Fifteen years of geotechnical limit state design in Australia Part I – Soil retaining structures

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ABSTRACT

Design of soil retaining structures in Australia to limit state design codes is currently a veritable mine field. Three separate codes with quite different approaches exist, with the selection of the appropriate code depending on where the retaining structure is sited. It is the authors' contention that the differences in approach of the three codes arises in part due to the complexities of applying limit state design to soil retaining structures. The paper gives a brief summary of the history of limit state design codes in Australia, the design principles behind limit state design, and some of the key aspects of retaining structure design where a lack of understanding of soil characteristics can lead to unrealistic designs. It concludes that there is definitely a place for limit state design in soil retaining structures, but there is still much work to be done to develop consistency of approaches that adequately address the probability of failure.

1 INTRODUCTION – A BRIEF HISTORY OF LIMIT STATE CODES IN AUSTRALIA

Limit state design was introduced to structural engineering in Australia and Europe in the mid 1980's, however its introduction to geotechnical engineering was slower. The first Australian Code to implement geotechnical limit state design methods was the 1992 Austroads Bridge Design Code (known as "Austroads 92"). This code was one of the first in the world to apply limit state design methods to a wide range of geotechnical applications. By way of contrast, a uniform approach to limit state design of geotechnical engineering only became codified in Europe in 2004 with the introduction of Eurocode 7 part 1.

Following on from Austroads 92 came the AS2159-1995 Piling Code which was an evolution of the Austroads 92 code, but retained many of its underlying principles, albeit with more scope introduced for engineering judgement. Since 2005, AS2159 has been undergoing major re-drafting and currently exists in draft form as DR05506.

In 1996 The New South Wales RTA first published a mandatory QA Specification R57 for the design of reinforced soil walls on NSW road projects. It is understood that the main reasons for introduction of this specification were a perceived inadequacy of the then-current Austroads 92 to address reinforced soil design. R57 in its current form (2005) has subsequently been adopted by Queensland Main Roads for design of reinforced soil walls on Queensland road projects.

AS4678-2002 Earth Retaining Structures was the next code to adopt limit state design methods. This code has many similarities to Austroads 92 when it comes to retaining structures other than reinforced soil structures, although some of the partial factors are altered. The code also devotes a significant part of its bulk to addressing construction details for retaining walls, and in particular, provision of adequate drainage within and around retaining walls. This emphasis was on the basis that poor drainage detailing is widely recognised to be the single biggest trigger for soil retaining structure failures. With respect to reinforced soil walls, R57 and AS4678 are similar. However, while AS4678 gives little guidance on the specifics of the calculations, R57 lays out a detailed design method to be followed including a specification of how design parameters are to be tested.

Ironically, Austroads 92, which started the limit state trend, has since been replaced with AS5100-2004 Bridge Design. In the design of "soil supporting structures", AS5100 moves almost completely away from the limit state design methods of Austroads 92 and returns to an approach more akin to the traditional working stress methods. One of the principal reasons for this shift was that the strict application of Austroads 92 resulted in some classes of soil retaining structures being significantly more robust than similar structures designed using traditional working stresses.

The up-shot of all this is that currently in Australia we have three different limit state design codes with quite conflicting requirements, depending on where the soil retaining structure is:

- AS5100-2004 for any retaining structure on a transport infrastructure project other than reinforced soil walls;
- RTA Technical Specification R57 for reinforced soil walls on NSW or Queensland road networks; and
- AS4678-2002 for anything else

2 PRINCIPLES OF LIMIT STATE DESIGN

In order to understand the difficulties with applying limit state design to geotechnical engineering problems it is useful to have an understanding of the underlying principles of limit state design. Limit state design was introduced to structural engineering principally to address two perceived deficiencies with the working stress methods in use at the time.

