SOME NOTES ON THE DESIGN OF PILE FOUNDATIONS IN SEDIMENTARY ROCK

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ABSTRACT

This paper presents some technical notes on the design of pile foundations in sedimentary rock based on the author's thirty-year research and consulting experiences. Firstly, the fundamental difference between the superseded working stress method and the current ultimate limit state (ULS) approach within the practicing codes will be discussed from a geotechnical engineer's perspective. Then some delusion and confusion encountered by practicing geotechnical engineers such as rock classification and the characteristics of each rock class will be highlighted. The importance of soil and structure interaction and establishment of design criteria for structures and substructures will be emphasized. An overview of the published methods for assessing the pile end bearing capacity and lateral resistance will be carried out to appreciate some issues that practicing engineers are often required to deal with. For typical bridge pile foundations and piled deep excavation retention or retaining structures, both serviceability limit state and ultimate limit state assessments are required to satisfy the requirements set out in current codes of practice. For vertically loaded piles in sedimentary rock it is found that the serviceability limit state is governing the design rather than the ultimate limit state condition for most road and railway projects, based on the commonly accepted design parameters. For laterally loaded piles in rock it is noted that the method based on the lateral force and bending moment equilibrium such as described in Hong Kong Geoguide 1 is frequently used to determine the pile socket length. The critical input parameter required by this method is the ultimate lateral resistance of the rock mass, which is often arbitrary with little guidance provided, and a degree of confusion is often noted by the author. It is proposed to undertake a lateral equilibrium assessment under ULS conditions for a piled wall along with analysis of the deformation characteristic of the rock mass to come up with the "mobilised" rather than the ULS lateral pressure for pile socket design. Worked examples will be given to demonstrate how the pile socket in sedimentary rock can be determined with reasonable confidence for a cantilever piled wall for tunnel projects, and for a bridge structure.

1 INTRODUCTION

Since the introduction of ultimate limit state (ULS) concept, which is driven by structural engineers, the geotechnical design approach has been evolved accordingly. There are two streams of the ULS design approaches: One is using partial load factors on both structural actions and material resistances such as Eurocode (2004) and the other by means of a "lumped" geotechnical strength reduction factor on the ultimate geotechnical resistance such as Australian Piling Code AS2159 (2009) and Australian Bridge Code AS5100.3 (2004). The Australian code approach is similar to the methods outlined in the AASHTO LRFD Bridge Design Specifications by the American Association of State Highway and Transportation Officials (2010). The current ULS codes of practice such as AS2159-2009 and AS5100-2004 require serviceability limit state (SLS) checks to ensure the total and differential movement criteria are met, while the ULS strength requirements are satisfied.

This paper will examine the fundamentals for the pile design under the working stress method (WSM) and ULS design approach, with an emphasis on the structural design criteria for key structural elements. Some issues experienced by the author in the reviewing of geotechnical interpretation and design reports are highlighted. Worked examples will be given to demonstrate the importance of the serviceability design for piles in good quality sedimentary rock. A proposed method for pile design in good quality sedimentary rock is also presented.

2 WORKING STRESS METHOD VS ULTIMATE LIMIT STATE DESIGN

The Australian piling code AS2159 (1978) was based on the working stress method. The permissible shaft resistance and the end bearing pressure of a pile in homogeneous bedrock are calculated to be 5% of the UCS and 30% of UCS respectively, as outlined in A1.1.3 of Appendix A of AS2159-1978. These recommended shaft resistance and end bearing capacity are generally in line with the recommended values in British Foundations Code BS8004 (1986) and Hong Kong Geoguide 1 – Retaining Wall Structure (1982, 1993). An extract of permissible stress for concrete piles based on codes of practice or guidelines around the world is presented in Appendix A. Usually a minimum factor of safety of 1.67 must be considered on the ultimate geotechnical capacity of a single pile when a static pile load test has been undertaken for specific site conditions.

The ultimate limit state design methodology of pile design, based on Australian codes such as AS2159 (2009) and AS5100.3 (2004), requires load factors to derive the structural actions on a pile or pile group and a “lumped” geotechnical strength reduction factor on the ultimate geotechnical resistance to derive the ultimate design geotechnical strength.

The ultimate geotechnical strength in AS2159 (2009) is defined as “The resistance developed by an axially or laterally loaded pile or pile group at which static equilibrium is lost or at which the supporting ground fails.”. The so-called ultimate limit state would probably never be realised during the service life of any structures that are properly designed, especially for superstructures supported by piles in rock, for example, total loss of static equilibrium or ground support.

The serviceability limit state in AS2159 (2009) is defined as “A limit state beyond which specified service criteria are no longer met, such as unacceptably large displacements, vibrations, cracking, spalling and other local damage.” This is often the governing case for superstructures supported by piles in rock due to the large deformation required to mobilise the ULS rock strength. The fundamental philosophy of SLS design is to ensure the structural elements are in an elastic state during the sustained loading conditions throughout the design life of the structure.

The load factors that should be considered for calculating the ultimate structural actions on a single pile or pile group are based on AS7110 (2002). For most superstructures, it is found the “lumped” load factor on the vertical structural action is typically 1.3, with the range being between 1.2 and 1.5. The lowest geotechnical strength reduction factor in AS2159 (2009) is 0.4, which will not require any testing for piles founded in rock or soils. This implies that an equivalent factor of safety (FoS) for a single pile or pile group is ranging between 2.8 to 3.75. Where a comprehensive geotechnical assessment including borehole drilling and pile static load testing is conducted, then the geotechnical strength reduction factor can be increased up to 0.9. This means that an equivalent factor of safety will be ranging between 1.33 to 1.67. It is noted that the equivalent FoS values are much lower than the recommend value of 2 when pile static load test is carried out as stated in AS2159 (1978).

