20 YEARS OF AS 2870 IN S.E. QUEENSLAND: A PERSONAL VIEW

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SUMMARY
This paper is about AS 2870 in its various revisions and addendums and its interpretation in SE Queensland with respect to site classification.

1 INTRODUCTION
AS 2870 was introduced throughout Australia in 1986. Some jurisdictions were a little slow adopting it and in SE Queensland a seminar presented by P. Walsh and D. Cameron at the Main Roads office in Spring Hill in 1986 divided those present into two groups. Group A were the believers who were thankful they now had a document which would allow domestic allotments to be interpreted in accordance with common criteria and allow the use of consistent footing systems. Group B were the non-believers who rejected the idea that six classifications would cover everything and insisted that each allotment would require a specifically designed footing.

AS 2870 also brought into common usage terms e.g. $y_s$, cracked zone, instability index, which were unknown to most consultants. Twenty years on most of the non-believers are now believers and many of the original believers are now wondering why such a simple document now appears to be very complicated.

2 THE DOCUMENTS
The evolution of AS 2870 has been as follows:

<table>
<thead>
<tr>
<th>Document</th>
</tr>
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<tbody>
<tr>
<td>AS 2870-1986</td>
</tr>
<tr>
<td>AS 2870-1986 – Commentary</td>
</tr>
<tr>
<td>AS 2870-1986 – Amendment June 1987</td>
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<tr>
<td>AS 2870.1-1988</td>
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<tr>
<td>AS 2870 Supp 1- 1988</td>
</tr>
<tr>
<td>AS 2870 Supp 2 - 1988</td>
</tr>
<tr>
<td>AS 2870.2-1990</td>
</tr>
<tr>
<td>AS 2870-1996</td>
</tr>
<tr>
<td>AS 2870-1996 Supp 1 1996</td>
</tr>
<tr>
<td>AS 2870-1996 Amendment No 1 – January 1997</td>
</tr>
<tr>
<td>AS 2870-1996 Amendment No 2 – June 1999</td>
</tr>
<tr>
<td>AS 2870-1996 Amendment No 3 – November 2002</td>
</tr>
<tr>
<td>AS 2870-1996 Amendment No 4 – May 2003</td>
</tr>
</tbody>
</table>

2.1 AS 2870-1986
How simple it was! There were six classes, five of which were defined by a $y_s$ range and a sixth class for problem sites. The classes and $y_s$ ranges were

<table>
<thead>
<tr>
<th>CLASS</th>
<th>$y_s$ RANGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0-10 mm</td>
</tr>
<tr>
<td>S</td>
<td>10 mm – 20 mm</td>
</tr>
<tr>
<td>M</td>
<td>20 mm – 40 mm</td>
</tr>
<tr>
<td>H</td>
<td>40 mm – 70 mm</td>
</tr>
<tr>
<td>E</td>
<td>&gt;70 mm</td>
</tr>
<tr>
<td>P</td>
<td>Problem Sites</td>
</tr>
</tbody>
</table>

Problem sites were defined as;

2.5 PROBLEM SITES. Where the site includes mine subsidence, uncontrolled fill, landslip conditions or soft soils (see Clause 2.3.5), the site shall be classified as a problem site (Class P) and a footing system shall be designed in accordance with Section 5.

The following guidelines were given for clay sites:
2.3.3 Clay sites. In addition to the general requirements of Clause 2.2, the procedure for the classification of a clay site shall include one or more of the following methods:

(a) Visual assessment of the site and interpretation of knowledge of existing masonry house walls on light strip footings which have existed for not less than 15 years in a similar soil assessed in accordance with Table 2.1.

(b) Identification of the soil profile and a classification in accordance with Appendix C or from established data on the performance of the soil profile.

(c) Computation of the predicted surface movement, $y_s$, in accordance with Appendix D, with the following limits:

<table>
<thead>
<tr>
<th>Surface Movement</th>
<th>Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>$y_s \leq 20$ mm</td>
<td>S</td>
</tr>
<tr>
<td>$20 &lt; y_s \leq 40$ mm</td>
<td>M</td>
</tr>
<tr>
<td>$40 &lt; y_s \leq 70$ mm</td>
<td>H</td>
</tr>
<tr>
<td>$y_s &gt; 70$ mm</td>
<td>E</td>
</tr>
</tbody>
</table>

2.1.1 Methods Adopted by Consultants

As the “computation method” (option (c) above) was the third and last option, most consultants chose not to use it and generally used their existing methods with modifications to arrive at one of these new classes. The computation method even today is the subject of debate. One school of thought is that to classify a site, a $y_s$ is required. The only way to calculate this $y_s$ is by the computation method, therefore the following formula must be applied:

$$y_s = \frac{1}{100} \int_0^H I_{pt} \Delta u \Delta h$$

where

- $y_s$ = characteristic surface movement
- $I_{pt}$ = instability index
- $\Delta u$ = suction change at depth (h) from the surface, expressed in pF units
- $\Delta h$ = thickness of soil layer under consideration (in metres)

