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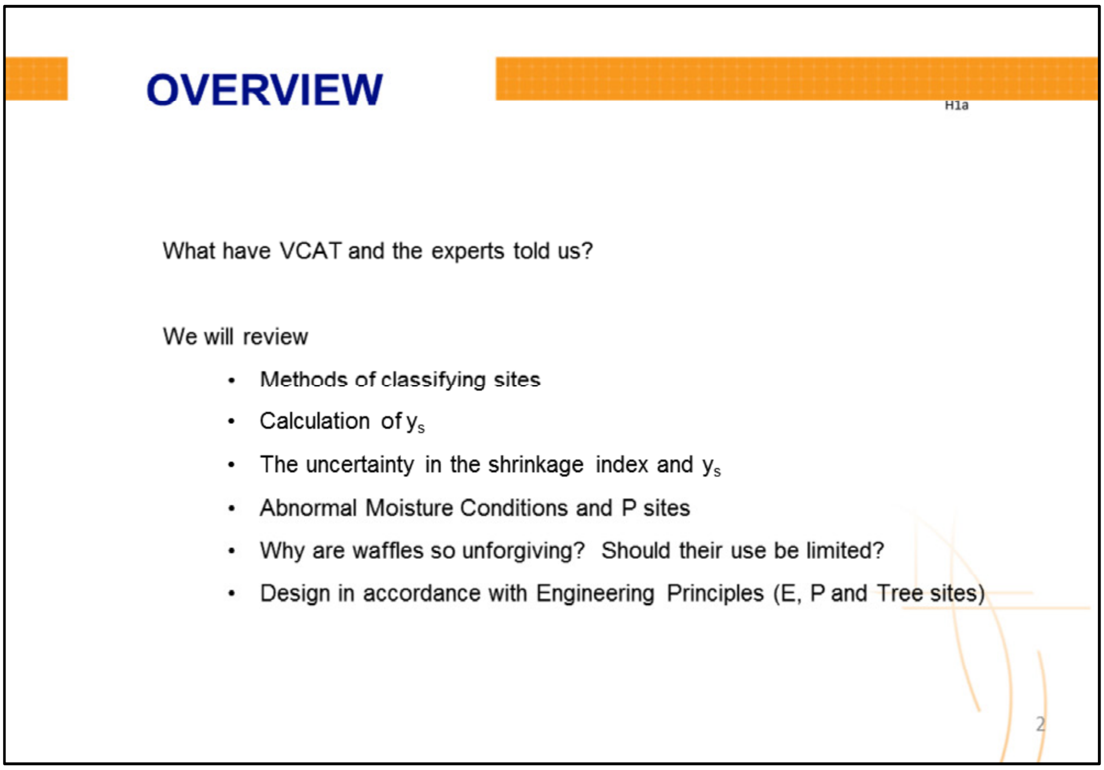
SRIA
Steel Reinforcement Institute of Australia

Compliance with AS2870-2011 – Recent Failure Investigations and Design By Engineering Principles
(Rev 2)

PRESENTED BY PETER BAYETTO
Tuesday 12 July 2016



Discuss:

I will review these issues in the context of AS2870 and the outcomes of the court cases. Following the requirements of AS2870-2011 is sufficient to avoid the issues that have occurred.

BACKGROUND

JUNE 8, 2014

INDEPENDENT. ALWAYS.

$2.80

sundayage

MY PLASTIC FAMILY
One woman's kooky stance
SUNDAY LIFE

ROUGH AND RAW
Greg Baum on the Socceroos
SPORT

TOUGH GUY
Guy Pearce's vanishing act
M MAGAZINE

Sinkhole suburbs: thousands face ruin

EXCLUSIVE

Simon Johanson

Thousands of near-new homes are cracking up in Melbourne's western and northern suburbs, leaving their owners facing financial ruin and long battles to fix them.

Estimates suggest up to 4300 homes in Wyndham, Melton and Hume local government areas may be suffering from "slab heave" where volatile soil movements under a home's foundations cause walls to crack, doors and windows to jam, and floors to tilt.