3 VERTICALLY LOADED PILES IN SENDMENTARY ROCK

3.1 END BEARING PRESSURES

For vertically loaded pile design, the practicing engineers are frequently referred to the published papers by Pells et al (1978) and (1998) for the permissible (allowable) and ULS end bearing pressures. The three principal factors that Pells et al considered in assessing the permissible (allowable) end bearing pressure of a pile founded in rock are the unconfined compressive strength (UCS), the allowable clay seams and defect spacing. The vertical permissible (SLS) and ULS end bearing pressures and Young’s modulus for various rock classes based on Pells et al (1998) are reproduced in Table 1 and Table 2.

Table 1: Summary of geotechnical parameters for sandstone

Sandstone Class	UCS Range (MPa)	SLS End Bearing Pressure (MPa)	Ultimate End Bearing Pressure (MPa)	Young’s Modulus (MPa)
I	>24	12	>120	>2000
II	12-24	0.5qu max 12	60-120	900-2000
III	7-12	0.5qu max 6	20-60*	350-1200
IV	2-7	0.5qu max 3.5	4-15	100-700
V	>1	1	>3	50-100

* The value was modified from 40 MPa to 60 MPa by the Author for consistency with Table 2.

Table 2: Summary of geotechnical parameters for shale

Shale Class	UCS Range (MPa)	SLS End Bearing Pressure (MPa)	Ultimate End Bearing Pressure (MPa)	Young’s Modulus (MPa)
II	7-16	0.5qu max 6	30-120	700-2000
III	2-7	0.5qu max 3.5	6-30	200-1200
IV	>1	1.0	>3	100-500
V	>1	0.7	>3	50-300

From herein the discussions in this paper will be focused on the sandstone rock. Some observations can be made from Table 1 as below:

- The SLS and ULS end bearing pressures are for vertical loading conditions only.
- The SLS end bearing pressure is recommended to be 0.5 times UCS, with an expected settlement of approximately 1% of the pile diameter or less.
- The ULS end bearing capacity of Class III sandstone or shale is approximately 10 times of the corresponding SLS end bearing pressure. The expected settlement of the pile must be 5% of pile diameter or greater to achieve the ultimate end bearing capacity.
- The Young's modulus value for each rock class varies between 50 to 100 times UCS values for sandstone and 100-200 times UCS for shale.

For many design cases reviewed by the author the requirement of the minimum representative UCS value or the point load value is often not shown for each class of rock. In addition, the allowable percentage of clay seams and minimum defect spacing are also ignored. Some design drawings are issued showing each class of rock with the upper bound ULS end bearing pressure being presented. It is important to understand that there is an applicable UCS range for each class of sandstone or shale, with each class of rock being assessed by a combination of the UCS value, allowable clay seam percentage and defect spacing. For example, the UCS of intact sandstone bedrock may be greater than 12 MPa, but due to a higher percentage of clay seams within the influence zone of the pile toe the rock mass is assessed to be Class III.

There are often two schools of thoughts as to if the SLS end bearing pressure of rock should be presented in the design or not: One is continuing to provide the SLS end bearing pressure together with the ULS end bearing pressure for rock; the other is only providing the ULS end bearing pressure. It should be noted that the provision of the SLS end bearing pressure is not required by the current Australian piling code AS2159 (2009) and bridge code AS5100.3 (2004), except for deformation checks under SLS loads. It is the author's opinion that the SLS end bearing pressure is critical for pile foundations in Class III or better sedimentary rock in that it will be demonstrated in the examples that the design of piles in rock is often governed by the SLS load case. There are a broad range of recommended SLS end bearing pressures but the correlation factor with the UCS value is typically ranging between 0.2 to 0.4 (e.g. Peck et al 1974, AS2159-1978, BS8004-1986, Tomlinson-1994 & 2004,ASHTO-2010, Pells et al-1998). Furthermore, the recommended ULS end bearing pressures vary from project to project and there was no established correlation between the UCS value and the ULS end bearing pressure. For example, the ultimate end bearing pressure to be 10 times UCS value recommended by Pells et al (1998) is different from equation of 4.8 times square root of UCS by Zhang and Einstein (1998) which is based on a best-fit of a set of data.

3.2 PILE SETTLEMENT

The settlement of an end bearing pile without any socket, as shown in Figure 1, can be assessed by the following formulas:

$$s = q_{/}E\, d\, (1 - \nu^2) \qquad \textbf{(1)}$$

where s is the settlement, q is the end bearing pressure at the tip of pile, E is the Young's modulus of rock mass within the influence zone, d is the pile diameter, ν is the Poisson's ratio of rock mass.

When q is equal to 0.5UCS, E is equal to (50-100)UCS; and a Poisson's ratio of 0.2 is considered, the settlement as shown in equation (1) can be approximated as follows:

$$s = (0.005\text{-}0.01)d = (0.5\%\text{-}\ 1\%)d \qquad \textbf{(2)}$$

For typical pile diameters, the estimated settlements are summarised in Table 3.

Table 3: Estimated settlements of piles with different diameters under the SLS end bearing pressure

Pile Diameter (mm)	600	750	900	1050	1200	1500	2000	3000
Estimated pile toe settlement (mm)	3-6	3.7-7.5	4.5-9	5.2-10.5	6-12	7.5-15	10-20	15-30