Then $I_{pt}$ was defined as:

- $I_{pt} = \infty I_{ps}$ in the cracked zone
- $I_{pt} = 2 - z/5$ in the uncracked zone

with

- $I_{ps} = Soil$ Shrinkage Index
- $z = midpoint$ of the layer being considered

Then the following conversions were given to reach an $I_{ps}$:

<table>
<thead>
<tr>
<th>Test</th>
<th>Symbol</th>
<th>Conversion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shrink/swell index</td>
<td>$I_{ss}$</td>
<td>$I_{ss} = I_{ps}$</td>
</tr>
<tr>
<td>Loaded shrinkage index</td>
<td>$I_{ls}$</td>
<td>$0.9 I_{ls} = I_{ps}$</td>
</tr>
<tr>
<td>Core shrinkage Index</td>
<td>$I_{cs}$</td>
<td>$I_{cs} = I_{ps}$</td>
</tr>
</tbody>
</table>

To attain a $y_s$ value by the computation method, all the consultant had to do was – drill a hole to determine the soil profile, do an appropriate laboratory test to arrive at an $I_{ps}$, and then use the other standard parameters which were predetermined and quoted in AS 2870. These parameters for Brisbane were:

- $H_s = 1.5$ metres
- $\Delta u = 1.2$ pF
- Cracked zone = $0.75xH_s$ (1.25 metres)

AS 2870 did not have a table for cracked zones, but offered the following advice for cities mentioned on Table D1.

Clause D 2.3 (in part) “In the absence of more specific information, the depth of the cracked zone can be taken as $0.75H$ where $H$ is given in Table D1 with other areas being assessed on the basis of climate.”

So, with no real way for consultants outside Brisbane to assess the cracked zone on the basis of climate, $0.75 \ H_s$ was adopted throughout SE Queensland (along with $H_s$ and $\Delta u$ because they also had no way of recalculating these either).
In AS 2870 Supp1-1988, the following advice was offered in Clause CD2 (in part);

"Although 0.75 H is a reasonable average estimate for Australia, in the more temperate climates a lower value may be appropriate. It has been suggested that H/3 is appropriate for Sydney."

Therefore, with no specific reference to SE Queensland, 0.75 \( H_s \) was still used.

In AS 2870.2 – 1990, the following was quoted in Clause D2.3 (in part);

"The depth of the cracked zone can be taken as 0.25 \( H \) to 0.75 \( H \) where \( H \) is given in Table D1 with other areas being assessed on the basis of climate. In Adelaide and Melbourne, the depth may be estimated as 0.75 \( H \). In the Hunter Valley, a value of 0.25 \( H \) is recommended and for Sydney 0.33 \( H \)."

And with no further reference to SE Queensland, 0.75 \( H_s \) continued to be used.

The second school of thought was that this was a “southern states code” but in order to reach a classification (and in some cases a \( y_c \)) there was modification of the consultants’ existing practices. A variety of methods was used, mostly related to Atterberg Limits and Linear Shrinkages, which was the preferred method of assessing soil reactivity prior to 1986, and is still used by some consultants today.

Most of these methods allowed for changes within a soil profile, but for simplicity within this paper, we have assumed a geotechnical model of a homogeneous stratum extending deeper than \( H_s \), with good bearing and no other issues such as mining subsidence, slope instability etc.

Method A \( y_c = LS \times 2.88 \)
Method B \( I_{ps} = (0.7234 \times LL \times \% \text{ finer } 75 \mu m \times 10^{-3}) - 0.5765 \)
Method C \( I_{ps} = 0.1 \times LL (\text{for } LL > 25) \)
Method D \( I_{ps} = 0.42 \times LS \)
Method E \( I_{ps} = 0.58 (LS - 4) \)
Method F \( I_{ps} = 0.14 (LL - 20) \)

A third school of thought existed (and still exists today) that rather than relying only on laboratory testing, drilling boreholes and assessing the strata based on experience was the best way to classify a site and this method is still being used.

### 2.2 AS 2870-1996

This edition of AS 2870 was a significant revision and it set up specific \( H_s \) parameters for \( y_c \) calculations. With respect to SE Queensland, these were;

Brisbane/Ipswich = 1.5 metres to 2.3 metres
Toowoomba = 1.8 metres to 2.3 metres

It also offered advice that “The variation in \( H_s \) depends largely on climatic variation.”, but it did not include any maps delineating the boundaries. I believe Toowoomba consultants, using local knowledge, decided on the 1.8/2.3 boundaries, but in Brisbane there was no consensus and most consultants decided that Brisbane City was deemed to be 1.5 m and the Ipswich general area was 2.3 m. Many new estates were being established in the Brisbane/Ipswich corridor and \( H_s \) values between 1.5 m and 2.3 m were adopted with little uniformity between consultants.