But the actual number may be far higher, said former academic and consulting engineer Peter Yttrup. Hundreds of other homes in Melton West have been built on a "sinkhole plain".

A report revealed in a legal case this week shows Melton Council knew about extreme soil conditions in the new housing estate but did nothing to inform builders or potential home owners.

Melton Council would not comment because of the legal action.

Concerns about slab heave have prompted 160 Melbourne home owners to contact Slater & Gordon Lawyers, solicitor Robert Auricchio said.

Others are taking individual legal action. If they are successful, the state government, which from January this year underwrites all builders warranty insurance, may ultimately foot the bill.

New estates from Grovedale on

Continued page 8

BACKGROUND

8 NEWS — JUNE 8, 2014 SUNDAY AGE

Thousands of home owners facing ruin

From page 1

Geelong's outskirts to Doreen in Melbourne's north are also affected in an epidemic being blamed on "waffle slab" foundations and poor classification of highly reactive volcanic clay soils.

Waffle slabs "float" on top of compacted ground and are cheaper and quicker to build than traditional footings.

Grovedale home owner Jamie Crouch is living with his young family on a "dam of sewer" that is collecting under his seven-year-old house, causing its slab to heave.

"We noticed some internal cracking and we patched it on numerous occasions. It just kept opening up. It's impacting on our personal and financial life," he said.

Internal walls were lifting off the floor, objects roll off the benchtops, and windows and doors won't shut in the Bailey Street home.

Mr Crouch blamed poor drainage and broken sewer pipes left by the builders for triggering the problem.

Both the previous Labor government and current Planning Minister Matthew Guy have sped up land release in Melbourne's fast-growing outer suburbs, many in areas with volatile soils, to encourage affordable new housing.

Between 2008 and 2011, 82,798 building permits were issued in Wyndham, Melton and Hume, the Victorian Building Authority said. A VBA investigation of 625 homes found 5.3 per cent had faults.

The problem of slab heave is "systemic", widespread and needs urgent government action to fix, Mr Yttrup said.

He said waffle slabs, poorly trained and regulated soil testers, and builders failing to follow the AS2870 standard for home footings were at the root of the problem.

Others say the standards themselves are not tough enough.

Melton West home owner Annette Watson this week took builder Cavalier Homes North Western to Victoria's planning tribunal, seeking at least $30,000 to fix problems with her home caused by soil movement under a waffle slab. Ms Watson said her difficulties began six years ago when cracks appeared in the walls and cornices dropped off as her new $225,000 house moved.

Now, the front bedroom is 71 millimetres higher than the rear of the home. Many neighbours are suffering similar issues, she said.

Barrister for Cavalier Homes Robert Squirrell told the Victorian Civil and Administrative Tribunal up to 80 other houses built by Cavalier could be affected.

"This is but one of many, this is the thin edge of the wedge," he said.

Mr Squirrell sought to have Melton Council, the soil tester and engineer joined to the case because a report given to the council, which labelled the area "sinkhole plain", showed thick expansive clay soil posed a hazard to development.

Earlier this year, a home owner in Melbourne's newest suburb, Tarneit, won $289,589 for the cost of replacing his home in Hallows Circuit. Metricon, Australia's largest new home builder, has since appealed the decision but faces a similar case in September.

Tribunal member Margaret Lothian said home owners are facing "little costs" – where legal fees outweigh recompense – when they seek to fix their problems in court.

Do you know more?
sjohanson@fairfaxmedia.com.au

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BACKGROUND

Financial Review, 7 April 2016

[redacted] slapped over slab

Michael Bleby

[redacted] Australia's fifth-largest home builder, failed in its final bid to overturn a ruling that it must demolish and rebuild a substandard home in Melbourne's west.

The Victorian Court of Appeal on Wednesday dismissed an appeal by Mount Waverley-based [redacted] against a December 2014 order to rebuild the Melton West house of Earl and Shelley Softley, which suffered from so-called slab heave – structural faults due to footing and foundation movements.

The judgment upholds an order that will cost [redacted] about $269,000.

It is the company's second loss in a year against lower-court rulings over faulty slabs.

