SOME DESIGN AND CONSTRUCTION ISSUES IN DEEP EXCAVATIONS AND SHORING DESIGN

Qijing Yang
Regional Technical Director, Hyder Consulting Pty Ltd
Level 5, 141 Walker Street, North Sydney, NSW 2065

ABSTRACT

This paper presents, firstly, a critical review of the current codes of practice on design and construction for retaining structures. The key factors impacting on the design and construction of shoring system is examined and discussed in detail. The importance of groundwater pressure on the design of a retaining structure is highlighted, which is as much as three times of the active soil pressure. Some design issues with respect to the SLS and ULS load combination and associated analysis are discussed through practical examples. A real case history in Sydney due to a burst water main behind the retaining wall is presented to demonstrate the significance of the groundwater pressure on deep excavation shoring design. A detail design approach is proposed as to how best the retaining wall structures could be designed to take account of the accidental water pressure induced by a burst water pipe. It is the Author's opinion that this proposed approach will provide useful guidelines to the future retaining structure design.

1 INTRODUCTION

With the increase of urban development deep excavation shoring is one of the most common design tasks in modern civil engineering. There has been a great deal of development in code of practice for retaining structures since the British Code of Practice – Retaining Structure (CP2) in 1951. The earlier codes were based on working stress method which is based on a minimum of factor of safety (FoS) of resistance against driving force of a structure for a potential failure mechanism. A factor of safety for each failure mechanism determined from observed performance of built structures is to cater for the likely variations and uncertainties of key facts affecting the retaining structure. The modern codes are mostly based on ultimate limit state (ULS) design with partial material and load factors such as codes for bridge AS5100 and retaining structure AS4678, with some using geotechnical strength reduction factors such as AS5100.3 where the dominant loading is from soil and water.

This paper describes the fundamental design philosophy of a number of current codes and outlines the limitations of their applications, in particular in the finite element analysis using commercially available programs. A number of project cases will be further discussed on some of the key factors that are required for the retaining structure design. In the end a proposed design approach is presented for the consideration of a retaining structure under accidental water pressure.

2 KEY FACTORS AFFECTING RETAINING WALL DESIGN

2.1 ULS DESIGN PHILOSOPHY

The philosophy of the current Australian code with respect to geotechnical design of retaining walls, eg AS4678 and AS5100, is based on Ultimate Limit State (ULS) design. The fundamentals of AS4678 are essentially the same as Austroad 1992 which was one of the first set of design guidelines in the world. AS4678/Austroad has been recognised by the geotechnical community that the geotechnical design would result in an overestimate of the structural actions on the retaining structures due to the specified material factors, load factors and load combinations, in particular the bending moment, the width of footing or embedment depth of a wall, and some time unrealistic predicted behaviour. This has been discussed by the Author in 2000 and others (eg Day et al, 2013?).

AS5100.3 adopts an approach to the design of soil-supporting structure such as a retaining wall based on where the primary load is coming from. If the load is primarily from a structure such as impact load from barrier, the retaining structures will be designed using a partial load factor method. On the other hand when the loads are primarily from soil (including water) then the retaining structure will be analysed by using ALL load factors equal to 1.0. To derive the ULS structural actions on the structure the loads are multiplied by a load factor of 1.5. The geotechnical design is based on load factors of 1.0. The calculated ULS geotechnical actions will be checked against the design geotechnical strengths with appropriate geotechnical strength reduction factors. A flow chart, which demonstrates how a soil-supporting structure should be analysed as per AS5100.3 commentary, is presented in Figure 1.

The approach in AS5100. 3 has been widely accepted or adopted by governmental authorities for design and construct projects' scope of works and technical criteria (SWTC) to overcome the conservatism of AS4678. However it should be pointed out that there is no clear demarcation as to what percentage of the load proportion is defined as "primary".

Therefore an engineering judgment will have to be made in assessment of the loading proportions for a concerned retaining structure.