In other areas, in-house judgements were made whether an estate or town should be in the 1.5 m, 2.3 m or in the “somewhere in between” zone.

Clause F2 of AS 2870-1996 also advised that the cracked zone for Brisbane/Ipswich was recommended as 0.5 \( H_s \) and this was the first time AS 2870 specifically mentioned this parameter for SE Queensland.

If, for simplicity’s sake, we assume that post 1996, SE Queensland has three \( H_s \) zones (1.5 m, 1.8 m and 2.3 m), and a cracked zone which has changed from 0.75 \( H_s \) (pre-1996) to 0.5 \( H_s \) (post-1996) then the following table of changes in \( y_c \) values is relevant for the geotechnical model outlined in Section 2.1.1;
On the face of it, this table looks innocuous, but let’s choose two examples to see what the changes in AS 2870-1996 really meant;

**Example 1** – A geotechnical model as in our Section 2.1.1, but with an $I_{ps}$ value of 4.0%.

<table>
<thead>
<tr>
<th>Period</th>
<th>$H_s$</th>
<th>$y_s$</th>
<th>Rounded $y_s$</th>
<th>% Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-1996</td>
<td>1.5 m</td>
<td>37.66 mm</td>
<td>40 mm</td>
<td>-</td>
</tr>
<tr>
<td>Post-1996</td>
<td>1.5 m</td>
<td>43.14 mm</td>
<td>45 mm</td>
<td>12.5%</td>
</tr>
<tr>
<td>Post-1996</td>
<td>1.8 m</td>
<td>51.33 mm</td>
<td>50 mm</td>
<td>25%</td>
</tr>
<tr>
<td>Post-1996</td>
<td>2.3 m</td>
<td>64.64 mm</td>
<td>65 mm</td>
<td>62.5%</td>
</tr>
</tbody>
</table>

**Example 2** – A geotechnical model as per our Section 2.1.1 but with an $I_{ps}$ value of 7.0%.

<table>
<thead>
<tr>
<th>Period</th>
<th>$H_s$</th>
<th>$y_s$</th>
<th>Rounded $y_s$</th>
<th>% Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-1996</td>
<td>1.5 m</td>
<td>65.9 mm</td>
<td>65 mm</td>
<td>-</td>
</tr>
<tr>
<td>Post-1996</td>
<td>1.5 m</td>
<td>75.5 mm</td>
<td>75 mm</td>
<td>15.4%</td>
</tr>
<tr>
<td>Post-1996</td>
<td>1.8 m</td>
<td>89.82 mm</td>
<td>90 mm</td>
<td>38.5%</td>
</tr>
<tr>
<td>Post-1996</td>
<td>2.3 m</td>
<td>113.11 mm</td>
<td>115 mm</td>
<td>76.9%</td>
</tr>
</tbody>
</table>

The implications of the above tables should be self-evident, but in effect the consequences are that if in pre-1996 a site in Ipswich had a calculated $y_s$ of 65 mm, post-1996 (using the same methodology but the updated standard parameters in AS 2870-1996), a $y_s$ of 115 mm would be applicable for the same site.

### 3 DEPTH OF SEASONAL MOISTURE VARIATION ($H_s$)

This parameter was originally designated “$H$”, but in AS 2870-1996 it was changed to $H_s$. Unless part of a direct quote, the term “$H_s$” is used by preference throughout this paper.

These are based on the Thornthwaite Moisture Indices and at best, the maps of the past have been of insufficient scale to accurately determine boundaries; the determination of the indices belongs to another science (meteorology), and the guidelines in AS 2870-1996 (with the exception of the maps for Victoria) have been minimal. However in Amendment 2 to AS 2870-1996 (June 1999), the following note appeared (Page 17, in part):

“*As a guide to designers, the values of design suction changes are given in Table 2.4 for those locations where data are (sic) available. The designer may extrapolate to other areas if due consideration is given to the climate and soil fabric. Alternatively, published values of $H_s$ based on consideration of regional Thornthwaite moisture indices, using the general principles in Appendix D and based on at least 20 years of climate data, may be used.*”

Back in 1999, this note did little to clarify the zones, but since that time, the following papers have been published so this note takes on greater importance.


