to BS 8006, combined with BBA HAPAS Certificate 13/H201.

Type of structure	Retaining wall	Abutment
End of construction	1 month	2 months
End of design life	120 years	120 years
Post-construction strain	1.0%	0.5%

2 TWO-PART WEDGE METHOD OF CALCULATION APPLIED TO SLS

The basis of the two-part wedge method of calculation is shown on Figure 2 (left). The geometry is typical of reinforced soil structures, but the method of analysis can incorporate all features shown (ie. berm, slope above the wall, isolated surcharge) without the need for any simplifying assumptions. The two-part wedge is defined by fixing a distance z_i below the top of the wall, then drawing a line at an angle θ_i across the reinforced soil block, defining Wedge 2. Starting at the point where Wedge 2 intersects the back of the reinforced soil block, Wedge 1 is defined as shown, with the inter-wedge boundary at the back of the reinforced soil block (RSB).

H_i is then changed and a further series of wedges are checked at various values of θ_i

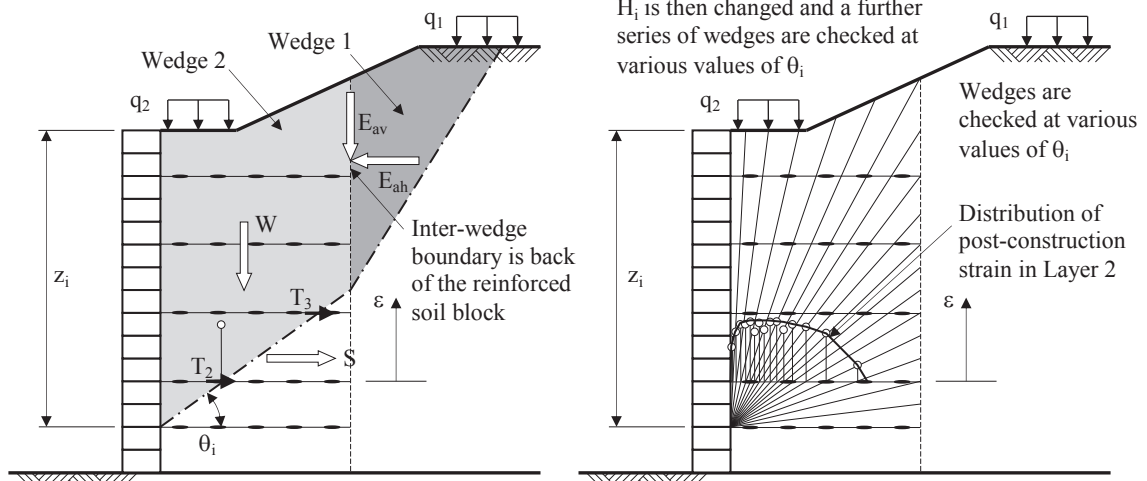


Figure 2. Two-part wedge method of calculation used to assess the distribution of post-construction strain in a layer of reinforcement.

The method of calculation is summarised as a series of steps, starting from Figure 2 (left):

- A search is carried out to find the angle of Wedge 1 which applies maximum earth pressure to the back of the reinforced soil block.
- The forces applied to Wedge 2 (earth pressure from Wedge 1, self-weight, surcharges) are resolved to find S , the force required to stabilise the two wedges, provided by the reinforcement.
- Wedge 2 intersects two layers of reinforcement in the case shown on Figure 2, so S is distributed between these two layers in proportion to their relative stiffness.
- This gives a value of load in the second layer of reinforcement, shown as T_2 , applied at its point of intersection with Wedge 2.
- The strain at this point is calculated based on Figure 1 as $\varepsilon = (T_2/T_{CS}) * 1\%$ which is then plotted as one point on a strain distribution diagram as shown (this assumes a linear relationship between reinforcement load and post-construction strain, which is reasonable for polymer reinforcement at load loads).

The remainder of the method of calculation is shown in Figure 2 (right) as follows:

- The angle of Wedge 2 (θ_2) is adjusted, and the procedure above is repeated many times creating a family of wedges as shown. Each value of θ_2 will create a strain value on the distribution as shown by the round symbol.
- This procedure is repeated at various values of z_i , normally at each geogrid level as well as from the base of the wall, so that in the case of the second layer of reinforcement as shown on Figure 2, two fans of wedges would be checked.
- Once completed, an algorithm is used to construct a distribution of strain which is based on the maximum values calculated, so that some of the calculated values shown by the round symbols may well be below the envelope.
- The final step in the calculation is to assess a mean strain in each layer of reinforcement, taking into account its full length, and to check that this maximum is less than the limit given in BS 8006-1:2010 based on the type of structure being designed according to the parameters in Table 1.