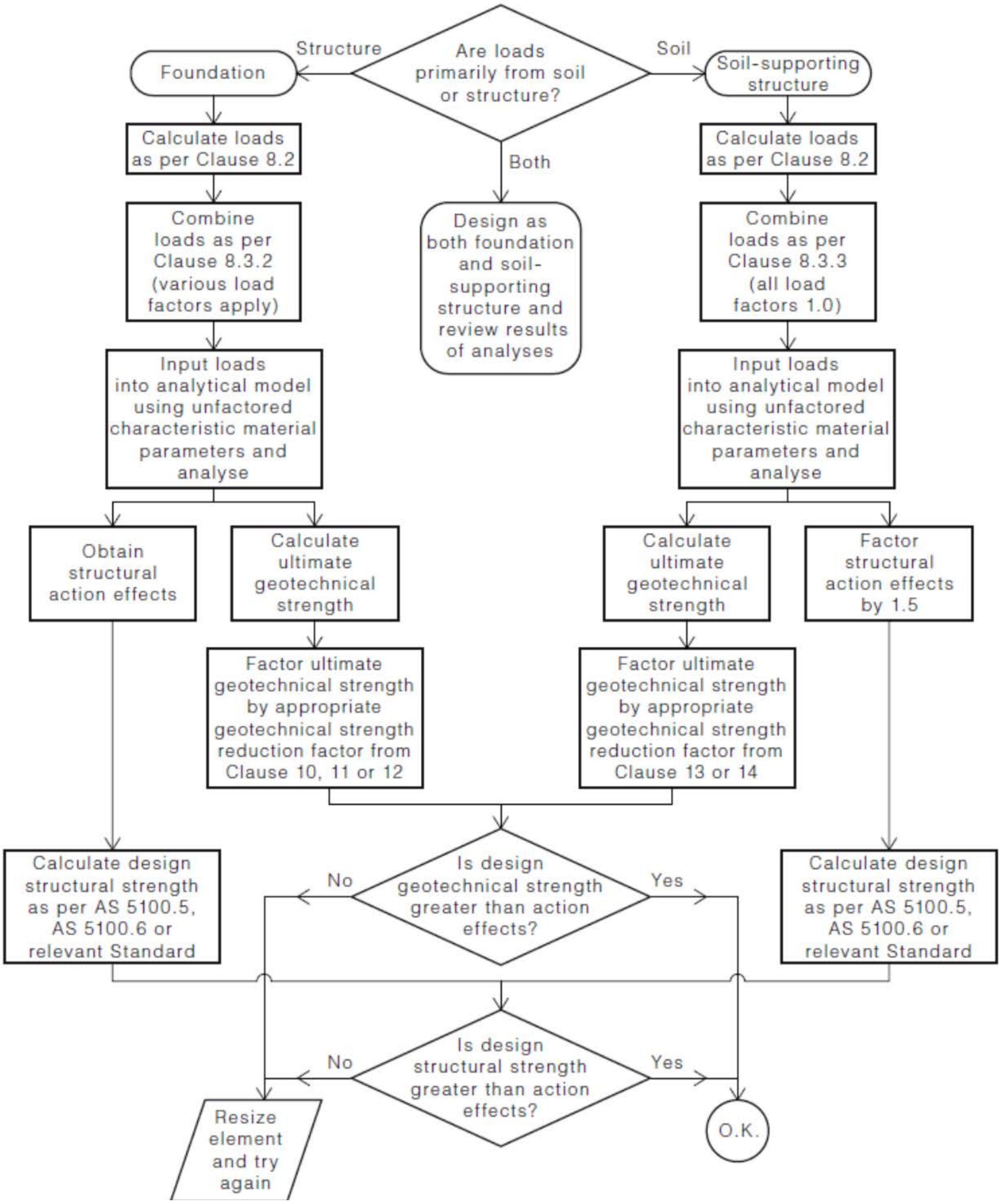

Figure 1 Design process for strength and stability (after AS5100.3 Supplementary 2008)

2.2 DESIGN LOADS AND FACTORS FOR EARTH RETAINING STRUCTURES

Loadings must be considered as per Clause 4.1 of AS1170.1 for an earth retaining structure. Three types of loads required for retaining structure design are described as follows:

- The dead loads should include: a) self weight of the structure; b) backfill weight on the top of structure; c) imposed earth pressure acting behind the structure; imposed water pressure behind the structure; and static sill beam loadings (both vertical and horizontal) on the earth retaining structures.
- The live loads should include: a) traffic surcharge; b) vertical live load acting on the sill beam; c) transient loads on sill beam, which might include vehicle braking, shrinkage, creep and thermal effects; and d) traffic/barrier impact loading.

- The determination of earthquake design loads should require: a) assessment of the seismicity of the site and the potential for liquefaction; b) determination of response characteristics of the structure and c) AS1170.4 for determination of earthquake induced loads.

Water pressure is often the most important load in the design of a retaining structure for a deep excavation and the following aspects associated with water should be considered: a) the level and fluctuation of permanent water table b) the inflow rates into excavations; c) effects of dewatering on the water table and on adjacent structures; d) the presence of and pressures associated with artesian and sub-artesian conditions; and e) the potential aggressiveness of the groundwater to buried concrete, steel and the like.

The load factors stipulated by Clause 5.4 of AS5100.2 shall be as follows: Soil loads and properties of the soil shall be obtained from AS4678. The design of foundations and soil-supporting structures shall be carried out in accordance with AS5100.3. Where structure induced loads are primary dominant, the density of soil and water shall be factored by the load factors in Table 1. The load factor for traffic has also been included in Table 1 for the example presented in Section 5.1 of this paper.

Table 1: Load factors for soils and water

Type of soil and water	ULS where soil		SLS
	Increases load	Reduces load	
Controlled fill with regular testing of soil density	1.25	0.85	1.0
All other fills and in-situ soils	1.5	0.7	1.2
Groundwater	1.0	1.0	1.0
Traffic load (excluding barrier impact load)	1.8	1.8	1.0

It is important to note from AS5100.2 that variations in water levels shall be taken into account by using design levels based on a return period of 1000 years for ULS or 100 years for SLS cases.

2.3 IMPORTANCE OF WATER PRESSURE FOR AN EMBEDDED WALL DESIGN

For an excavation through loose dense sand, having a friction angle of 30 degrees and a bulk unit weight of 18 kN/m^3, down to a depth of 6 m below the existing ground surface. Case A is for the water table at the base of excavation on both side of retaining wall and Case B is for groundwater table at the ground surface behind the wall, as shown in Figure 1. The calculated total horizontal load above the excavation level of Case B is 2.2 times that of Case A. The water load is 3 times of the earth load for Case B. The primarily reason are:

- The active soil pressure coefficient is 1/3 while the coefficient of water pressure is unity which is 3 times of that of earth pressure;
- The bulk unit weight is about twice of the submerged/effective unit weight of a soil.

As such it is critical to appreciate the significant impact of groundwater pressure on the design of an embedded retaining wall.