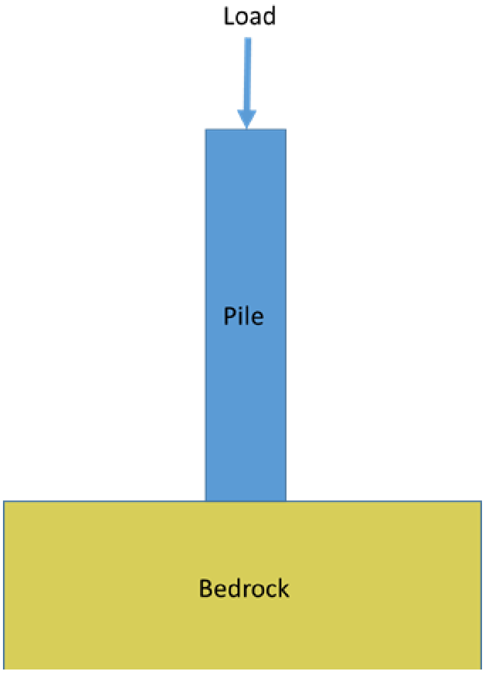

Figure 1: End Bearing Pile on Rock Only

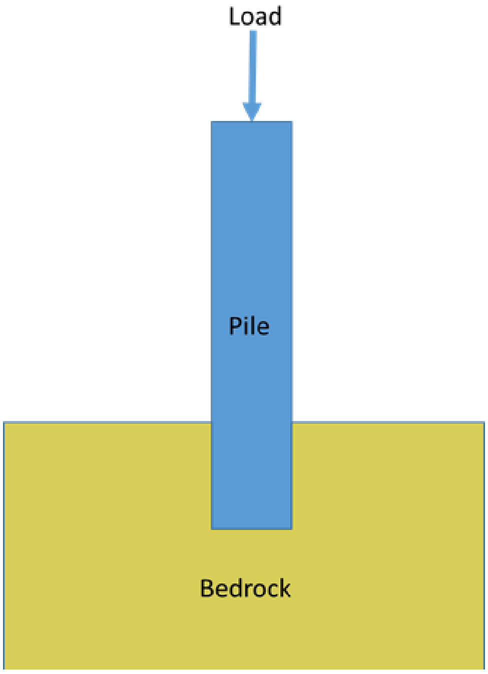

Figure 2: Shaft and End Bearing Pile in Rock

It can be observed from Table 3 that the calculated vertical settlement for piles under the recommended SLS end bearing pressure of 0.5UCS is generally less than 10mm for piles of up to 1 m diameter. Where a larger pile diameter is considered then the pile settlement will be proportionally greater. As such the debate about what is the appropriate ULS end bearing value is not critical in the design of pile foundations in Class III or better rock.

Where a pile is designed to take account of both shaft resistance and end bearing capacity the SLS end bearing pressure, as shown on Figure 2, may not reach the expected end bearing pressures shown on Tables 1 and 2 due to contribution from shaft adhesion. The corresponding settlement will be lower than those shown in Table 3 under the same vertical structural load. The settlement calculation of a single pile under vertical loading, considering both shaft and end bearing pressure, can be referred to from William and Pells (1981) or Rowe and Armitage 91984, 1987).

3.3 CONSTRUCTION VALIDATION

The requirements for construction validation of each class of rock on site are often not clearly defined in either the design report or on the drawings. It is frequently relied upon the geotechnical engineer’s experience and judgement during construction. Where there are boreholes at or close to the proposed pile locations then the geotechnical engineer will be able to use the available information to assess the rock strength and possibly the clay seam content and defect spacing. However, it is nearly impossible to check the defect spacing during pile shaft excavation. In addition, the rock strength, i.e. UCS value, can, at most, be only an estimate of rock strength based on the recovered rock chips from the drilling bucket. It is also important to note that the base of the pile shaft is impossible to be totally cleaned of all debris as compared to the pad footing considered by Pells et al (1978). To this end the author believes the geotechnical validation of the design end bearing pressure should be a best estimate rather than a scientific calculation.

3.4 STRUCTURAL CRITERIA

The issue for the SLS check is that the structural engineer needs to provide the design criteria for piles in relation to the total and differential settlements for the superstructure. Most bridge structures are designed to a differential settlement limit of 10mm, which is often defined as between two structural elements such as two piers for bridges, with no specific total settlement described when piles are founded on bedrock.

The differential settlement could occur within a pile group or between pile groups, which is largely dependent upon the pile spacing within a group or the distance between the two groups. The level of differential settlement for two bridge piers can be readily satisfied based on the SLS settlements presented in Table 3 when the piles are founded on Class III sandstone or better quality bedrock.

The other important factor is to assess the SLS bearing pressure based on the deformation limit for the concrete pile. The American Highway Standard provides a summary of the permissible stresses in a pile, as shown in Appendix A. AS2159

(2009) considered ULS strength check of either reinforced or unreinforced concrete but it is referred to AS3600 (2009) for the strength reduction factor and for the SLS deformation checks. The fundamental requirement for the concrete piles under SLS loading is to maintain the stress level within the elastic condition to avoid excessive concrete creep under the SLS loading during design life. The ratio of permissible stress to the unconfined compression stress of concrete pile is typically ranging between 0.25 and 0.45 for various codes of practice. In Australia, a ratio of 0.3 has been considered acceptable as shown in Appendix A. For a pile concrete having a UCS value of 40 MPa, a permissible stress in the pile will be 12 MPa. This is comparable with the maximum permissible end bearing pressure of 12 MPa for Class I sandstone recommended by Pell et al (1998).

3.5 WORKED EXAMPLE

A single pile with 40 MPa compressive strength is required to carry a total axial SLS load of 120 MN and a ULS axial load of 165 MN. The pile is to be founded on Class I sandstone without any shaft socket. If we consider a ULS end bearing pressure of 120 MPa and a geotechnical strength reduction factor of 0.6 then the pile diameter required is 1.60 m. Should a pile of 1.6 m diameter be adopted then the end bearing pressure under the SLS load is approximately 60 MPa. This stress level is already greater than the 40 MPa concrete, leading to excessive movement of pile itself and the settlement of the rock mass. Hence this is not an acceptable design. To this end geotechnical and structural engineer interaction is required to ensure the final design is acceptable.

When an SLS end bearing pressure of 12 MPa is considered for the SLS load of 120 MN then the required pile diameter is 3.57 m. The stress level within the pile is about 30% of the unconfined compressive strength of concrete, which is within the elastic limit. As such there will be little plastic long term settlement due to pile concrete movement in the long term. It can be seen the diameter based on the SLS load condition is 2.22 times that of the diameter derived from the ULS load condition while the required base area under the SLS condition is about 5 times that under the ULS design case. As such a design considering both shaft and end bearing pressure a pile shaft with high compressive stress concrete may be considered to reduce the pile diameter.