In limit state design methods, the margin against failure in the ULS is normally provide by applying

either partial load factors, or partial material and resistance factors. For most published methods partial factors are all set to 1.0 for the SLS, with the exception of live load, where guidance in different codes may vary. For example BS 8006-1:2010 sets the partial load factor on live load to 0 for SLS, whereas AASHTO/LRFD sets it to 1.0.

The method used to assess post-construction reinforcement strain described above is examined in the following section by making a comparison with actual performance data measured for an 8m high vertical reinforced soil retaining wall.

3 JAPANESE TRIAL WALL 1995

A trial reinforced soil retaining wall was built in Japan in 1995, and has been reported by several authors in a number of papers. Some of the earliest information was provided by Nakajima et al (1996) and later Tsukada et al (1998). The wall is 8m high with a vertical precast concrete block facing. A sand fill was used with a reported $\phi' = 29^\circ$ and $\gamma = 18.6 \text{ kN/m}^3$. The fill was reinforced with 11 layers of a relatively low strength HPDE reinforcement, manufactured by a process of punching and stretching. All layers are 6m long and have the same strength, with a layout as shown in Figure 3 (left).

The reported fill strength of $\phi' = 29^\circ$ is assumed to be ϕ'_{cv} based on the fill description. ULS analysis of the structure using $\phi' = 29^\circ$ indicates a very low margin against failure. Bearing in mind the very good performance of the wall, together with the fill description, it is assumed that ϕ'_{peak} may well be considerably higher, and in the SLS calculation presented below, $\phi'_{peak} = 37^\circ$ has been used.

The trial wall was built at a testing facility and is not part of a permanent structure. A uniform surcharge of 9.8 kPa was placed on top of the fill, which is considered to be a dead load. The published papers describe extensive instrumentation which was installed during construction, and was monitored for several years after construction was completed. Instrumentation included measurement of lateral wall movement, reinforcement strain, vertical settlement and earth pressure. More recent information was presented by Onodera et al (2004), providing lateral wall movement data after 8 years.

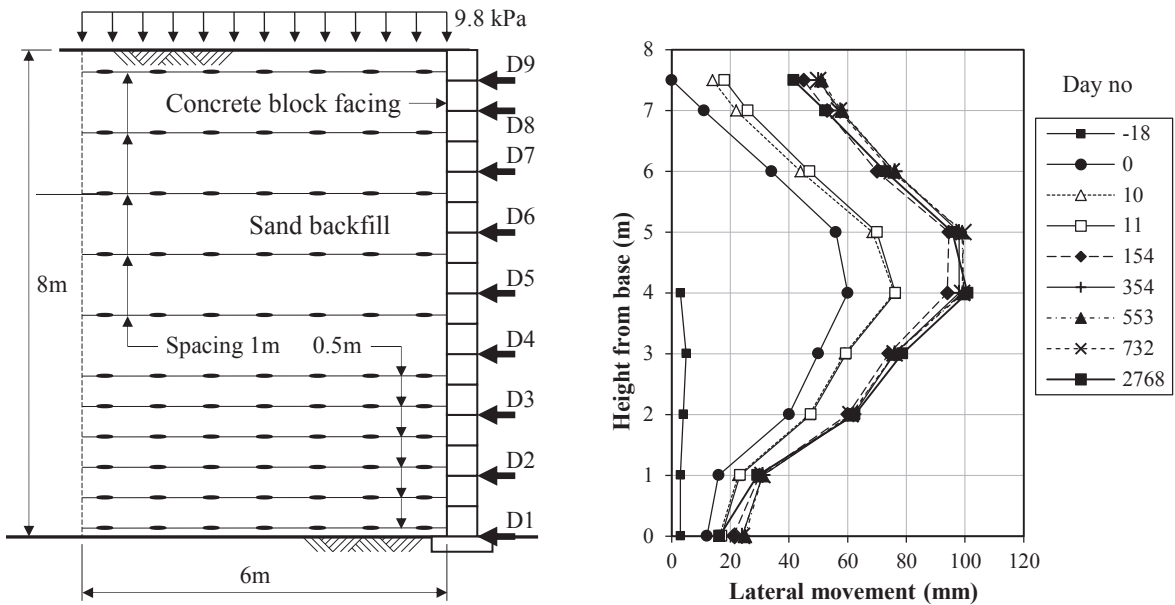


Figure 3. Section of Japanese trial wall together with profile of lateral movement as normally reported.

Figure 3 (left) shows a detailed cross section of the wall indicating the positions of the targets set up to measure lateral wall movement (D1 to D9), at 1m intervals up the face of the wall. Figure 3 (right) shows a profile of lateral movement where each set of data represents a different number of days since completion of the wall. Day 0 = end of filling, actu-

ally 28th May 1995. The data for Day 2768 (approximately 8 years) was added to the earlier data by Onodera et al (2004). The immediate impression gained from Figure 3 (right) is that the wall is bulging, and this is often how it is described in papers or presentations. However in the Authors' opinion this description and method of presentation are misleading.