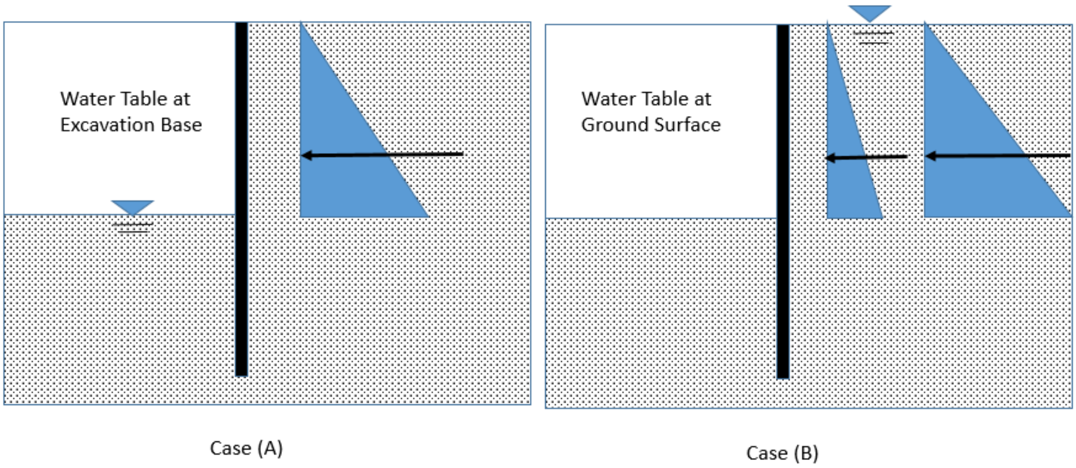

Figure 1: Diagram of lateral load on an embedded retaining wall

3 SOME DESIGN ISSUES IN DETERMINING GROUNDWATER PRESSURE

A number of current codes of practice and published papers/books have provided guidance on how to determine the water pressure acting on the retaining wall structures under either temporary or long term conditions. A summary of commonly used codes or references that are used by many practicing engineers for assessing the groundwater pressure is presented in Table 2. It can be seen that some of the guidance is more generic as to what the groundwater pressure should be. It requires the engineer to interpret the code in detail during the design process. However the majority of the requirements are clear enough for the engineer to apply the requirements for the wall design. Most importantly it is necessary for the engineer to determine what the most appropriate groundwater table for normal condition and abnormal or accidental circumstances are.

It can be concluded from the guidelines related to the current code of practice that:

- The groundwater table will have to be determined for any retaining wall design for both temporary and long term conditions, including the permeability of soils and their flow characteristics.
- A regional groundwater condition will have to be considered, in particular potential changes of the groundwater regime due to insertion of the retaining structures such as damming effect.
- Breakage of existing water main should be considered in the retaining wall structure design during construction and in the long term.

This is a risk management exercise that requires engineers to make informed design decisions. The design is required to be carried out to take account of all the reasonable risks for the concerned wall system.

Table 2: Typical code requirements for groundwater pressure

Reference	Requirements for Water Pressure
AS5100 – Australian Bridge Design	As discussed in Section 2.2 of this paper. A water table of 1 in 100 year return period has to be considered for SLS case and 1 in 2000 return period for the ULS case. It may be inferred that water main breakage is essential for retaining structure design for ULS case as a minimum.
Section 2.1.3 - Ground water, BS8002 (1994)	*Possible changes in ground water levels due to the presence of the retaining wall and seasonal or other causes, including future trends and accidental circumstances, should be investigated. Future works, in the vicinity of the wall, may give rise to changes in the long-term groundwater conditions; where such future works can be reasonably anticipated, the potential changes in the ground conditions should be assessed.*
Eurocode 7, EN1997-1, (2004) Section 2.2 (page 22) ***Geotechnical Design – Part 1: General Rules***	*Variations in ground water levels including e.g. the effects of deviating possible flooding, failure of drainage system, water exploiting.*
GCO Publication No 1/90 (1990), Paragraph 4 (page 41) ***Review of Design Methods for Excavation***	*Deformation of the ground adjacent to excavations may cause breakage of water-carrying services. In some cases, the amount of water released can adversely affect the stability of excavations. Therefore, it is important that the location, size and condition of such services in the vicinity of a proposed excavation are ascertained. In situations where significant water flow can occur in the event of a breakage, it will be prudent to allow for the possible increase in water load in the design. Alternatively, consideration should be given to diverting water-carrying services prior to construction.*
CIRIA Report C580 (2003) – Section 5.5 ***Embedded Retaining Walls - guidance for economic design***	*Determination of Groundwater Pressures* *"…the designer should check that the following have also been considered:"* *"…the proximity of sources of free water and the likelihood of such sources becoming available over the design life of the wall."* *"… the effects on the local hydrology of the site due to the construction of the wall,*

	e.g., potential damming to natural groundwater flow patterns, long term rise in aquifer groundwater levels."

4 SOME ISSUES IN APPLICATIONS OF CODE REQUIREMENTS

4.1 DEEP EXCAVATION

It is often asked to provide advice on what should be the design water table and load factor for a deep excavation support design. Some of the codes of practice requires an application of a load factor of 1.1 by Austroads 1992 and 1.4 for permanent case and 1.05 for exceptional case by Singapore Land Transport Authority (2010) to the groundwater pressure such as uplift for underground permanent structures. This often leads to the problem for the case of underground stations where excavation depth could be of the order of 30m or more. The realistic water pressure for the base slab of the station is the maximum flood level above the ground surface level, rather than the water head times a load factor of 1.1. For example, using the load factor the groundwater head can be 3m above the ground surface when the station base if about 30m and the standing water table is at ground surface.

Although it was an issue for Austroad 1992 which required a load factor of 1.1 on water pressure the current AS5100 has resolved this load factor of 1.1 issue by flood modelling for a defined return period, and adopting a load factor of 1.0 for water pressure.

4.2 SEEPAGE CONDITION AND STABILITY

The seepage condition of a deep excavation should take account of soil characteristics with respect to hydraulic conductivity. When excavating through sands, the retaining wall should be designed such that there will be no piping due to hydraulic gradient at the base of the excavation and the inflow towards the excavation is manageable by sumps and pumps. In addition the potential settlement due to dewatering behind the retaining wall is within the acceptable limit in order to avoid any damages to the existing infrastructures or buildings.