Last March the Supreme Court upheld an order for damages worth over $280,000 to the owner of a house in Tarneit, also on Melbourne's west, that was defective because of inadequately compacted soil.

The company said the judgment set no precedent.

"We are firmly of the view that this case is unique and the technical aspects being contested are not applicable to other [redacted]," chief executive [redacted] said in a statement.

Slater and Gordon associate Robert Auricchio, who acted for the Softleys, said the case set a precedent in any state where houses were built on "reactive soil" – soil likely to expand during adverse conditions such as rain.

"It reinforces the importance of builders to ensure that proper drainage is taken care of in the course of construction," Mr Auricchio said.

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These are the most recent headlines.

CAUSATION / CLAIM AGAINST THE ENGINEER

I am satisfied that the slab was not constructed as designed. It was built upon an inadequate foundation. I find that the Second Plan was adequate, notwithstanding that it was not the result of any contemporaneous calculations. Consequently although I have found various breaches by the Engineer of its duty of care to the Owner there is no proof of actual damage arising from those breaches. The responsibility for the foundation failure lies wholly upon the Builder. (210)

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CLAIM AGAINST THE ENGINEER

- Failed to take reasonable steps to calculate the Y_s of the site
- Significantly underestimated y value (<70 against 85^+ in evidence)
- Did not design the slab using calculations per AS2870
- Did not perform computations or further geotechnical investigations upon receiving the compaction data
- Failed to take the reactivity of the site into account in designing the slab
- Incorrectly applied a maximum allowable deflection of 40mm for the inside
- Specification of "rolled fill" instead of "controlled fill" for edge beams "was negligent" BUT
- there is no evidence that the Engineer was retained by the builder to design the site drainage

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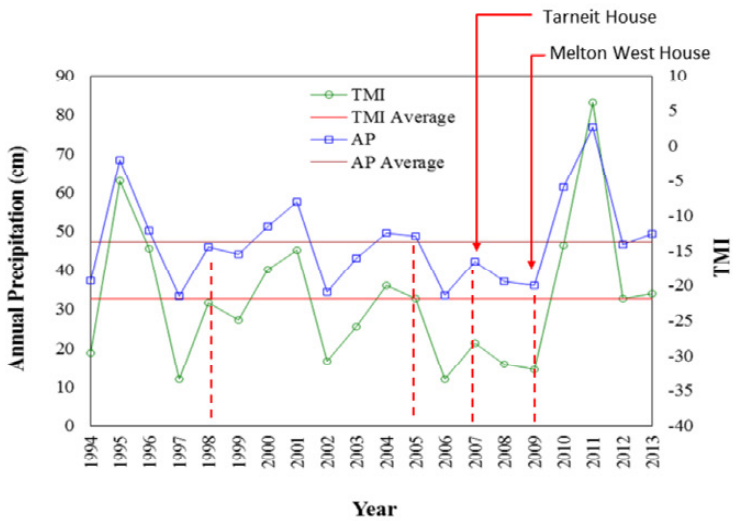
BACKGROUND



Variation in Annual Precipitation and *TMI* (Laverton, 1994-2013)

Ref Dr Jie Li, RMIT

Hollows Circuit was subdivided from treeless farmland in 2005. The whole subdivision was regraded with reactive clay controlled fill (subdivisional fill).