At one end of the spectrum there were instances of catastrophic failure occurring under extreme, low probability events. At the other end of the spectrum, measurements of stresses in medium and high rise buildings with multiple fixed columns and continuous floor slabs often showed stress distributions and magnitudes that were very different to the stresses predicted by the designers. It could be demonstrated that redundancy in the structure could resist much higher loads.

Out of these issues came the concept of an Ultimate Limit State (ULS) in the structure for strength design. There are several slightly different definitions of the Ultimate Limit State, but the definition most useful to this discussion is the definition in Austroads 92, which states it is: “(a) a limit state at which a mechanism is formed in the ground; or (b) a limit state which involves a loss of static equilibrium or rupture of a critical section of the structure due to movements in the ground”

The importance of this definition with respect to the deficiencies of the working stress methods was that it introduced the ability to allow a part of the structure to yield and redistribute loads to other parts of the structure, provided this happens in a ductile way and didn’t lead to a collapse. For highly redundant structures this could result in increased efficiency of designs, while less redundant structures generally required increased strength to resist the loads under worst credible conditions of loading and strength.

Equally important to the ultimate limit state is the Serviceability Limit State (SLS). The loadings for the SLS state are similar to the old working stress loads, however the strength of the structure is not directly considered. Rather the acceptance criteria relate to the structure being able to perform its intended function in terms of deflections, crack widths, appearance etc.

Although the terms and symbols vary from code to code, the fundamental approach of all limit state design codes is that under the ultimate limit state, the applied loads/actions (S) are increased by a load factor (Ψ) to give a design action with a very low probability of occurrence (S*), while the resistances (R) are reduced by a strength factor (Φ) to give a design strength (R*), which must be greater than or equal to the design action. This is shown in the widely recognisable formula below:

$$S^* = \Psi S \leq \Phi R = R^* \qquad (1)$$

For the simple case of a load and resistance with a probability distribution, the adopted loads and resistances used as a starting point for limit state design are “characteristic values”. A typical definition of a characteristic value of resistance is “a value with a likelihood of less than 5% of a more adverse value occurring”. However this is not uniformly adopted. For example AS4678 adopts the definition with respect to geotechnical resistance that it is “a cautious estimate, that is, close to but not greater than, of (sic) the mean value”. Comparison of various code factors suggests that design to the different codes results in a different probability of failure.

With respect to selection of the characteristic loads, the aim is generally to select the worst credible combination of loads. For example, a three span bridge, continuous over two piers, might consider full traffic loads on all spans to get the maximum hogging moment over the pier, but zero loads on the middle span to get the maximum hogging moment at the centre of the middle span.

Figure 1 shows a probability distribution for a simple load case with an upper and lower characteristic load ranging from a positive to a negative value, requiring two modes of resistance.