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1 In Clause C2.2.3 of AS 2870 Suppl-1996, it is recommended that the $y_s$ be reported to the nearest 5 mm.
• Fox, E; (Australian Geomechanics 2002) “Development of a Map of Thornthwaite Moisture Index Isopleths for Queensland”
• McManus, Dr K; Lopes, D; Osman, N Y; (9th ANZ Conference on Geomechanics, Auckland, 2004) “The Effect of Thornthwaite Moisture Index Changes on Ground Movement Predictions in Australian Soils”

Copies of the maps accompanying these papers have been included at the end of this paper and whilst the definition of these maps is not as precise as would be preferred, they do clearly indicate that not only is only a small area of SE Queensland in the $H_s = 1.5$ metre zone, but the $H_s = 3.0$ metre zone has encroached into just west of Toowoomba.

4 ABNORMAL MOISTURE CONDITIONS

Pre-1996, the importance of abnormal moisture conditions was, in the view of the author, understated in the various editions of AS 2870, unless the appendix and referenced CSIRO publication Sheet 10-9 were studied.

AS 2870-1996, Section 1.3 gave this area more prominence.

Consultants have applied various methods of dealing with this issue, from disregarding it completely to fully embracing its implications, and all points between. There seems to be a reluctance by many consultants to classify sites affected by abnormal moisture conditions as Class P, but, at various seminars, speakers such as Fox, Lopes, Cameron and others have clearly stated that this was the intent. The conditions classified as “abnormal” according to AS 2870-1996 in Clause 1.3.3 are:

1) Recent removal of an existing building or structure likely to have significantly modified the soil moisture conditions under the proposed plan of the building.
2) Unusual moisture conditions caused by drains, channels, ponds, dams or tanks which are to be maintained or removed from the site.
3) Recent removal of large trees prior to construction.
4) Growth of trees too close to a footing.2
5) Excessive or irregular watering of gardens adjacent to the house.2
6) Lack of maintenance of site drainage.2
7) Failure to repair plumbing leaks.2

It is quite clear that if the strata encountered during testing has the potential to change volume with changes in soil moisture, and if abnormal moisture conditions exist, then AS 2870-1996 requires a “P” classification. A consultant recently confided to the author “If I embrace the abnormal moisture conditions provisions, then about 50% of my site classifications will be Class P.”, to which the author replied, “The figure is probably closer to 70-80%.”

4.1 SIMPLE CASE OF ABNORMAL MOISTURE CONDITIONS

Two flat sites with identical Class M soils, adequate bearing, $y_s$ in the range 35 – 40 mm. Site B has a 30 m high tree in the middle of the building footprint.

We assume that the tree will be removed only days prior to slab construction so there will be insufficient time for the onsite soils to re-hydrate naturally.

Assume the following moistures at the time the tree is removed;

Site A 20 – 25%
Site B 10 – 15%

After construction, the soils under a slab naturally wet up to a new equilibrium (which is generally about 30% – 35%). This will happen on both sites, but will take years to occur (generally 3 – 5 years), therefore;

Site A has gone from 20-25% to 30-35% which is about a 50% increase.
Site B has gone from 10-15% to 30-35%, which is about a 200% increase.

As the $y_s$ quantifies clay soil movements which are directly related to soil moisture variations, obviously the soil movement under the slab on Site B will be greater than that under Site A and will therefore require a stronger slab. Unfortunately AS 2870-1996 states; “Quantifying these movements is beyond current scientific knowledge”; “…but these movements must be considered.”

This situation becomes more complicated with groups of trees, neighbouring trees and sites with existing houses and gardens (knock-downs/rebuilds). If a timber floor house is demolished, it will be dry underneath (maybe drier than if a

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2 Generally these latter four conditions occur/develop post-construction and are beyond the control of the classifier.
tree was there), but where a dwelling on a slab is removed, the sub-floor will already be reasonably moist; however the new dwelling will most likely be larger than the old, so the new house will be partially on moist soil and partially on the dry adjacent garden soil or sub-floor soils.

**Simple Solution** – prepare the site with sufficient time gap after clearing and prior to slab construction for the strata to naturally re-hydrate.

**Problem** – This time period is undefinable within current scientific knowledge and, dependent on climate at the time, it could take anywhere between 3 months and a few years.

**Complex Solution** – When preparing the site, artificially re-hydrate the soils.

**Problem** – No agreed method, difficult to control, difficult to quantify.

4.2 MORE COMPLEX (BUT NOT RARE) CASE OF ABNORMAL MOISTURE CONDITIONS

Site at time of testing – timber floor house with a leaking roof gutter at front, garage on slab at rear left, above-ground swimming pool with lush grass around and row of trees along right boundary. The new building footprint will encompass all of these moisture conditions. The area under the existing house has been consolidated for many years and the rest of the site has not. (This is particularly important on filled sites on the Gold Coast, where preloading/long term settlements are an issue). If the footing solutions in the Code are not capable of withstanding simple and common “abnormal” moisture conditions, some guidance must be given, otherwise the Code will only apply to 30% - 40% of sites.