The settlement of the pile will be of the order of 0.5% to 1% of pile diameter when the SLS end bearing pressure is considered. This value also implicitly considers the potential risks of some debris left at the base of the pile that could not be practically cleaned out prior to pour of concrete.

It could be deduced from this example that great caution should be exercised to determine the pile size when the ULS load only is considered as it could lead to unsafe design.

4 LATERALLY LOADED PILES IN SEDIMENTARY ROCK

4.1 LATERAL BEARING PRESSURE

The stability of a pile embedded in rock under lateral loads is often checked for both strength and deformation. For the strength and stability of a pile under lateral loads, the ultimate limit equilibrium method presented in Hong Kong Geoguide 1- Retaining Structures (1982, 1993), requires two failure mechanism checks: One is the pile lateral stability by assuming the rock pressure in a rectangular distribution, which requires the ultimate lateral bearing pressure of the rock mass. The other is the stability of a potential rock wedge on the passive side of the pile. The ULS lateral bearing pressures are taken by some engineers to be up to 50% of the ULS end bearing pressures under vertical load as presented in Table 1. The use of these high ULS lateral bearing pressure values is likely to result in two problems:

1. To underestimate the pile socket length to a level that is too short to achieve the practical solution as will be demonstrated in the later section.
2. Large lateral displacement that will be required to mobilize the ULS lateral bearing capacity, for example, when the piles are used for cantilever or propped structures.

4.2 LATERAL DISPLACEMENT

As can be estimated the ultimate lateral bearing pressure will require a lateral movement of about 50% of that required for a pile under same vertical bearing pressure based Pells et al (1998). This means it will require a lateral movement of 22.5 mm for a 900 mm diameter pile to mobilise the ultimate lateral bearing pressure within a rock socket. It can then be extrapolated that the lateral movement of a cantilever piled wall of 6m height above the socket will yield 62.5 mm deflection at the top of the wall if an allowable differential of 1/150 of wall height is considered. This level of movement is considered unacceptable. As such the author, would like to use the terminology of “mobilised” lateral bearing pressure for the piles under lateral loads.

The testing results in Hong Kong indicated the use of a lateral pressure of 2000 kPa for determining pile socket in slightly weathered granite was considered conservative. Rather, this lateral bearing pressure was used to achieve an adequate pile socket to ensure the lateral movement of cantilever structure was not excessive (personal communication Garth Powell, 1994). Greenway (1986) proposed to use the allowable rather than ultimate bearing pressure in the pile socket calculation to limit the lateral deflection.

The displacement required to destabilise the potential wedge failure in front of the pile socket is difficult to assess as it is often along the weak bedding plane or joints. As such stability of the potential wedge will be the key to ensure adequate passive resistance will be provided for the design socket. Once this stability condition is ensured then the lateral movement can be calculated in the same manner as normal rock mass.

4.3 STRUCTURAL CRITERIA

The allowable lateral displacement of a single pile is often not specified, though the Australian Piling code AS2159 (1978) allows a 10 mm limit for lateral pile testing. Often a geotechnical engineer would be informed of what type of structure will be constructed during the preparation of the geotechnical interpretative report or geotechnical design report. The author suggests the structural damage criteria for differential movement presented by Poulos et al (2004) be considered in preparation of the geotechnical interpretative report or geotechnical design report if there is no structural performance criterion provided. For a piled cantilever wall a permissible lateral deflection of 1 in 150 to 1 in 250 is considered reasonable. For tall building structures, a vertical deflection criterion of 1 in 1000 to 1 in 2000 may be considered.

4.4 WORKED EXAMPLE

For example, for a 6m high cantilever wall, the lateral deflection at the top of wall is ranging between 24 mm to 40 mm by allowing a differential lateral deflection of 1/250 to 1/150. If we allow a lateral movement at the top of pile socket of 5 mm then the total lateral deflection at the top of wall will be 29 mm to 45 mm. This, in turn, can be used to assess what is the *'mobilised'* lateral bearing pressure for pile socket assessment. To demonstrate this a series of finite element analyses (FEM) have been undertaken for a typical piled retaining wall, as shown in Figure 3. In the FEM analysis, it was assumed there are 6 m soil having a stiffness of 20 MPa, friction angle of 30 degrees plus a surcharge of 20 kPa at the ground surface. The bedrock was assumed to be class II sandstone with a Young's modulus of 2000 MPa and shear strength as per Bertuzzi and Pells (2002). It has found that a pile socket embedment of 1.3 times pile diameter is required to control the lateral deflection to within the limit of 1/150 to 1/250 of the wall height as described in Section 4.3. It is interesting to note that the lateral bearing pressure distribution against the pile within the rock socket is non-uniform and may be generalised as a hyperbolic shaped curve with a maximum being about 2000 kPa. This is comparable to Greenway (1986), indicating that an allowable of 2000 kPa over the upper part of the pile based on Figure 51 of Hong Kong Geoguide 1 (1993), as show in Appendix B, is conservative. It may be concluded that the nominated ULS lateral bearing pressure, as shown in Figure 51, Appendix B, for pile socket design should be the *'mobilised'* lateral bearing pressure, that being approximately 50% of allowable vertical end bearing pressure for socket strength check, plus an appropriate lateral deformation assessment.