When excavating through soft to firm clays, the retaining wall must be designed to ensure the overall stability of the wall system to avoid potential heave at the base of excavation. In addition although the permeability of clays are usually very low, the groundwater table drawdown should be controlled to avoid excessive settlement due to its sensitivity to water level changes.

For urban development it is often critical to make sure the collected groundwater at base of excavation will be collected and treated where necessary prior to discharging to the either natural or storm water system.

The current methods of analysis of the base heave and piping failures are working stress method. It can be readily analysed using design charts or the commercially-available finite element packages. The overall global stability is also based on the conventional limit state equilibrium method that the partial load factors cannot readily be applied in that it is impossible to forecast which part/component will be in favour or non-favour of stability. A strength reduction method has also been developed in software such as Plaxis. It should be realised that the strength reduction factor cannot be readily converted into the conventional FoS although it provides useful information on the potential mechanism and the magnitude of the mobilisation factor along the potential failure plane.

4.3 GROUND ANCHORS AND DESIGN LOADS

Two types of ground anchors are used: soil anchors and rock anchors. Ground anchor design requires to check three possible failure mechanism: 1) mechanical anchor (tendon or steel) capacity; 2) the interface bond capacity between grout and soil/rock; and 3) the cone pullout capacity.

Soil anchors are dependent upon the effective stress at the anchorage length, which could be reduced due to rise of the groundwater table as a result of water main breakage or during a storm event. The soil wedge pullout capacity could be influenced by the potential breakage of water main and interaction when the orientations are not appropriately arranged.

Rock anchors are often inserted in competent rock which are not sensitive to the change of water level changes as it is relied upon the bond at the interface of anchorage shaft and parent rock which is not dependent upon the effective stress. The rock anchor pullout wedge is often governed by the potential defects rather than the intact strength.

Many of the geotechnical reports provided to the designer follow the recommendations for excavations by Terzaghi and Peck (1967) as shown in Figure 2. It is often not clear as to how the potential rise in the groundwater table should be considered, in particular due to the breakage of water main near deep excavation. Furthermore the actual design by the contractor is often less than those recommended by the geotechnical engineer, either through soil structure interaction or similar project experience.

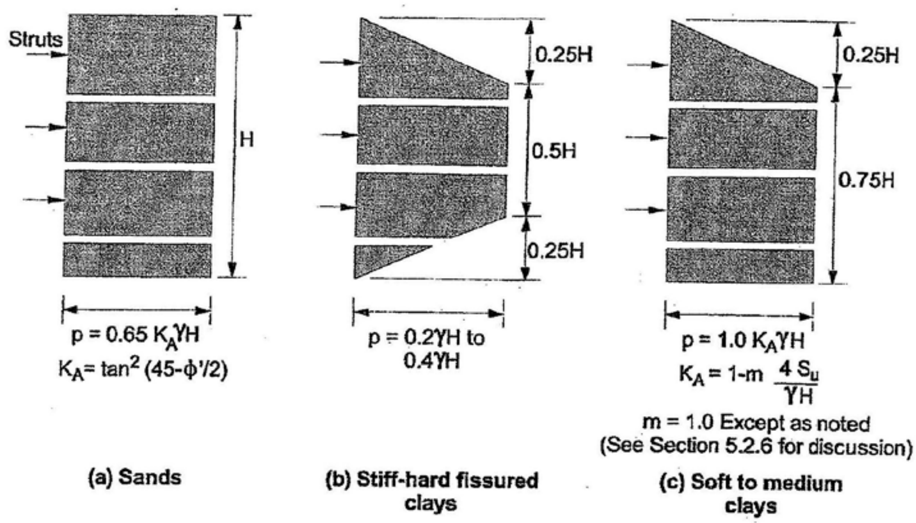

Figure 2: Typical earth pressure distribution for braced excavation (Terzaghi and Peck 1967)

5 CASE STUDIES

Two project cases are examined in the following sections. The first one is about an L-shaped retaining wall with medium performance level crash barrier sitting on a reinforced soil wall. This case is the without water pressure and discusses about how the AS5100 is to be applied and the results are compared against that from AS4678. The second case is for a retaining wall subject to water pressure resulting from a burst water main during construction. It led to significant litigation due to closure of a major road to airport in Sydney for a few months. Some of the limitations and issues identified in these project examples will be discussed in detail in the following sections.

5.1 RETAINING WALL GOVERNED BY CRASH LOAD

Figure 3 shows the typical cross-section of the L-shaped wall and the key dimensions of the L-shaped wall were determined from AS5100.3, which is founded on a reinforced soil wall. The L-shaped wall is primarily to withstand medium performance level crash load.