AS 2870 has always been aware of the potential for home owners to change a “normal” site to an “abnormal” one, through landscaping and watering practices and one of the appendices in AS 2870 entitled “Performance Criteria and Foundation Maintenance” has always been part of the Code. It was Appendix A in 1986 and it is Appendix B in AS 2870-1996.

This appendix has always directed readers to the CSIRO Information Sheet “Guide to Home Owners on Foundation Maintenance and Footing Performance” (also known as Sheet 10-91). This CSIRO sheet was usually handed to the new home owner by the builder but in 2003 this sheet was superseded by BTF 18 and in this re-write some of the references to trees have been removed and builders are now reluctant to hand this updated version to the home owner. In 2004, the Building Services Authority released a booklet entitled “A Simple ‘How to’ Guide to Preventing Structural Damage to your Home”, which in plain language explains the relationship between reactive clays, foundations, landscaping and drainage. This booklet will go a long way in educating home owners about these issues and will rapidly become the document of choice for builders to pass onto homeowners in Queensland.

5 RECLASSIFYING FILLED SITES

AS 2870 has always allowed the reclassification of adequately compacted filled sites and Clauses 2.4.6 and 6.4.2 should be referred to.

The concern is that the link in AS 2870-1996 to AS 3798-1996 is ambiguous as it does not refer to which level of certification (Level 1, 2 or 3 in AS 3798-1996) is appropriate. Therefore, where the site classifier doesn’t have control of the earthworks, he must rely on the opinion of the geotechnical testing company in charge of the testing.

On large scale developments, this is usually not an issue as the level of certification is almost always stated but, on individual house lots, rarely will the person testing the compaction warrant the earthworks as sufficient to proceed without piers. Clearly this leaves the site classifier with no alternative but to define the fill as “uncontrolled” and classify the site as Class P, and the engineer will have to incorporate piers in his design. The shortcomings of this approach are that if the fill is clay based and has been compacted, it will have the potential to heave. The site classifier has probably given advice about the reactivity of the natural soil, but not the fill and the design engineer may not be aware of the heave potential of the fill when preparing his design.

Clause 2.4.6 c (ii) of AS 2870-1996 recommends the depth of the cracked zone should be taken as zero for reactive clay in controlled fill placed less than 5 years prior to building construction. This seems a straightforward clause until the $y_s$ calculation is carried out. Again, using the geotechnical model in Section 2.1.1 but this time cutting the site by 1200 mm and filling by 1200 mm, then compacting the fill to support a high level footing without piers. This zero cracked zone has the following effect on the $y_s$ value, using the post-1996 parameters for SE Queensland;

| Table 4 |
As Table 4 indicates, compacting of clay fill significantly increases the $y_s$ value and in most cases, a 50 – 66% increase is not unreasonable. Although in the above example the post-1996 parameters have been used, this “zero cracked zone” clause first appeared in Section 2.3 of AS 2870.2-1990. The author believes the Code should give this situation and calculation more prominence since it is clearly not being considered by many classifiers or engineers.

### 6 LOWLAND FILLED ESTATES

This is particularly important in SE Queensland and these estates fall into two categories:

a) Areas where the proposed development is underlain by strata capable of supporting the fill and the proposed structure without any bearing pressure/consolidation issues.

b) Areas where there will be long-term consolidation of the strata at depth, which has the potential to influence the performance of the structures.

The former is easily dealt with within the provisions of Clause 2.4.6 of AS 2870-1996, but the latter is literally a can of worms and there are significant coastal areas from Noosa to Coolangatta (as well as in other states where there are underlying swamp or mangrove deposits) where long-term consolidation is an ongoing issue. Where these allotments fall into the AS 2870-1996 classification is not clear.

These sites are clearly underlain by “soft” or “collapsing” soils so, according to Clause 2.1.2 they should be classified as Class P, however Clause 1.3.2 “Normal Sites” (which was amended in January 1997) states;

“No normal sites are those which are classified as one of the Classes A, S, M, H and E in accordance with Section 2 of this Standard and where foundation moisture variations are caused by seasonal and climatic changes, effect of the building and subdivision and normal garden conditions, without abnormal moisture conditions (see Clause 1.3.3). Compliance with the recommendations in CSIRO 10-91 is deemed to provide normal garden conditions.”

If the long-term consolidation can be quantified, does the term “effect of the building and subdivision” mean that a classification other than Class P can be used?

When these estates are developed, the following procedures are implemented:

- Initial geotechnical investigation, normally down to beyond the depth of compressible strata which is sometimes deeper than 30 metres.
- Once the parameters of the underlying strata are known, the estate is planned.