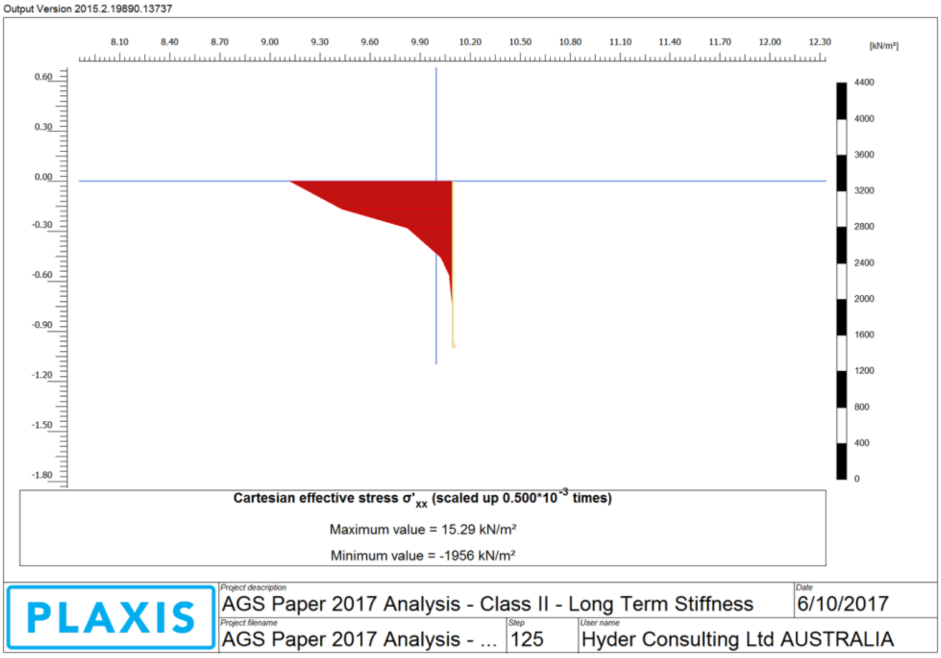

Figure 3 – Lateral bearing pressure distribution for a cantilever piled wall in Class II sandstone

5 RELIABILITY AND CONFIDENCE OF WEDGE FAILURE CHECK

5.1 AVAILABLE DATA AND RISKS

For most of the projects there are limited geotechnical data such as boreholes to characterise the defects in front of the retaining piled wall. The Australian Bridge code AS5100.3 (2004) specifies that the boreholes shall be drilled at 30m centres along the retaining wall. In practice, most projects would only provide boreholes at 50m to 100m centres along the piled wall due to either budget constraints or access to the locations. Furthermore, there is no specific requirement for the borehole data in the 3D context for defining the potential wedges in front of the wall. As such the reliability of the possible defects in front of the piled wall can only be assessed by means of available defect data in the proximity of the proposed piled walls. The designer would have to consider a range of the possible failure mechanisms and carry out sensitivity analyses to determine the minimum pile socket length below the bulk excavation level. The analysis method can be based on that shown on Figure 54 of Hong Kong Geoguide 1 in Appendix B.

5.2 CONSTRUCTION VALIDATION AND MINIMUM SOCKET LENGTH

It is often quite difficult for the contractor to fully appreciate the potential risks of wedge failure and the field mapping of the potential wedge formation is almost impossible. As such the author believes a minimum pile socket depth should be developed so that the reliability of the wall toe kick out can be enhanced.

Hong Kong Geoguide 1 proposes to have a minimum pile socket of 1m below the bulk excavation level. This is likely to be reasonable when the pile diameter is less than 1m. However, with the increase of pile diameter the minimum pile socket may not be valid. As such it is proposed to consider a minimum pile socket length of 1.3 times pile diameter, which is the low bound for cantilever wall for a 6 m high cantilever retaining wall, based on the as-constructed project database of the author. Table 4 presents a summary of minimum pile socket length based on previous projects that are in line with this proposed minimum pile socket length for either cantilever or propped/anchored wall.

Table 4: Recommended minimum pile socket length for 6m high cantilever wall under lateral loading

Pile Diameter (mm)	600	750	900	1050	1200	1500	2000	3000
Minimum Pile Socket Length (mm)*	780	980	1170	1370	1560	1950	2600	3900

*- The length was rounded to nearest 10mm.

It should be noted that the actual minimum embedment for other retaining wall, cantilever or propped or anchored retaining walls, will need to be assessed on a project specific case-by-case basis.

6 PROPOSED DESIGN CONSIDERATIONS

Based on the assessment and worked examples in the above sections a proposed design approach is summarised below:

6.1 VERTICALLY LOADED PILES

It should be recognised that the pile foundation design is often governed by the SLS condition due to the permissible deflection for the proposed structures. The ULS end bearing pressure will require large deflection which is often never realised under most of the structures.

For vertically loaded pile foundations the ULS end bearing pressures recommended by the geotechnical interpretative report based on Pells et al (1998) should be treated with caution as these high values are often not governing the design and could lead to unsafe design. The structural capacities of the pile itself under both SLS and ULS loading conditions should be established as these values are likely to be much lower than the ULS end bearing pressures for Class III sandstone or better. The *'mobilised'* end bearing pressure is encouraged for the designer to consider in the SLS checks. The debris remaining at the base of the pile after cleaning should be considered in the pile design for the *'mobilised'* end bearing capacity under SLS checks. This concept of the *'mobilised'* end bearing pressure would allow a level of risk of not perfect cleaning of debris at the base of pile.

The settlement of piles under the SLS load can be estimated using the rule of thumb of 0.5% to 1% of the pile diameter based on the correlations of the monitored results. Consideration of the allowable stress in rock under SLS loading condition is encouraged for the designer to ensure the pile foundation is to be within the permissible differential settlement either within a pile group or between the adjacent structural elements.

For the design reporting and documentation on the drawings it should note the practical constraints in assessing the rock quality by geotechnical engineers on site. The percentage of clay seams and defect spacing are often difficult to quantify on site unless there is a borehole immediately next to the pile shaft. These factors should be considered in determining the “mobilised” end bearing pressure for a vertically loaded pile.

6.2 LATERALLY LOADED PILES

There are no recommended lateral bearing pressures under both SLS and ULS provided by Pells et al (1998). The use of up to 50% of the ULS end bearing pressure of those corresponding vertical values are over-estimates for determining the socket length of piles in rock, especially for Class III sandstone or better. Based on analysis of past projects and numerical analysis of cantilever walls it is recommended that a minimum pile socket length of 1.3 times pile diameter be considered in the design of 6m high piled retaining structure or similar.