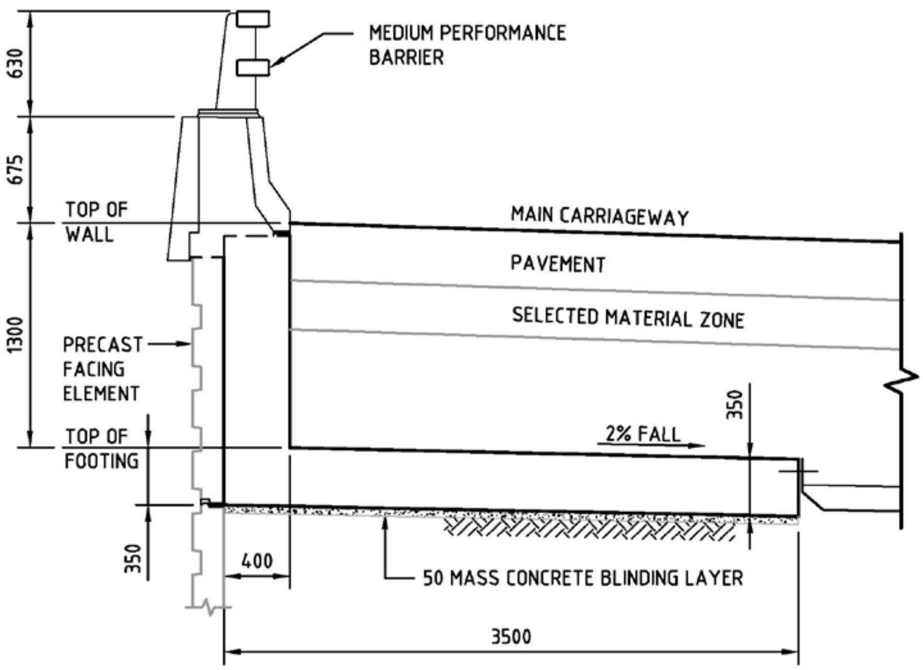

Figure 3: L-shaped wall support crashing barrier

A sensitivity study of the required base widths using AS4678 and AS5100 was carried out and a summary of the calculated base widths is presented in Table 3.

For this case the potential water level rise behind the retaining wall is secondary in that the probability for water table rise to the ground surface at the same time as collision occur is very low.

Some observations of the calculated base length based on AS5100.3 and AS4678 are as follows:

- Sliding is the most critical failure mechanism governing the minimum requirement of base width of the L-shaped.
- For the application of AS5100.3 it is important to assess if the soil load or the structure load is the primary load acting on the wall. As the height of soil and pavement above the L-shaped wall is about 1.3m and the soil pressure is about 9 kN/m. The lateral load induced by traffic load at the virtual back is approximately 11 kN/m. The collision load per metre run of wall is (500/30) = 16.7 kN/m. It can be seen that the governing load against sliding is around 50/50 impact load (structure induced) and soil load (earth induced including traffic). As such the L-shaped wall was checked for both cases with partial load factors and with ALL factors of 1.0. The design was found to be governed by the partial load factor case.
- When AS4678 was used for the partial load factors and material factors based on Load Case B shown in Table J1 of AS4678 made the wall base 0.4m longer. This is primarily due to AS4678 being silent on how to deal with the impact loading case and Load Case B did not allow for the traffic load on the top of L-shaped wall to be considered in the sliding stability check.
- It should be recognised that the characteristic value of friction angle of the fill defined in AS4678 should be the cautious estimate, which is close to but not greater than, of mean value. AS5100.3 states that the characteristic value of geotechnical parameter should be the conservatively assessed value of that parameters.
- AS5100.3 states that the geotechnical parameter should be assessed for the identified failure mechanism and highlights that a low characteristic value of geotechnical parameter may not provide a conservative design such as for dynamic or earthquake loading cases. Furthermore AS5100 encourage an appropriate testing and consideration of geotechnical parameters for design analysis. As such engineering judgement is often required to determine the characteristic values of geotechnical parameters.

Table 3: Comparison of L-shaped wall design between AS4678 and AS5100

Code	Barrier / Stem Height (m)	Stem/Base Thickness (m)	Friction Angle of Fill/ Foundation (Deg)	Geotechnical Strength Reduction Factor	Calculated Interaction Factor Against Sliding	Base Width (m)
AS4678	1.305/1.3	0.4/0.35	30/34	N/A	1.07	3.9
AS5100.3	1.305/1.3	0.4/0.35	30/34	0.55	1.07	3.5

The author is of the opinion that the geotechnical design parameters defined by AS4678 and AS5100 are virtually the same. The other important factors are that a) There is limited laboratory testing results for each soil unit; and b) The statistical analysis of deriving the characteristic values is often not meaningful but abused by some design engineers when the number of samples are too small. The key issue of conservative design outcome is due to that fact that the live traffic on the wall/pavement during a crashing event is not allowed to be considered for load combination Case B in AS4678.

5.2 DEEP EXCVATION SENSTIVE TO WATER PRESSURE

This case study is related to a retaining wall "failure" resulting from water pressure changes behind the wall caused by a burst water main on 6th March 2008. The water pipe running parallel to a major road in Sydney was some six metres away from an excavated site. It is understood the water flowed unabated for ten hours before Sydney Water were able to isolate and then turn off the supply to the broken pipe. The Road was immediately closed to traffic.

The excavation was flooded, and the Road suffered damage through loss of road base material rendering it unsafe. The retaining wall supporting the Road was found to be leaning into the site approximately 75 mm, but it did not collapse, nor did adjacent buildings. The wall designer declared on 9 March 2008 that the wall was unsafe.

It is a typical design and construct case, as the structural engineer developed a concept design for the shoring wall and a subcontractor was commissioned to a D&C contractor to build the wall. The ground anchors were installed by a specialist sub-contractor.

The actual name of the project and parties involved could not be mentioned in this paper as it was a litigation case. However the key factors impacting the "failure" will be discussed in the following sections:

5.2.1 Geotechnical Model and Design Parameters

A geotechnical investigation including boreholes and CPT profiling was carried out and a report was provided to the designers. The geotechnical profile adopted for the design was based on two boreholes available in the vicinity of the concerned retaining wall. As shown on Figure 4 the subsurface profile consisted four geological Units: Unit 1 – Fill and loose sand; Unit 2 - Medium to dense sand; Unit 3 – Stiff to hard clay; and Unit 4 – Bedrock. Note that the dense sand was not encountered at the borehole to the right hand side of the wall.

The groundwater table varied across the project site and a groundwater level at RL9.5 m or RL10 m was adopted for design for various sections. The existing road level varied between approximately RL15 m to RL13.5 m with the proposed excavation level down to about RL 7 m.