BACKGROUND

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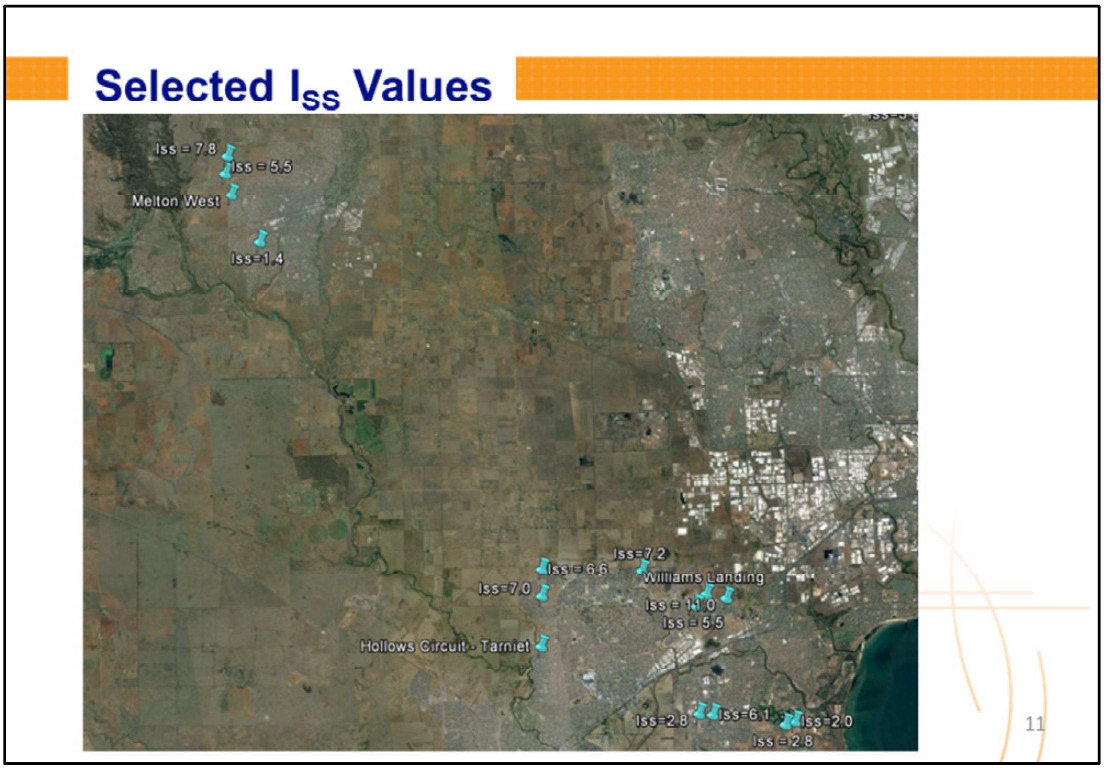
The cracks in the road are an indicator of the extreme reactivity of the soil.

This house is on a 385mm deep waffle with no allowance for the effect of the street trees.



Discuss:

I have put up some key shrink swell values that demonstrate the distribution of test results for the West.

Tarneit and Melton West were the two VCAT decisions appealed to the Supreme Court. I have highlighted Williams Landing because later we'll calculate the Y value for that site.

You will note that the higher values are clustered in the western suburb development areas within the shires of Wyndham, Melton and Hume. Hoppers Crossing and Tarneit are suburbs which have had particular problems, having the highest reactivity soils which were not recognised for what they were at the beginning of development.

BACKGROUND

DISTRIBUION OF I_{SS} VALUES OVER GREATER MELBOURNE METROPOLITAN AREA

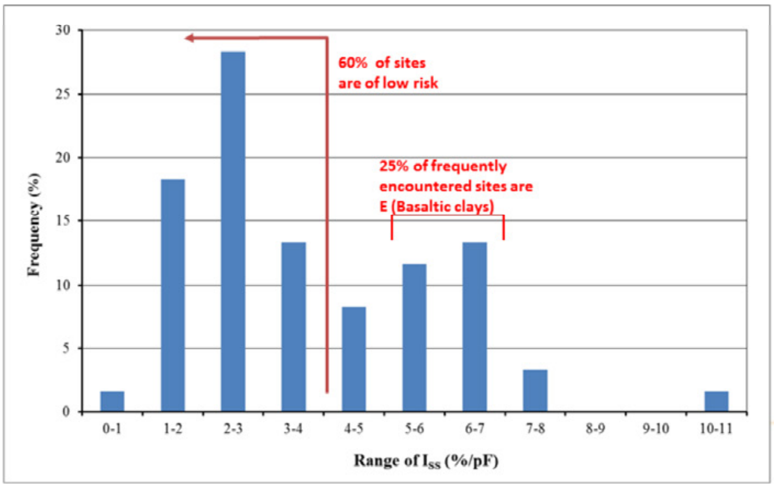


Figure 9: Histogram of all shrink-swell indices

REFERENCE: SHRINK-SWELL INDEX DATABASE FOR MELBOURNE - Dr Jie Li, Jian Zou, Peter Bayetto and Nick Barker

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NOTE: This is the distribution of I_{ss} values across the greater Melbourne area. This is for the sites we tested distributed across 37 suburbs.