It is recommended that the “mobilised” lateral bearing pressure against the rock pile socket be 50% of the vertical allowable pressure, which is assumed to be 0.3 times UCS of rock based on AS2159 (1978). That is, the *‘mobilised’* lateral bearing pressure is estimated to be 15% of the UCS value of pile rock socket. Table 5 presented the low bound *‘mobilised’* lateral bearing pressure for each class of sandstone.

Table 5: Mobilised lateral bearing pressure for piles in sandstone

Sandstone Class and minimum UCS	Class I –24 MPa	Class II – 12 MPa	Class III – 7 MPa	Class IV – 3 MPa
Mobilised lateral bearing pressure (kPa)	3600	1800	1050	450

It is worth noting that for Class I sandstone the mobilised lateral bearing pressure is greater than 2000 kPa, which would require a deformation check for the pile where lateral deflection is critical, such as in the case of a cantilever wall.

Dependent upon the available defect data and adequacy of the borehole data the pile socket length should be checked against the minimum calculated length derived from a potential passive wedge sliding failure, using the method described on Figure 54, Appendix B. It must be recognised that this is only a best estimate, in that it will be nearly impossible for the geotechnical engineer to confidently assess the potential wedges given the drilled boreholes will be spaced at least 30 m centre to centre along the retaining wall as per AS5100.3 (2004). As such it is prudent to undertake sensitivity analyses to make a sound judgement for the pile socket based on available data.

The potential over-excavation and temporary trench excavation in front of the pile toe should be considered in the final pile socket length consideration.

7 CONCLUSIONS

This paper provides some notes on the pile foundation design under vertical and lateral loading conditions. For the vertical pile foundation design, it has been highlighted that the ULS end bearing only design for pile size could lead to unsafe design and the deformation check under the SLS load is usually more critical to determine the pile socket. A concept of the *‘mobilised’* lateral bearing pressure, as shown in Table 5, has been proposed for laterally loaded piles due to the unsafe design resulting from the misuse of the ULS lateral bearing pressure for lateral pile design. A minimum pile socket length of 1.3 times pile diameter for typical retaining structures of 6 m height has also been proposed based on the FEM analysis and the author’s database of the as-built records of previous projects. It must be emphasised that lateral deformation check under SLS load should be carried out when a *‘mobilised’* lateral bearing pressure is greater than 2000 kPa. The lateral potential wedge stability check, based on Figure 54 in Appendix B, is onerous due to lack of reliable geotechnical data on defect details. Therefore it is necessary to undertake a sensitivity analyses to make a reasonable judgement if this is a governing case or not.

8 ACKNOWLEDGEMENT

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APPENDIX A

Extract from “Allowable Stresses in Piles”, U.S. Department of Transportation, Federal Highway Administration, Report No FHWA/RD-83/059, First Published 1983.