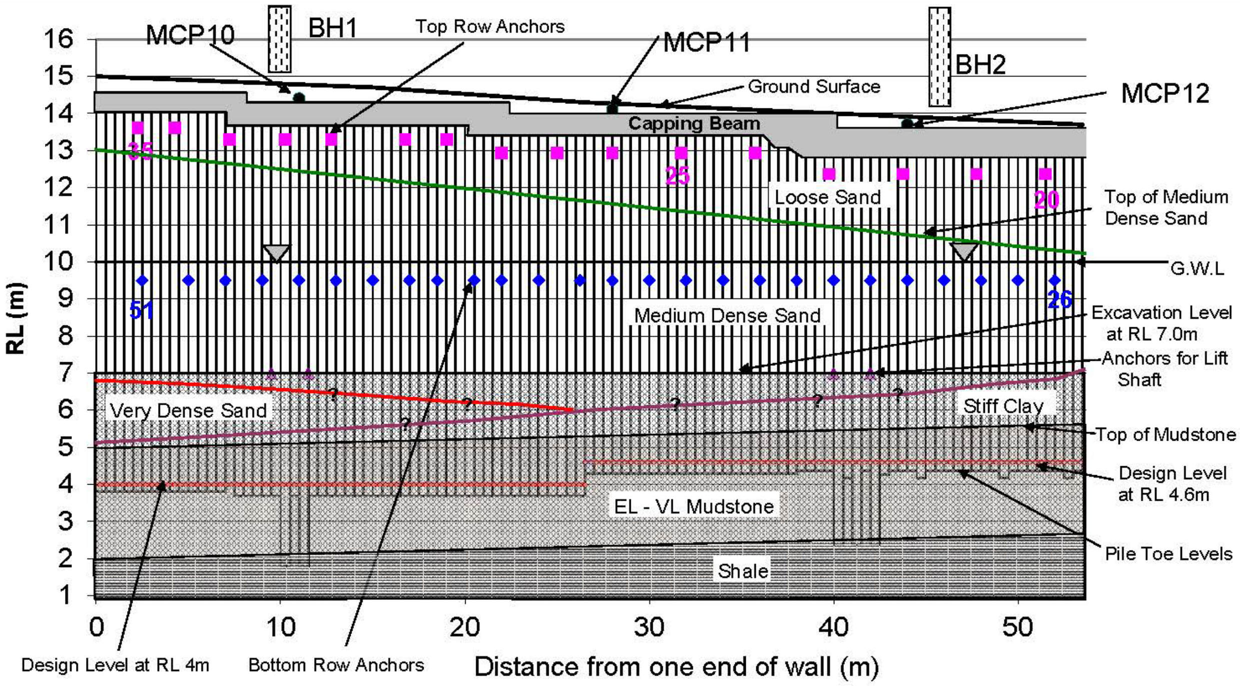

Figure 4: Elevation of as-built secant pile wall

The geotechnical units, material description and parameters adopted for the retaining wall design by the original designer are reproduced in the Table 4.

Table 4: Adopted geotechnical design parameters for each unit

Geological Unit/Description	Unit Weight (kN/m^3)	K0 [(1)]	Effective Cohesion (kPa)	Effective Friction Angle (Deg)	Ka/Kac [(2)]	Kp/Kpc[(2)]	Young's Modulus (MPa)
Unit 1 – Fill /Loose sand	18	0.5	-	30	0.23/0	4.37/0	12
Unit 2 – Medium Dense to Dense Sand	20	0.5	-	35	0.28/0	5.88/0	32
Unit 3 – Very stiff to Hard Clay	20	0.5	8	25	0.35/1.39	3.25/4.83	25
Unit 4 – Class V Mudstone/ Shale/Sandstone	22	0.5	8	30	0.29/1.23	4.37/5.84	50

Note: 1) K0 = At rest coefficient; 2) Active and passive earth pressure coefficients, Ka and Kp, were calculated based on a wall friction of 2/3 effective friction angle on active side and 1/3 on the passive side respectively.

5.2.2 Original Design Methodology and Outcome

The design standards quoted in the designer's report included:

- AS 2159 - Piling - Design and Installation
- AS 3600 - Concrete Structures
- BS 8002: 1994 - Code for Earth Retaining Structures
- CIRIA Report C580 - Embedded Retaining Walls - Guidance for Economic Design
- Eurocode 7 1995 - Geotechnical Design - Part 1: General Rules

The design analysis was carried out using program Wallap for calculation of the anchor loads and wall actions with a subgrade reaction model based upon Code of Practice –Retaining Structures, CP2 (1951) method. A minimum factor of safety of 1.5 was adopted to determine the required pile embedment depth and ground anchor loads. A construction staging including dewatering, excavation, installation of anchors and permanent basement slabs were modelled in the Wallap analysis.

The retaining wall along the Road, consisted of secant piles of 600 mm in diameter, with a spacing of 1000 mm between hard and hard piles (a theoretical overlap of 100 mm without any tolerance), was formed for a basement excavation for a residential development. The piles were designed to be at RL4 m for left portion and RL4.6 m for the right portion. There is a stepped capping beaming with two levels of temporary ground anchors, with the first row being about 0.5 m below the capping beam and the second row at approximately RL9.5 m which is below the measured groundwater table at RL9.5-10 m. The excavation was to be down to RL7 m along the wall with. It was determined that two levels of anchors were required. The top row of anchors were required to have a working load of 25 tonnes with a spacing of 3m for the left part and 5m for the right portion respectively while the bottom row of anchors are to be at 3m centres, with a working load of 30 tonnes and a continuous waler. The inclination angle of all anchors were to be at 30 degrees below the horizontal. The details of the ground anchors were left for the specialist contractor.

The reinforcement for the hard piles were based on the calculated bending moment from Wallap analysis for both temporary and permanent case when the basement has been constructed.