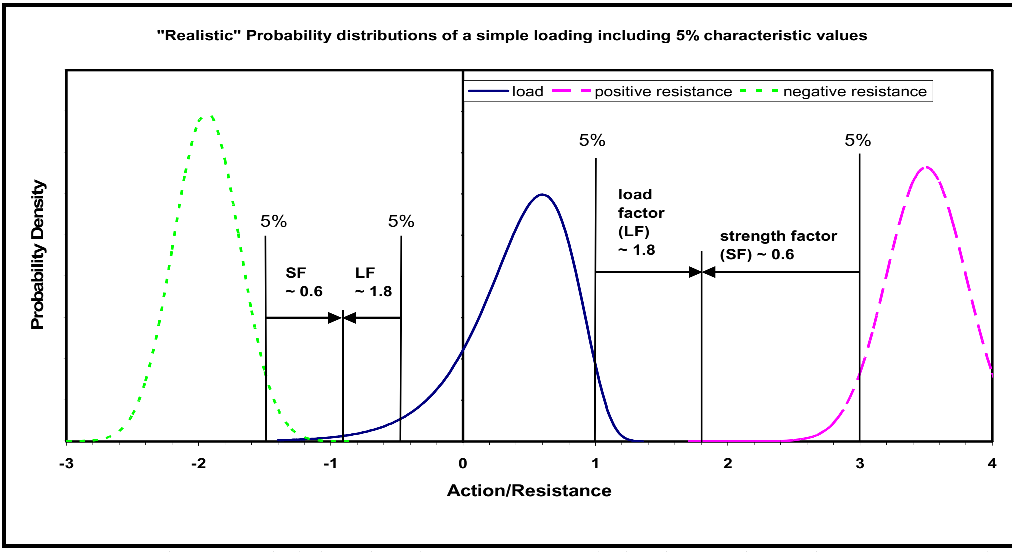


Figure 1 - "Realistic" probability distribution of a simple loading with 5% characteristic values

It is important to note that there are two very significant assumptions in adoption of the ULS design approach which have the potential to significantly impact on the validity of the method when applied to complex problems. Firstly, it is assumed that multiplying a design value by a strength or capacity factor results in a consistent probability of occurrence. This implies that the variance of a parameter is directly proportional to the characteristic value of that variable, and is constant for a wide range of possible actions and resistances. Figure 1 demonstrates some of the deficiencies of this assumption and shows that the closer the characteristic value is to zero, the smaller the offset produced by the load and strength factor, resulting in a bigger probability of overlap.

Secondly, for a probability calculation to work there is an assumption that the actions and resistances are independent variables, i.e. that changes to the loading do not influence the resistances and vice versa. As will be discussed further below, in the design of soil retaining structures this is not true because the same soil acts as a load and a resistance.

3 FACTORS OF SAFETY IN GEOTECHNICAL ENGINEERING

The concept of checking for the formation of a mechanism is by no means a new concept to geotechnical engineers. The bearing capacity of a foundation, slope stability and sliding of a retaining wall are all examples of a calculation that allows formation of progressive failure until a mechanism forms. The traditional approach is to calculate the factor of safety (FOS) against a mechanism forming. The acceptable FOS is however a complex issue and various acceptable FOSs have been developed for differing failure mechanisms based on a combination of theory, judgement and experience. For example a factor of safety of 1.3 to 1.5 might be acceptable for a slope stability analysis but a bearing capacity FOS might be 2.5 to 4.

One reason for these variations is that the failure mechanisms are complex and sometimes highly non-linear. For example the bearing capacity of a typical footing on sand with a friction angle of 45 degrees is more than double the same footing on sand with a friction angle of 40 degrees.

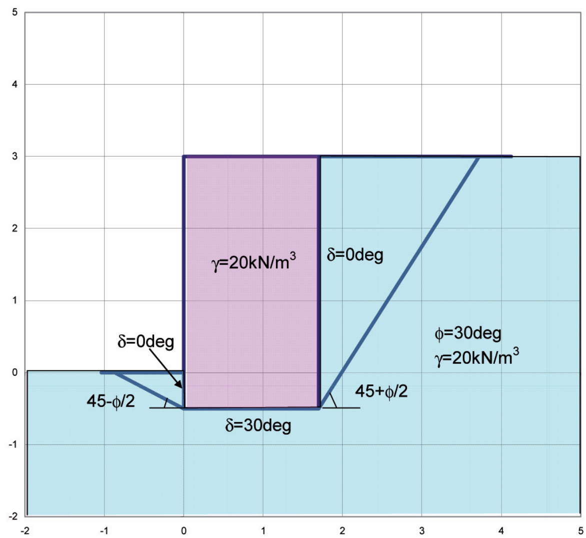


Figure 2 – A simple gravity wall problem

Additionally, ultimate capacities divided by a factor of safety, termed the allowable capacity, are often selected based on achieving semi-empirical estimates of acceptable settlement rather than a probability against failure. For example, many of the design charts for allowable bearing pressures on sands in text books are based on achieving a target settlement of 25mm.

Given these complexities in assessing acceptable factors of safety, it should not be surprising that attempts to apply limit state design to these problems can lead to some unexpected results.