AS2870-2011

2.2 METHODS FOR SITE CLASSIFICATION

2.2.2 Identification of the soil profile
(This is no longer stands up to the vigours of VCAT and the Supreme Court)

2.2.3 Site classification based on characteristic surface movement
Logging of soils and calculation of y_s in accordance with Clause 2.3 of the Standard including bench mark shrinkage index (I_{ps}) testing.

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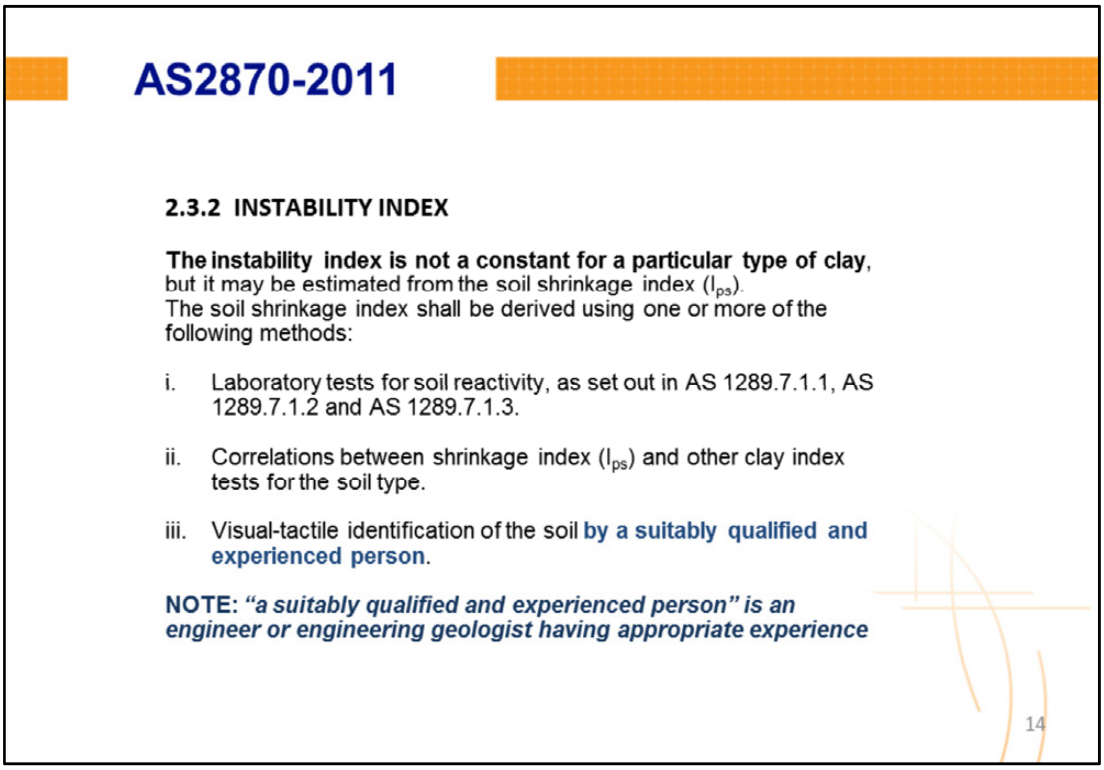
Discuss:

How should we classify sites?

The Engineer cannot adequately determine the level of risk associated with the sites potential response to construction changes without detail of the soil profile and where the reactivity is. You also need to account for the effects of sub-divisional fill and any future cut and fill proposed.



Discuss:

The visual tactile procedure is the most time and cost effective estimation process when one accounts for the commercial imperatives. It is almost universally used.