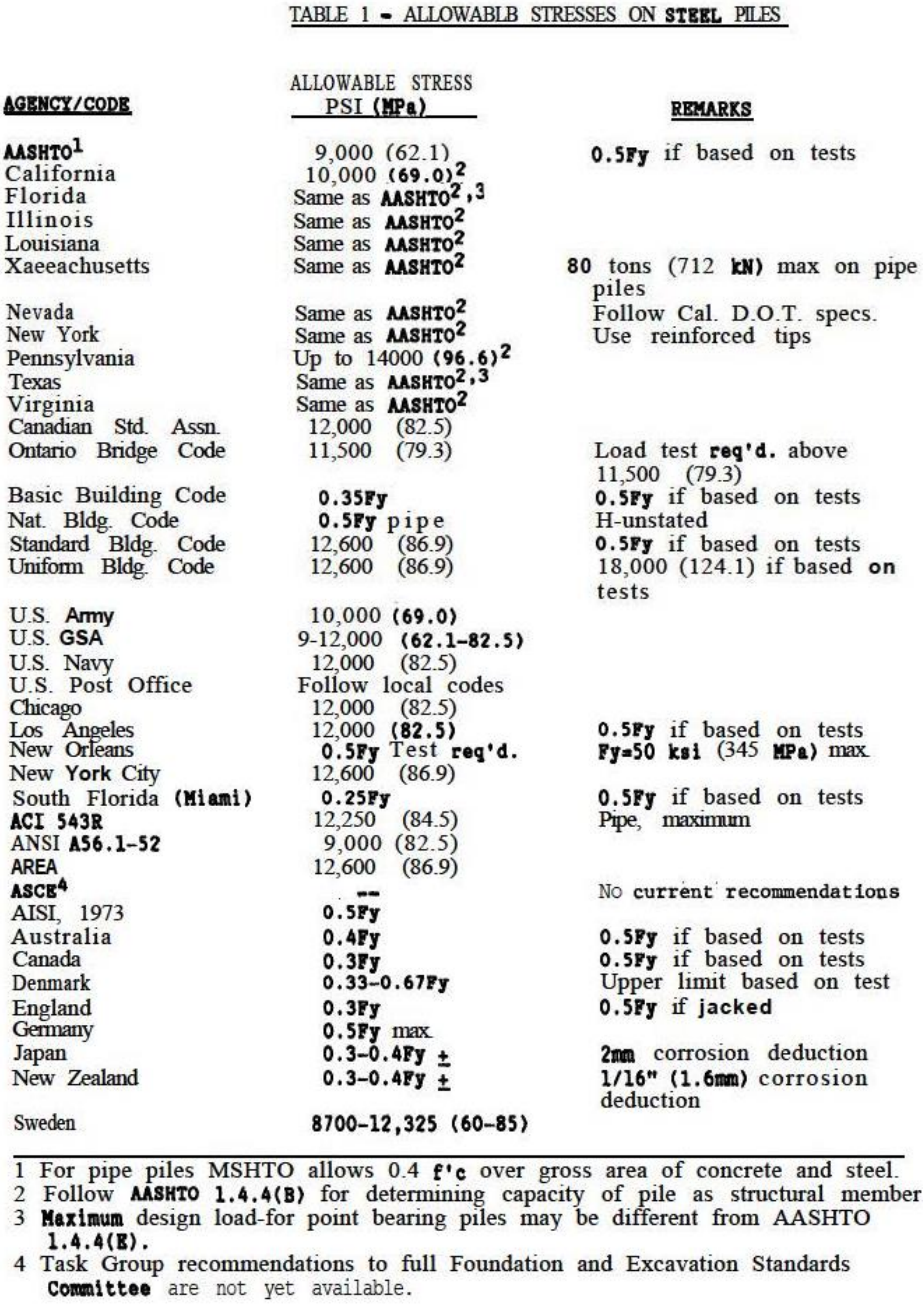

TABLE 1 - ALLOWABLB STRESSES ON STEEL PILES

AGENCY/CODE	ALLOWABLE STRESS PSI (MPa)	REMARKS
AASHTO[1]	9,000 (62.1)	0.5Fy if based on tests
California	10,000 (69.0)[2]	
Florida	Same as AASHTO[2,3]	
Illinois	Same as AASHTO[2]	
Louisiana	Same as AASHTO[2]	
Xaeeachusetts	Same as AASHTO[2]	80 tons (712 kN) max on pipe piles
Nevada	Same as AASHTO[2]	Follow Cal. D.O.T. specs.
New York	Same as AASHTO[2]	Use reinforced tips
Pennsylvania	Up to 14000 (96.6)[2]	
Texas	Same as AASHTO[2,3]	
Virginia	Same as AASHTO[2]	
Canadian Std. Assn.	12,000 (82.5)	
Ontario Bridge Code	11,500 (79.3)	Load test req'd. above 11,500 (79.3)
Basic Building Code	0.35Fy	0.5Fy if based on tests
Nat. Bldg. Code	0.5Fy pipe	H-unstated
Standard Bldg. Code	12,600 (86.9)	0.5Fy if based on tests
Uniform Bldg. Code	12,600 (86.9)	18,000 (124.1) if based on tests
U.S. Army	10,000 (69.0)	
U.S. GSA	9-12,000 (62.1-82.5)	
U.S. Navy	12,000 (82.5)	
U.S. Post Office	Follow local codes	
Chicago	12,000 (82.5)	
Los Angeles	12,000 (82.5)	0.5Fy if based on tests
New Orleans	0.5Fy Test req'd.	Fy=50 ksi (345 MPa) max.
New York City	12,600 (86.9)	
South Florida (Miami)	0.25Fy	0.5Fy if based on tests
ACI 543R	12,250 (84.5)	Pipe, maximum
ANSI A56.1-52	9,000 (82.5)	
AREA	12,600 (86.9)	
ASCE[4]	--	No current recommendations
AISI, 1973	0.5Fy	
Australia	0.4Fy	0.5Fy if based on tests
Canada	0.3Fy	0.5Fy if based on tests
Denmark	0.33-0.67Fy	Upper limit based on test
England	0.3Fy	0.5Fy if jacked
Germany	0.5Fy max.	
Japan	0.3-0.4Fy ±	2mm corrosion deduction
New Zealand	0.3-0.4Fy ±	1/16" (1.6mm) corrosion deduction
Sweden	8700-12,325 (60-85)	

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1 For pipe piles MSHTO allows 0.4 f'c over gross area of concrete and steel.

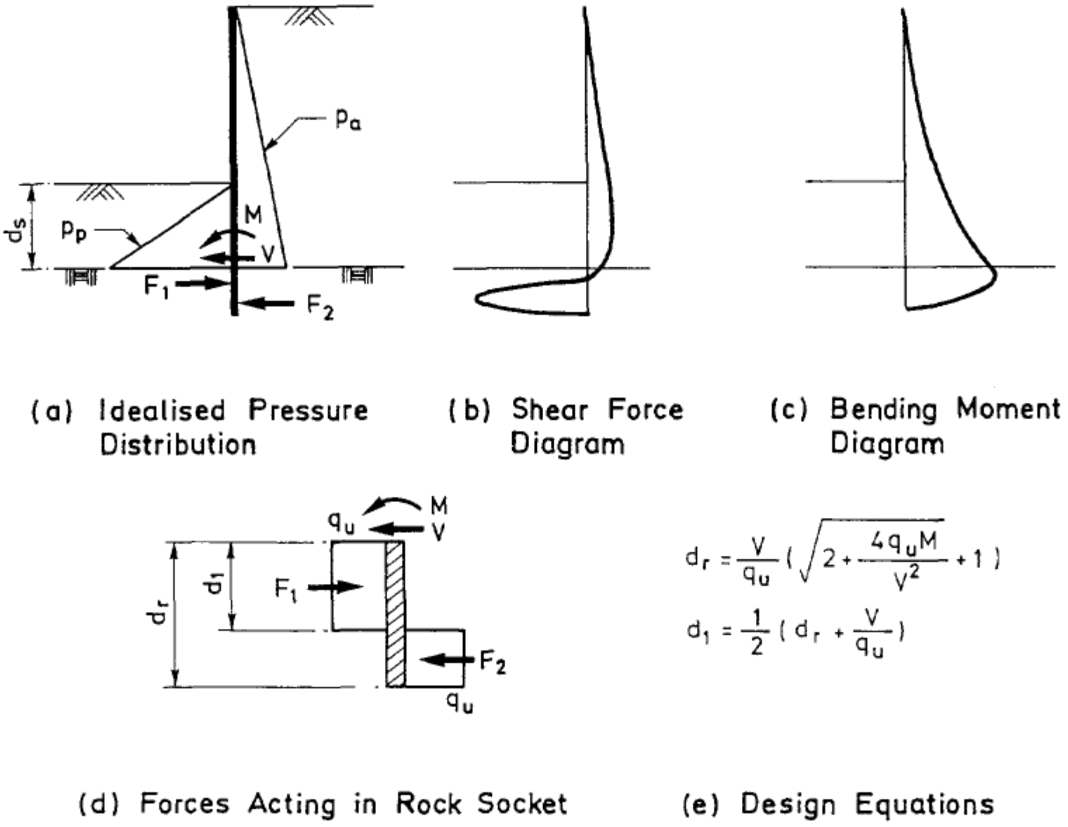
2 Follow AASHTO 1.4.4(B) for determining capacity of pile as structural member