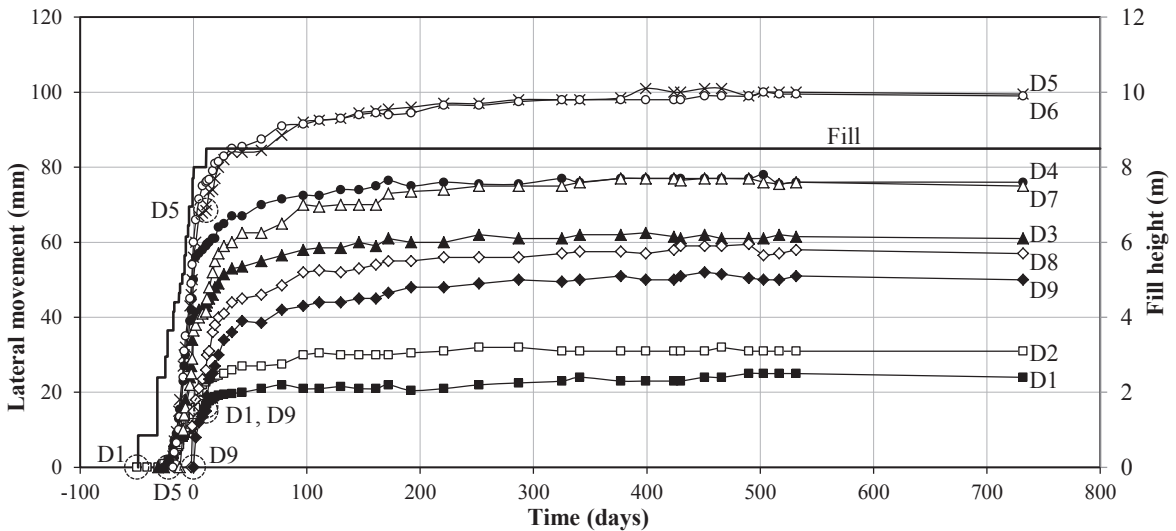


Figure 4. Japanese trial wall record of lateral movement and construction history versus time

Figure 4 presents the detailed lateral deformation data against time, together with the construction history of the wall as fill height in metres (the final 0.5m represents the surcharge). The wall facing consists of 0.5m high concrete blocks, placed incrementally as the fill was raised. This means that deformation readings could only start once any specific level was reached. Therefore D1 (lowest point) was measured from Day -50, D5 (largest reported lateral movement) from Day -22 and D9 (top point) from Day 0 as indicated by the large circles on Figure 4. This means that each lateral deformation record starts from a different point in time, which is the reason why the wall appears to bulge in Figure 3 (right).

Day 11 represents the completion of placing the surcharge (ie. completion of applying permanent load). The lateral movements of D1, D5 and D9 on Day 11 are indicated by large circles on Figure 4, and it can be seen that by Day 11, most of the deformation at D5 (about 68mm) has already taken place. Figure 5 shows the profile of lateral movement plotted taking Day 11 as the starting point in time, and in this case, by Day 732d (about 2 years later, with “d” denoting deformation), the wall has tilted forwards slightly, by about 30mm at the top, but has remained essentially straight, and there is no noticeable sign of bulging.

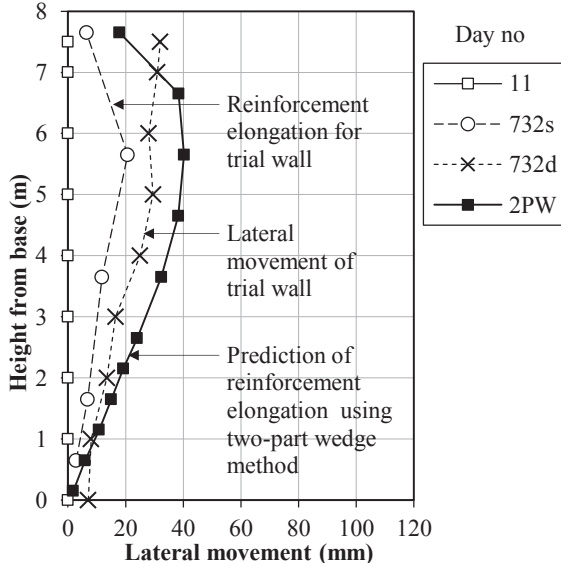


Figure 5. Profile of lateral movement and reinforcement elongation measured from Day 11 (end of loading).