This planning is generally designed with a target of either Class M or Class H type slabs being used.

By commencing with the final classification in mind, the earthworks and development are strictly controlled and there are procedures which can be varied to ensure that the target classifications are reached. These include (but are not limited to) depth of fill, depth of pre-load, time of pre-load (can be shortened by the use of wick drains), the shrink/swell potential of the strata within $H_s$ of finished pad level (this controls the $y_s$).

All of the above is based on the most critical factor of all – the long-term “house plus landscaping” load which in recent times is normally assumed to be 20 kN. If the final result is “suitable for a Class H type footing” what this really means is that performance according to the AS 2870-1996 performance criteria can be expected, providing the house and landscape load does not exceed 20 kN. It doesn’t mean that the $y_s$ is in the range 40–70 mm, because $y_s$ is defined in Clause 1.7.8 of AS 2870-1996 thus;

“The movement of the surface of a reactive site caused by moisture changes from characteristic dry to characteristic wet condition in the absence of a building and without consideration of load effects.”

The author believes that the 40–70 mm figure should not be considered $y_s$, because it doesn’t meet the above definitions criteria. This 40-70 mm figure quoted for these estates is the sum of the actual $y_s$, plus the calculated differential
settlement predicted under the 20 kN load, which is a percentage of both the remaining primary and secondary settlements under this load (which is another topic of debate).

The footing designers must be aware that if the load increases above 20 kN, although the \( y_i \) won’t change, the long term settlements may increase significantly, and if they increase to over the Class H design criteria, adverse performance may develop.

All of the above is applicable for land-locked allotments, but on canal allotments, there is a new variable - batters down to the revetment wall which are generally not pre-loaded. In most cases, to achieve AS 2870-1996 type performance on these batters, the applied load reduces to somewhere in the 5–8 kN range, which in some cases equals a load applied by about 500 mm of compacted fill. This basically excludes any construction on these batters on high level footings, including swimming pools in some cases.

As it appears that common usage in SE Queensland now includes these deeper movements within the \( y_i \) figure, the Code should clarify whether this was the intent of the \( y_i \), or that the classification cannot be varied from Class P.

The engineer must have sufficient information to assess all ground movement, so perhaps the solution is to have a new parameter which is the sum of all known differential movements. This new parameter would also have some application in the abnormal moisture situation, where various other writers have used a “\( Y_{ST} \)” or “Abnormal \( y_i \)”.

7 MINING SUBSIDENCE

This is referred to in Section 2.1.2 and up until the early 1990’s this was an issue in the Ipswich Coalfields, but the government has bought back many unsuitable allotments and converted them to parks, and in the newer estates these areas are now excluded from the “buildable areas” at the time of subdivision.

There are occasional problems in the Gympie Goldfields and local consultants are generally more aware of the suspect areas than the Brisbane based consultants, but caution must always be exercised in mining areas.

8 SLOPE STABILITY

This also falls under the Class P classification of Clause 2.1.2 of AS 2870-1996 and unfortunately many site classifiers either ignore it, or put in a disclaimer in their reports similar to; “This site classification report hasn’t considered the slope stability of the site.” Most people relying on the site classification report interpret this statement as meaning that there are no issues related to slope instability, whereas it really means that the classifier doesn’t know if there is a problem.

In SE Queensland there is a wealth of knowledge on slope stability areas and some of this information is in the following publications, all of which are in the public domain. The following publication and maps can be purchased from the Gold Coast City Council;

SMEC Australia Pty. Ltd; 1999 -Landslip Study for the City of Gold Coast.

The following geological survey reports and maps are also available from the Department of Natural Resources;

Willmott, WF 1981/14 -Slope stability and its constraints on closer settlement on the Tamborine Mountain, SE Qld
Willmott WF 1983/09 -Slope stability and its constraints on closer settlement on the Mapleton Maleny Plateau, SE Qld
Willmott WF 1983/64 -Slope stability and its constraints on closer settlement in the Canungra Beechmont Numinbah area, SE Qld
Hofmann GW, Willmott WF 1984/10 -Landslide susceptibility of natural slopes in the City of Brisbane
Willmott WF 1984/44 -Slope stability and its constraints on closer settlement in the foothills of the Toowoomba Range, Gatton Shire
Willmott WF 1987/04 -Slope stability and its constraints of building in the Rosewood-Marburg area
Willmott WF 2001/01 -Slope stability and its constraints on closer settlement – Springbrook Plateau and Upper Tallebudgera and Currambin Valleys, SE Qld
Willmott, WF Suwitadiredja, DA; 2003/03 -Slope stability and its constraints on closer settlement – Mt Mee Plateau, SE Qld

Unfortunately, AS 2870-1996 doesn’t offer any specific guidelines. The author believes that the Code should at least make it clear that in classifying a site the possibility of slope instability must be considered and that the engineer be made aware of such areas and the stability of proposed or existing excavations also be considered. This could simply be
done by including a statement in AS 2870 similar to “Where slope stability maps of an area are within the public domain, the slope stability zoning of a specific site must be referred to on the site classification report.”