4 POTENTIAL PROBLEMS WITH COMBINING FACTORS

Strength and load factors are supposed to be an approximate method of representing the probability of a producing a parameter less than (or greater than) a design value. However, when there are a large number of parameters or a large number of variables influencing a parameter, design results can be very sensitive to the way in which factors are applied.

Strength reduction factors on geofabric reinforcement in a reinforced soil wall are an example of this. AS4678 has multiple strength factors on the tensile strength of the reinforcement to take into account variables such as aging, installation damage, etc. The code requires that the design resistance is equal to the characteristic strength of the material multiplied by each of these factors in series.

The problem with this approach is that basic statistics demonstrates that the probability of such a design value occurring is miniscule. For example, if a strength factor of 0.8 is meant to represent a 0.1% (0.001) probability of a more adverse value occurring, then if there are five independent strength factors of all equal to 0.8, the resultant strength factor is $0.8^5 = 0.3$, and the probability of occurrence (assuming all have an equal probability) is $0.001^5 = 1x10^{-15}$, which is an unrealistically low probability of occurrence for design, particularly remembering that the loads will also be factored up.

This simple example demonstrates the importance of understanding the underlying probabilities associated with the various parameters and the likelihood of them occurring concurrently. The arbitrary use of multiple strength/load factors can lead to unrealistically low probabilities if the relationships between parameters and the variables influencing them are not properly understood.

5 PROBLEMS WITH COMBINING LOAD AND STRENGTH FACTORS FOR EARTH PRESSURES

5.1 Gravity Walls

Using the design approach of AS4678 or Austroads 92, the earth pressures on a gravity wall are calculated by a two step approach. Firstly, because active and passive earth pressures are calculated from soil strength parameters, these strengths are reduced by a strength factor before calculating the earth pressure. For example, to satisfy the requirements of Austroads 92, the internal angle of friction of a soil used to calculate active and passive earth pressures and sliding resistance had to be reduced to:

$$\phi^* = \mathrm{atan}(0.8\tan(\phi)) \qquad (3)$$

Secondly, because the earth pressures are considered a load, active earth pressure has to be increased by a load factor of 1.25, passive pressure reduced by a load factor of 0.6, and self weight of the gravity wall used for sliding on the base reduced by a factor of 0.8.

The result of this for the simple case given in Figure 2 is that a 35% wider gravity wall is required to satisfy sliding. Experience shows that applying Austroads 92 to retaining walls supporting a back slope rapidly degenerate until it is difficult to satisfy sliding stability.

The problem that becomes readily apparent with the Austroads 92 approach is that applying both a strength and load factor to the same material double compensates for variations in parameters since the loads and resistances are not independent.

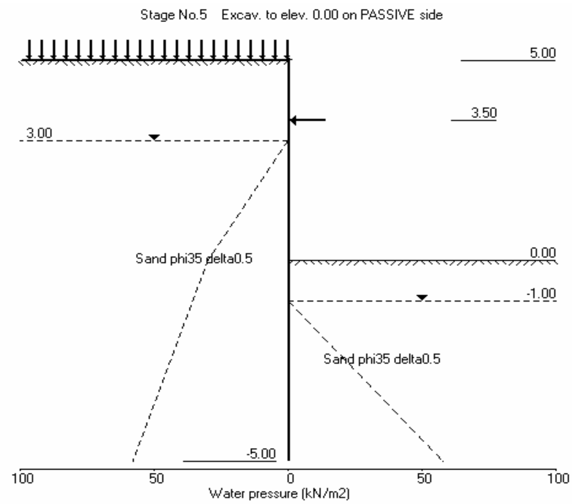
It was primarily because of observations such as this, as well as issues with flexible walls discussed in the next section, that the limit state method was largely abandoned for retaining walls in AS5100. However, the problem with completely abandoning the limit state approach is that the identification of low probability events is not considered.