Settlement prediction was carried out and it was estimated to be less than 20mm for the Road. It was mandatory for RMS (formerly RTA) to accept the design with respect to the ground settlement prediction for pavement in that the excavation is immediately next to the Road. However it is clear the design was not reviewed and approved by Sydney Water even there is a high pressure old cast iron pipe of 450mm diameter present underneath the Road.

5.2.3 As-Built Wall System

The as-built records of piles and ground anchors were provided for review during the litigation process. The pile toe levels were generally slightly deeper than original design with minor misalignment of piles due to verticality imperfection during installation.

The top row of temporary anchors is numbered 20 to 35, as shown on Figure 4. Records provided show that the “design working load” for these anchors was 200kN, with each anchor having 2 No 15.2 mm diameter 7-wire strands. The free length of the anchor is noted as 5 metres with the bonded length as 7 metres. These anchors were installed in 100 mm diameter holes, with a PVC casing of 12 metres and an enlarged bulb at the end of the bond length. The spacing of installed anchors varies from 2 metres to 4 metres. The anchors were installed at 30 degrees below the horizontal at approximately 2 to 4 metres, as compared to original design of 5 metres. The lock-off load (pre-load) was taken to be equal to the working load, with test load at 1.3 times its working load. This gives an equivalent horizontal anchor working pre-load of 58 kN/m (for 3m centres).

The bottom row of temporary anchors on Botany Road is numbered 26 to 51 as shown on Figure 4. The anchors were installed at 10 degrees below the horizontal and each anchor comprised 3 No 12.7 mm dia 7-wire strands. The design working load for these anchors varied from 176 kN to 220 kN. The spacing of installed anchors is generally at 2 metres centres with only 2 anchors at 1.75 metres and 2.25 metres. The free length of the anchor is noted as 5 metres with the bonded length 7 metres. These anchors were installed in 100 mm diameter holes, with a PVC casing of 12 metres and an enlarged bulb at the end of the bond length. The lock-off load (pre-load) of anchors varied from 100 kN to 220 kN, with the majority being 176 kN, with test load generally at 1.25 times its working load. Since the spacing is variable the total anchor pre-load of 4507 kN spread over 54 metres gives an average anchor pre-load of 83.5 kN/m at 10 degrees below the horizontal. This relates to a horizontal design working pre-load of 82 kN/m.

During the anchor stressing, these anchors were taken to 125% or 130% of the working load. The design required the anchor working load should be held for 5 minutes in order to monitor stress loss or anchor elongation. The records do not indicate that jack extensions were monitored during this 5 minute hold period.

Table 5 presents a sensitivity analysis by assuming the potential rises of water table behind the retaining wall using the same model and parameters as the original designers. A subgrade reaction model was adopted in these sensitivity analyses. Three cases were analysed as follows:

- Case 1 – The same model and groundwater table as per original design presented by the Designer
- Case 1A – Same as Case 1 but with groundwater table raised to the ground surface due to water main breakage, with the maximum water pressure at the interface of sand and clay and the water pressure balance at the toe of pile.
- Case 1B – Same as Case 1A but with reduced anchor stiffness by trial and error in increasing the free length of ground anchors until the calculated lateral defection is equal to 75mm which was measured. This was to simulate the partial loss of anchor capacity or possible pullout due to loss of vertical effective stress resulting from water level rise after the burst water main.

It can be seen that the calculated wall lateral deflection of Case 1 is approximately 5 times for Case 1A and 7 times for Case 1B. The bending moment and anchor loads are approximately doubled or greater for both Cases 1A and 1B after the groundwater table rise to the road surface as a result of the burst water main.

Table 5: Calculated wall deflection, bending moment and anchor loads prior to and after burst water main

Case Description	Water Level (m)	Horiz. Pre-load for Top / Bottom Anchor (kN/m)	Lateral Deflection (mm)	Bending Moment of Pile (kNm/m)	Shear Force (kN/m)	Horiz. Top / Bottom Anchor Loads (kN/m)
Case 1- Original Design Model	RL 10.0	52.0/57.7	11	122	64	58/74
Case 1A – Higher Water Table	RL 14.1	52.0/57.7	52	248	115	99/156
Case 1B - Relaxed Anchor Stiffness	RL14.1	52.0/57.7	75	254	118	101/161

Although the original design anchor may have achieved some 50% bond length in rock, as shown in Figure (a), for this particular sectional model it is apparent that all the as-built anchors have been constructed as soil anchors, as shown in Figure 5 (b). It can be seen that the as-built anchors configuration will reduce the anchor capacities due to the interaction among of the “pullout cone/wedge”. In addition, when the water table rises, the shaft adhesion of the soil anchor, dependent upon the effective overburden stress and friction coefficient at the interface between grout and soils, will be reduced significantly. The wedge pullout capacity will also be reduced due to the reduction in effective weight resulting from the rise of water level. These two factors are the main reason why the capping beam has moved approximately 75mm after the water main burst.