Gold Coast City Council has specified that any new allotment which even partly falls within the “moderate” soil instability hazard rating on their maps, must have a slope stability report carried out.

On Tamborine Mountain, the Beaudesert Shire Council normally requires a report on allotments which plot in a “B2” or more severe category on the 1981 Willmott map.

SMEC (1999) in their report to the Gold Coast City Council offered the following conversion from Willmott’s zones to their own classifications as follows:

<table>
<thead>
<tr>
<th>DME Slope Instability Zoning</th>
<th>Hazard Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Af, A1</td>
<td>Very Low</td>
</tr>
<tr>
<td>A2, B1, B2</td>
<td>Low</td>
</tr>
<tr>
<td>C1, D1</td>
<td>Moderate</td>
</tr>
<tr>
<td>C2, D2</td>
<td>High</td>
</tr>
<tr>
<td>D3, E</td>
<td>Very High</td>
</tr>
</tbody>
</table>

9  CLAUSE 2.3.3 - DEPTH OF INVESTIGATION AND CLAUSE 2.1.3 - EFFECT OF SITE WORKS ON CLASSIFICATION

Clause 2.3.3 “Depth of Investigation” was a new clause in AS 2870-1996 and states;

“The soil profile shall be examined to a minimum depth equal to 0.75 times the depth of the suction change, Hs as given in Table 2.4, but not less than 1.5 m, unless rock is encountered or in the opinion of the classifier, further drilling is unnecessary for the purpose of identifying the soil profile in accordance with Clause 2.2.1(a).”

Clause 2.4.5 first appeared as Clause 2.1.3 in AS 2870.1-1988 and states;

“The classification of a site shall take into account the effect of site works when these are known at the time of classification. Where the effect of site works is not taken into account, the classification shall be reconsidered if-

a) the depth of cut on an S, M, H or E site exceeds 0.5 m; or
b) the depth of fill exceeds the limits provided in Clause 2.4.6

These clauses are closely related and sometimes their implications are overlooked, resulting in inadequate onsite drilling and the under-classification of some sites. Site drilling for site classification purposes in SE Queensland pre-1996 averaged between 1.2 and 1.5 metres deep but more recently the average has increased to 2.0 to 2.5 metres (when reading a range of site classification reports from various consultants). In general, Clause 2.3.3 implies that the total drill depth is related to Hs, therefore unless rock is encountered, the following Table 5 is applicable;

<table>
<thead>
<tr>
<th>Hs</th>
<th>Minimum Drill Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5 m</td>
<td>1.5 m</td>
</tr>
<tr>
<td>1.8 m</td>
<td>1.5 m</td>
</tr>
<tr>
<td>2.3 m</td>
<td>1.7 m</td>
</tr>
<tr>
<td>3.0 m</td>
<td>2.2 m</td>
</tr>
</tbody>
</table>

The author believes that the intent of this clause is to explore to the depths specified in Table 5 below the finished platform level, which is why Clause 2.4.5 requires the site classification to be reconsidered if the cut is to exceed 500 mm. Therefore in the previous example (where a 1.2 m site cut is proposed) the following minimum drill depths should apply (assuming rock isn’t encountered at shallower depths);

<table>
<thead>
<tr>
<th>Hs</th>
<th>Proposed Cut Depth</th>
<th>Minimum Drill Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5 m</td>
<td>1.2 m</td>
<td>2.7 m</td>
</tr>
<tr>
<td>1.8 m</td>
<td>1.2 m</td>
<td>2.7 m</td>
</tr>
<tr>
<td>2.3 m</td>
<td>1.2 m</td>
<td>2.9 m</td>
</tr>
<tr>
<td>3.0 m</td>
<td>1.2 m</td>
<td>3.4 m</td>
</tr>
</tbody>
</table>
There has been some argument that the above quoted “Minimum Drill Depth” could be reduced by 500 mm, because of the “500 mm provision” in Clause 2.4.5 of AS2870-1996, but a case could also be mounted that the borehole on the lower side of an allotment should be extended to find the same strata in which the borehole terminated on the high side.

Although there are many soil profiles in SE Queensland where the reactivity reduces with increasing depth and bedrock is approached or encountered, there are also several areas where the reactivity levels of the soil increases with depth and the site cut can expose this strata. This possibility must be considered, and drilling to adequate depths will ensure this strata is discovered.