5.2 Flexible Walls

Under Austroads 92, or AS4678, earth pressures acting on a flexible retaining wall are treated the same way as on a gravity wall as described above. However, with flexible walls, an important aspect of the design is identifying the maximum bending moments in the wall. Figure 3 shows a typical example of a simple single propped sheetpile in medium dense sands below the water table. The computed moment distribution in the sheetpile is shown for four cases: unfactored strength; strength decreased by a factor of 0.8; strength increased by a factor of 1.2; and strength reduced by a factor of 0.8 plus the addition of load factors of 0.6 and 1.25 on active and passive pressures respectively.

The first important thing to note from Figure 3 is that the application of the strength factors alone in this example leads to about a 30% increase in the maximum moment, but also changes the distribution of moments, so that the back-face moment is almost doubled, the locations of peak moment change, and the direction of moment is reversed at some depths.

This observation highlights a potential problem with the AS5100 ultimate limit state approach, which calculates the moment using unfactored strengths and then multiplies the resultant moment by a load factor of 1.5 to give a ULS design action. For a steel sheetpile with constant section properties, a variation in soil strength in this instance would still result in an adequate design. However, if this were a concrete diaphragm wall with reinforcement provided to match the moment distribution, using un-factored strengths could potentially result in the wall being under-designed. This type of issue is the very essence of what limit state design is meant to address. By returning to what is essentially a working stress design, the benefits of limit state design are lost.



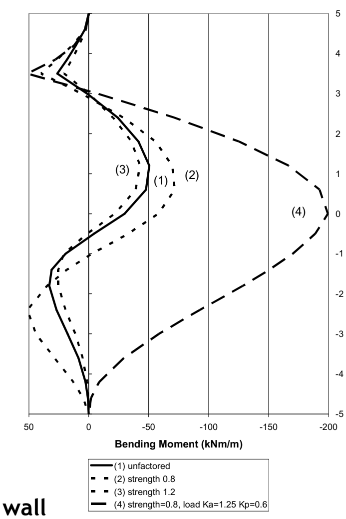


Figure 3 - Effect of strength factors on a typical sheetpile wall

The second thing to note from Figure 3 is the dramatically different moment distribution resulting from application of both strength and load factors together. The resulting peak moment is four times the unfactored moment, and the distribution of moment is almost unrecognisable compared with the unfactored distribution representing a "best estimate" of the moments that would be encountered in the wall. Once again, if this were a diaphragm wall, a strength design based on this distribution would excessively over-design for the front-face moments while potentially under-designing the back-face moments. The retaining wall problem shown in Figure 3 is a relatively simple, single-propped wall where only one parameter, soil strength, is varied. Parametric studies carried out on large multiple-propped diaphragm walls show that variations in soil stiffness, insitu horizontal stress, Poisson's ratio, wall stiffness, prop stiffness, groundwater pressures, construction sequence, and time-dependent effects on strength/stiffness can all have significant effects on the moment distribution in the wall both in the serviceability limit state and at the ultimate limit state. Furthermore, with many of these parameters it is higher rather than lower parameters than produce the most adverse effects. Despite this, no limit state code puts a partial factor on stiffness.

6 CONCLUSIONS

This paper presents some of the pitfalls and deficiencies with the current state of the art in Australian limit state design codes applicable to soil retaining structures. One of the key concerns is that we do not have a consistent set of design standards, with very different approaches currently adopted by the two applicable Australian Standards. The combined use of load and strength factors in the calculation of earth pressures adopted by AS4678 can potentially lead to unrealistic retaining wall designs. However, with respect to soil retaining structures, the AS5100 approach does not embrace a limit state design philosophy of designing for worst credible combinations of actions and resistances, consistent with the remainder of the AS5100 code. There would seem to be an argument for adopting a middle ground between these two codes which considers a parametric study of the sensitivity of designs to various parameters and the probability of occurrence of these combinations.

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