The instrumentation records include strain of the reinforcement measured at 5 levels. The detailed results for the layer of reinforcement 1.65m above the base of the wall are shown in Figure 6, with each record corresponding to a specific day, including Day 11 and Day 732. If the recorded strain between Days 11 and 732 is summed over the length of the reinforcement, then this presents an increase in length of 6.8mm. This interpretation may be carried out for all five instrumented layers of reinforcement.

The reinforcement elongation values are also plotted on Figure 5, denoted by 732s (“s” denoting strain). It can be seen that the plotted values are slightly lower than the wall outward movement, as would be expected. The exception is the top point, where the outward movement may have a contribution from pull-out due to the very low overburden pressure.

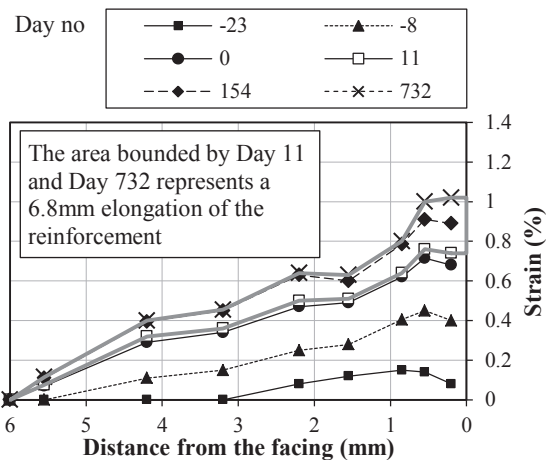


Figure 6. Distribution of strain in the reinforcement layer 1.65m from the base at various time intervals.

Based on the cross-section shown in Figure 3 (left) and the data presented and discussed earlier in this section, a post-construction strain assessment has been carried out using the two-part wedge method described in Section 2. The result of this calculation is a predicted mean post-construction strain which may be interpreted as elongation of the reinforcement by taking into account its length of 6m. The resulting profile of predicted reinforcement elongation has been added to Figure 5 (denoted as “2PW”), where it has a similar distribution to the measured values, but with approximately twice the magnitude.

4 DISCUSSION AND CONCLUSIONS

BS 8006-1:2010 provides guidance for carrying out serviceability limit state checks for reinforced soil retaining walls, including defining limits on post-construction reinforcement strain. However the code does not prescribe or define a method of calculation in order to establish the likely magnitude of post-construction strain for any particular design situation. Section 2 of this paper describes a method of calculation based on the use of multiple two-part wedge mechanisms to predict a load distribution for each layer of reinforcement. This load distribution is then combined with information from isochronous load-strain curves for the reinforcement, in order to predict the likely post-construction strain, which may then in turn be compared with the guidance from BS 8006-1:2010.

In order to examine this method of calculating post-construction strain, the measured behaviour of an 8m high trial reinforced soil retaining wall built in Japan in 1995 is summarised in Section 3. On first examination of the published data, the deformation appears to be excessive, indicating a mid-height bulge in the facing, and the two-part wedge method of analysis would under-predict the observed maximum lateral deformation. However closer examination of the data permits it to be re-interpreted in terms of post-construction deformation and strain, giving a consistent picture, with magnitudes slightly smaller than those predicted by the two-part wedge method. The actual post-construction deformation of the wall is a 30mm forward tilt at the top about the base, with the wall remaining essentially straight. Over the same 2 year time period, the two-part wedge method indicates a maximum reinforcement elongation of about 40mm, with post-construction strain remaining well within the limits given by BS 8006-1:2010.

The slight over-prediction of reinforcement elongation may be due to a number of causes, possibly conservatism in some of the parameters being used and possibly due to the observation that the combination of reinforcement and soil used to form such a structure has a composite behaviour which is significantly greater than the simple sum of the two components. However the method offers an important check which is independent of the ULS calculations,

providing an indication of the likely upper bound of post-construction deformation of the reinforcement.

Another technique which may be used to assess likely deformation of a reinforced soil structure is the finite element method (FEM). With sophisticated soil models and appropriate parameters, this has the potential to predict overall deformation of the structure. However there are aspects of modelling polymer reinforcement using FEM which require special attention. This can be seen in relation to Figure 3 (right). At the practical level, all deformation up until Day 11 is a construction issue, and any such deformations should be addressed by the construction techniques used, and therefore by the contractor. The owner of the wall is interested in the deformation which takes place after Day 11, namely during service. However between Day 11 and the end of design life, load is constant. In order for FEM to make a meaningful prediction of deformation during service, the model for reinforcement behaviour should incorporate time-dependent stiffness (as per Figure 1).

There appears to be a general view that many existing design methods for reinforced soil structures are over conservative, and techniques might be developed to reduce the quantity and grade of reinforcement required. In this situation, an independent SLS check based on an assessment of post-construction strain becomes even more important.

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