Clause 2.4.5 also requires the fill depths and nature of the fill to be considered. Section 5 of this paper refers to this and the designer must decide whether settlement or heave will be a factor, depending on the nature of the fill and whether it has been compacted or not.

10 CONCLUSIONS

As can be seen, a simple 1986 document has evolved into a complex standard and in this paper only the geotechnical issues have been considered. The design implications have also become more complex.

Most forensic investigations in SE Queensland do calculations based on the “computation” method. The author has on his records over 300 site classification reports from eleven different consultants dated between August 1999 and March 2001 and only one of these consultants carried out a shrink/swell test. On a volume basis, these eleven consultants would account for well in excess of 80% of all site classifications done for project builders and individual owners, therefore it is not unreasonable to assume that the computation method of determining a $y_s$ (and hence site classification) has been the least preferred of the methods of determining a site classification in S-E Queensland.

Further support for not using an individual shrink/swell test to reach a $y_s$ (and site classification) was found in the following publications;

AS 2870-1996 Note 2 on Page 17 states (in part);

“Estimations based on a single test result to describe a full soil profile may be misleading.”

SAA, HB28-1997, page 21 states;

“Nonetheless, the use of suction profiles and instability index values is the most accurate method of calculation available (but not necessarily the most accurate method of classification).”

These statements are an oversimplification of the following quote by Prof Karl Von Terzaghi (1883 – 1963) who is remembered as the father of soil mechanics;

“Unfortunately, soils are made by nature and not by man, and the products of nature are always complex...As soon as we pass from steel and concrete to earth, the omnipotence of theory ceases to exist. Natural soil is never uniform. Its properties change from point to point while our knowledge of its properties are limited to those few spots at which the samples have been collected. In soil mechanics the accuracy of computed results never exceeds that of a crude estimate, and the principal function of theory consists in teaching us what and how to observe in the field.”

This statement by Prof. Karl Von Terzaghi adds support to some consultants’ belief that they would rather trust their field observations, experience, soil borelogs and geological maps than a core of soil 50 mm in diameter, 50 mm long and a computer to classify a site.

- The changes to the cracked zone and $H_s$ depths in AS 2870-1996 resulted in a dramatic change in $y_s$ values. These changes shouldn’t infer that the pre-1996 classifications were incorrect, rather that these soil conditions were reinterpreted in line with advances in scientific knowledge.

- There is some uncertainty about the boundaries between the 1.5 m, 1.8 m and 2.3 m $H_s$ zones in SE Queensland, but I hope that the forthcoming revision to AS 2870 will provide clearer guidelines and in the interim, the relevant published papers should be consulted.

- The abnormal moisture conditions provisions of AS 2870-1996 have not been widely embraced, but in more recent times the uptake has been more widespread, possibly driven by the Sept 2004 changes to the BSA subsidence policy (not discussed here). It has been the writer’s experience that the majority of problems with dwellings on clay based sites are related at least in part to the last four conditions of Clause 1.3.3 of AS 2870-1996, all of which are beyond the control of the site classifier and design engineer. In Queensland, the BSA is now trying to address this by a public education program. It is disappointing also that the new CSIRO sheet BTF 18 has placed less importance on trees than in the past.
• The reclassification of filled sites, although clearly explained in AS 2870-1996 appears to be poorly understood by a significant number of consultants.

• The interpretation of classification of the canal estate sites is far from reaching consensus, and there appears to be an ignorance of the geotechnical processes at work on these sites by some consultants.

• Although an issue pre-1990, mining subsidence (although still of great importance) is not in the forefront of concern since the local authorities have been more proactive in this area.

• Slope stability is largely ignored by site classifiers, even though there is a wealth of easily accessible information available.

• Often, onsite drilling is done to a set formula (possibly due to a flat fee structure to a client) rather than to provide the engineer with the necessary information, although the guidelines in AS 2870-1996 are clearly defined. Many allotments are classified in the “as tested” state even when the proposed earthworks are known, which leads to both over and under classified sites.

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Fox, E; (Australian Geomechanics 2002) “Development of a Map of Thornthwaite Moisture Index Isopleths for Queensland”
McManus, Dr K; Lopes, D; Osman, N Y; (2004) “The Effect of Thornthwaite Moisture Index Changes on Ground Movement Predictions in Australian Soils”
12 THORNTHWAITE MOISTURE INDEX MAPS

Figure 1: Climate zones SE Queensland McManus, Lopes & Osman, 2004.
Figure 2: Climate zone map of Queensland – E Fox (Australian Geomechanics 2000).
Figure 3: TMI map of Queensland – E Fox (Australian Geomechanics 2002).