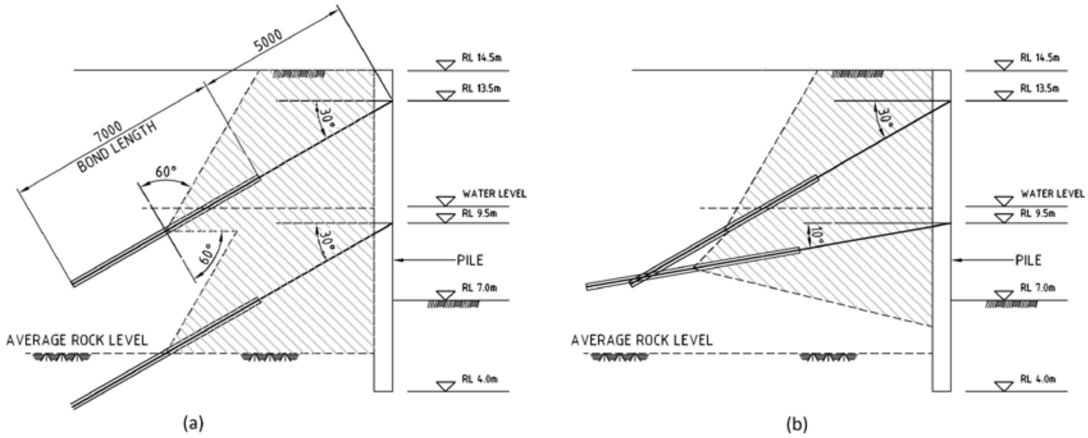

Figure 5: Sketch showing the interaction of pullout cone of ground anchors

5.2.4 Lessons Learnt from This Case:

Although a series of design code or references were presented in the design report the design was carried out using CP2 which is a working stress method. As such the design requirements with respect to the ultimate limit state design were not appropriately addressed. Some of the points that are worth noting for future considerations are summarised below:

1. None of the Australian Standards such as AS4678 and AS5100 was referred to for the retaining wall design.
2. Presence of loose sand to medium dense sand surrounding above the existing water table means that there is a very high risk associated with a significant water table rise due to breakage of water main behind the retaining wall.
3. In accordance the current code of practice both SLS and ULS conditions should have been considered for the design of the retaining wall system.
4. As discussed in Table 2, the design water table behind the retaining wall due to heavy rainfall events or the breakage of water main should have considered as an ULS case in accordance with the current code of practice to provide a robust design.
5. BS8002 required the design to consider the ultimate limit state water pressure yet lacking detail guidance on how to determine the ULS water table. However the over-excavation of the greater of 0.5m or 10% of the retaining wall height was not considered mandatory, which would provide an extra safety margin in the event of potential burst water main.

6. The as-built ground anchors were ALL soil anchors with the top row installed at 30 degrees below the horizontal, at 2 to 4 metre spacing along the wall; and the bottom row installed at 10 degrees below the horizontal at approximately 1.75 to 2.25m centres. This resulted in overlapping of the anchorage part of the soil anchors, leading to reduction in the wedge pullout capacity of each soil anchor, especially when the water table rise to the existing ground surface.
7. The interface between structural engineer and geotechnical engineer is not clear as to if the base slab will be designed as "tanked" or drained at the time of wall "failure".

After checking the reinforcement within the hard piles its capacity was of the order of 260 kNm which is not necessary overstressing the pile itself based on the Wallap sensitivity analysis. However the loss of effective stresses in the anchorage of the soil anchors is quite significant resulting in loss in the pullout (both shaft and wedge) capacity. In addition the overlapping of installed anchors due to different inclination angles of top and bottom anchors resulted in the reduction in their capacity. To this end, arguably the original designer would be more confident when asked if the wall was safety after burst water main got controlled rather than declared that the wall was unsafe should the ULS case be assessed at the design stage for the higher water level behind the wall.

6 RECOMMENDED DESIGN APPOACH FOR AN EMBEDDED WALL

A recommended approach for an embedded retaining wall design with dominant earth and water pressure may be described as follows:

1. To characterise the site conditions, including the geological profile, model, design parameters and groundwater condition.
2. To investigate all the adjacent underground utilities information, including the presence of water main.
3. To undertake the design based on both the serviceability limit state (SLS) and ultimate limit state (ULS) design for the retaining wall, ground anchor or struts using an appropriate code such as AS5100.3.
4. To assess the regional ground water condition and adopt a reasonable design water table, including the potential damming effect after installation of the retaining wall system.
5. To follow the guidelines set out in AS5100.3 for soil load dominating case for both SLS and ULS design checks.
6. To carry out an ULS analysis of the retaining wall system after the breakage of water main if it is near the excavation, in particular for a wall through sandy soils and the assessed design water table is not near or at the ground surface.

It is expected that if the above design approach is to be followed then designer will be more confident when asked for a decision on if the wall is safe after a burst water main. This is more important with aging of the buried water mains in the large and heavily populated cities. From an overall cost savings point of view the extra investment may not be a huge additional component as compared to the whole project, taking account of the potential risks of delay, program and the associated claims.

7 SUMMARY AND CONCLUSIONS

This paper reviewed the design philosophy of ultimate limit state design and the design process based on the current Australian Standards such as AS5100 and As4678. Importance of water pressure in the design of retaining structures have been highlighted through typical examples, with some of the design issues and limitations in applying the code of practice by engineers further discussed. The two case studies has demonstrated how a retaining structure should be appropriately analysed and designed in accordance with the available codes whether there is water pressure behind the wall or not.

The crash barrier design example indicates that AS5100 should be adopted in design a retaining wall rather than AS4678 in that it will lead to a conservative design event there is no presence of water pressure.

It has been emphasized that the consideration of water pressure due to a burst water main should be considered in the design of a retaining structure for both temporary and permanent conditions as an ultimate limit state case. This is more important when the wall excavation is through sandy soils, which is highly reactive to change of water table due to its high permeability. The ULS analysis and design should consider the retaining wall system, including the wall, the anchor or strut, deformation characteristics and its impact on the existing infrastructure such as underground utilities and water pipes.

An appropriate method of designing a retaining wall should review the source of loading, for example AS5100.3, so that the most economic design can be achieved. It has been found that AS4678 will generally give the designer a more conservative design.

8 DISCLAIMNER

Neither the author/s, contributors, nor their respective organisations make any representation or warranty as to the accuracy, completeness or suitability or otherwise of the information contained in this paper and shall have no liability to any person in connection therewith.

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