3 Maximum design load-for point bearing piles may be different from AASHTO 1.4.4(E).

4 Task Group recommendations to full Foundation and Excavation Standards Committee are not yet available.

APPENDIX B

(after Hong Kong Geoguide 1-Guide to Retaining Wall Design, 1993)



(a) Idealised Pressure Distribution

(b) Shear Force Diagram

(c) Bending Moment Diagram

(d) Forces Acting in Rock Socket

$$d_r = \frac{V}{q_u}\left(\sqrt{2+\frac{4q_uM}{V^2}}+1\right)$$

$$d_1 = \frac{1}{2}\left(d_r+\frac{V}{q_u}\right)$$

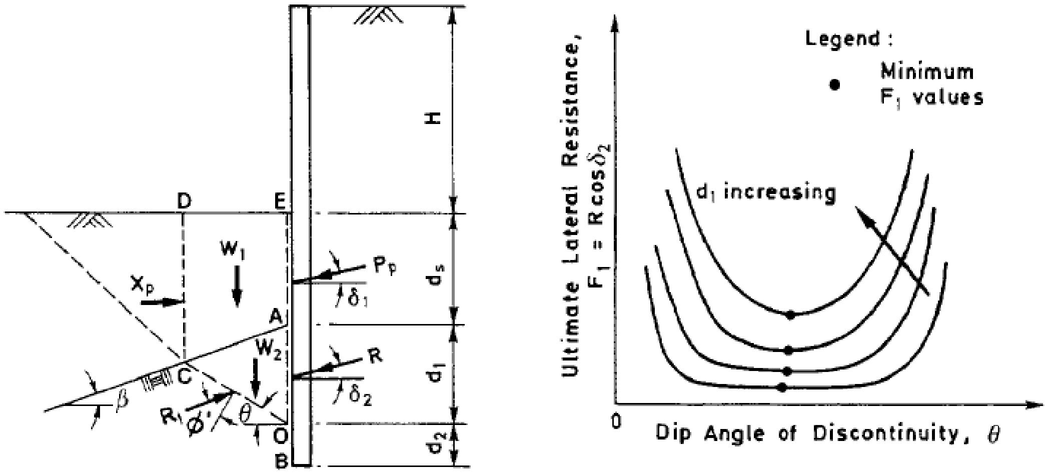
(e) Design Equations

Legend :

F_1,F_2	Reactions in top and bottom portions of rock socket respectively
M,V	Bending moment and shear force respectively at the top of rock socket
q_u	Ultimate lateral bearing capacity of rock

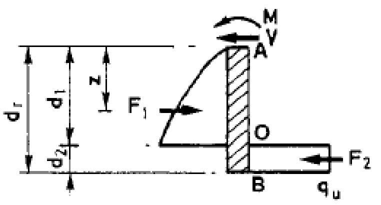
Notes :
(1) For cantilevered sheet retaining walls, M, V, F_1 and F_2 should be calculated per unit width of the wall.
(2) For a wall composed of vertical members of width D at a spacing S, the values of M and V to be used in the above equations should be calculated based on active and passive earth pressures acting over a width equal to S. An exception to this is where S > 3D, in which case the contribution from the active and passive earth pressures below excavation level should be based on an effective width of 3D only. F_1 and F_2 should always be based on a width equal to D, i.e. q_u should be multiplied by D.
(3) The possibility of excavation in front of the wall should be allowed for in evaluating passive resistance.

Figure 51 - Design of Rock Socket Against Bearing Failure for the Case Where the Top of the Socket is Above the Point of Zero Shear



(a) Model for Calculating Rock Socket Capacity Against Planar Discontinuity-controlled Failure

(b) Variation of Rock Socket Capacity with Dip Angle of Discontinuity



(c) Forces Acting in Rock Socket

$$d_1 = \frac{M + F_1 z}{F_1 - V} - \frac{F_1 - V}{2q_u} \quad \text{(full equilibrium)}$$

$$d_1 = \frac{M}{F_1 - V - F_1 (z/d_1)} \quad \text{(simplified)}$$

$$d_r = d_1 + \frac{F_1 - V}{q_u}$$

(d) Design Equations

Legend :

F_1, F_2	Reactions at the top and bottom portions of the rock socket respectively
M, V	Bending moment and shear force respectively at the top of rock socket
P_p, X_p	Passive forces acting on the vertical boundaries AE and CD
R, R_t	Reactions acting on the presumed rock wedge
W_1, W_2	Weight of soil in ACDE and weight of rock in AOC respectively
δ_1, δ_2	Mobilised angles of wall friction in soil and in rock respectively
ϕ'	Angle of shearing resistance of the presumed discontinuity OC

Notes :

(1) If point A is below the point of zero shear, then the direction of V reverses.

(2) For cantilevered sheet retaining walls, M, V, F_1 and F_2 and R should be calculated per unit width of the wall.

(3) For a wall composed of vertical members of width D at a spacing S, the values of M and V to be used in the above equations should be calculated based on active and passive earth pressures acting over a width equal to S. An exception to this is where $S > 3D$, in which case the contribution from the earth pressures below excavation level should be based on an effective width of 3D only. The values of R, and hence F_1, may be based on a width equal to S. However, F_2 should always be based on a width equal to D only, i.e. q_u should be multiplied by D.

(4) The possibility of excavation in front of the wall should be allowed for in evaluating passive resistance.

(5) Where the depth of soil cover in the passive zone is large, the calculated d_1 value can be negative. For such cases, reference should be made to Figure 53.

Figure 54 - Design of Rock Socket Against Planar Discontinuity-controlled Failure