Section 2.3.2 I refers to:

7.1.1 Shrink Swell
7.1.2 Drying Shrinkage
7.1.3 Core Shrinkage

AS2870-2011

INSTABILITY INDEX

The visual-tactile procedure is the dominant logging process used:

> ***"For method (iii) above, the suitably qualified and experienced person shall check the soil property identification against laboratory testing on reactive soils at a period not longer than six months and at least once in every 50 sites personally classified."***

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Discuss:

AS 2870-2011 requires that the visual-tactile procedure is routinely calibrated or bench marked against laboratory testing.

Depending upon the geographic spread of the work being undertaken, 1 in 20 may be a more suitable frequency. The decision is about choosing the level of risk one will operate at.

How can a site classifier's reports be QA/QC'd without this information and demonstration of compliance?

CALIBRATING THE VISUAL TACTILE PROCEDURE

FIVE SITES CHOSEN IN SA

- Current development areas
- Geographic spread over greater Metropolitan Area
- Representative of a range of surface soil types and geological settings

They were logged "blind" by 5 major engineering/logging companies.

Soil samples tested (shrink swell, core shrinkage and atterberg limits)

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Discuss:

We should consider the uncertainty in the logging process and assessment of shrinkage index and hence the consequent calculation of y_s

We need careful logging of the soil layers and assigning of the instability index to allow Engineers in the design supply chain to assess the level of risk.

Last year a comparative logging exercise (an industry snapshot or audit if you will) was carried out by the EA Footings Group in SA.

This was combined with a logging course teaching program.

The sites were chosen based upon the above

CALIBRATING THE VISUAL TACTILE PROCEDURE

- Significant uncertainty in the logging process (and testing?)
- Uncertainty possibly varies with soil type
- Average $\frac{\text{Est } I_{ps}}{I_{ss}}$ (all sites, all layers tested, all loggers) = 120%
- For all sites, the variation in logging bridged two site classifications (eg S and M, M and H or H and E)
- On average estimates are precautionary (but can vary wildly)

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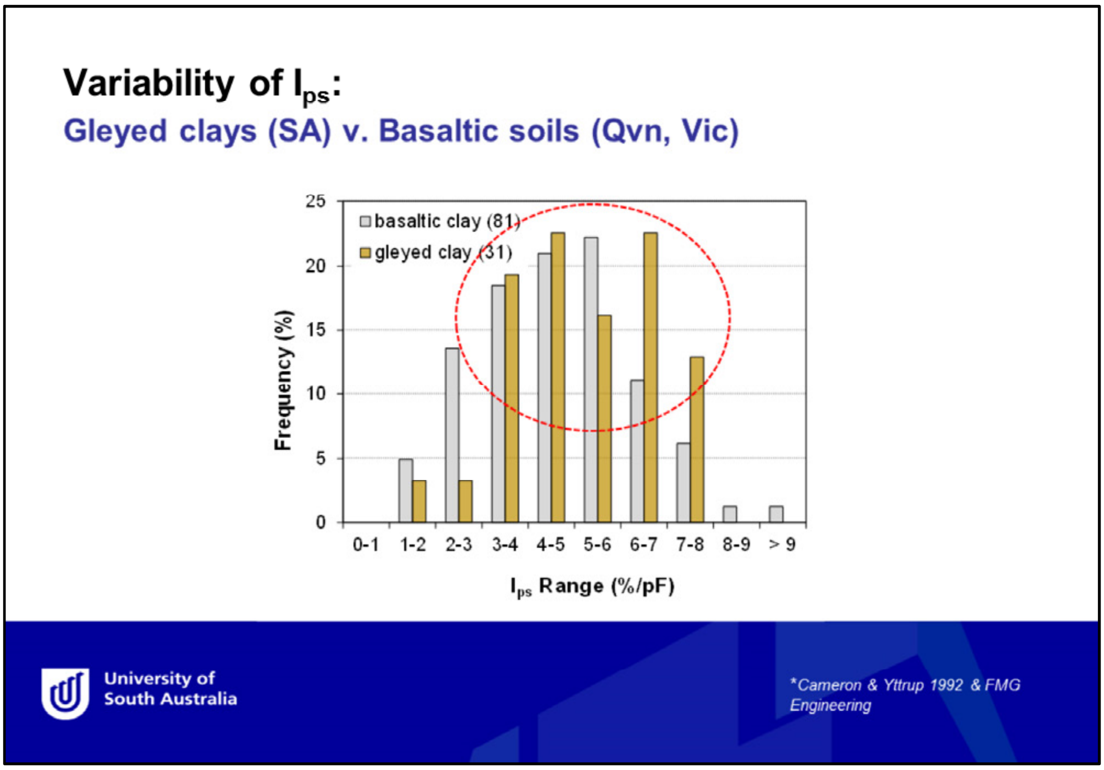
Discuss:

NOTE: that + 20% is comparable to the uncertainty of measurement, so you want it that way. 20% is significant but not unacceptable in geotechnical testing terms. The uncertainty of measurement is common to all soil testing.

In aggregate Engineers were logging conservatively. Individual loggers varied from site to site and from day to day but on average they were not systematically inconsistent.

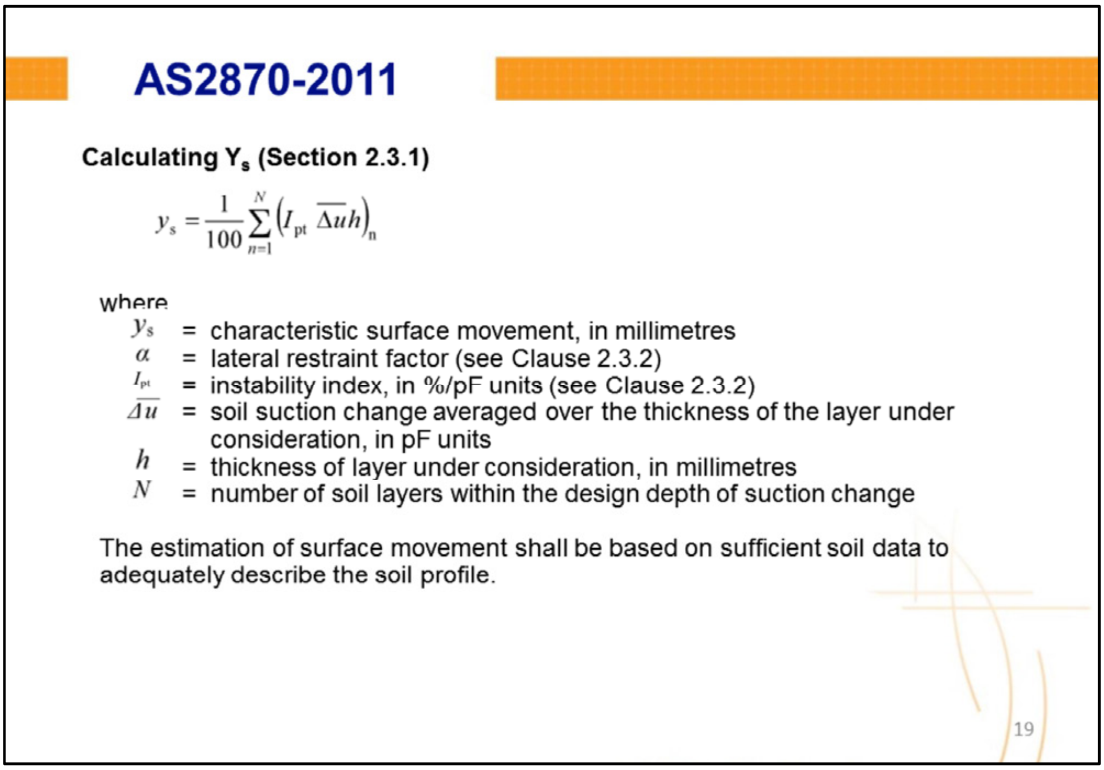


This gives an idea of the distribution of Soil Shrinkage Index for the basaltic clays.

So you have a basaltic clay. Well what I_{ps} would you like to attribute to it?

Melbourne's Quaternary basaltic soil shown in the same plot is consistently recorded as having a shrinkage index in the range of 3 to 6%/log(kPa), a little lower than that of the gleyed clays of Adelaide.

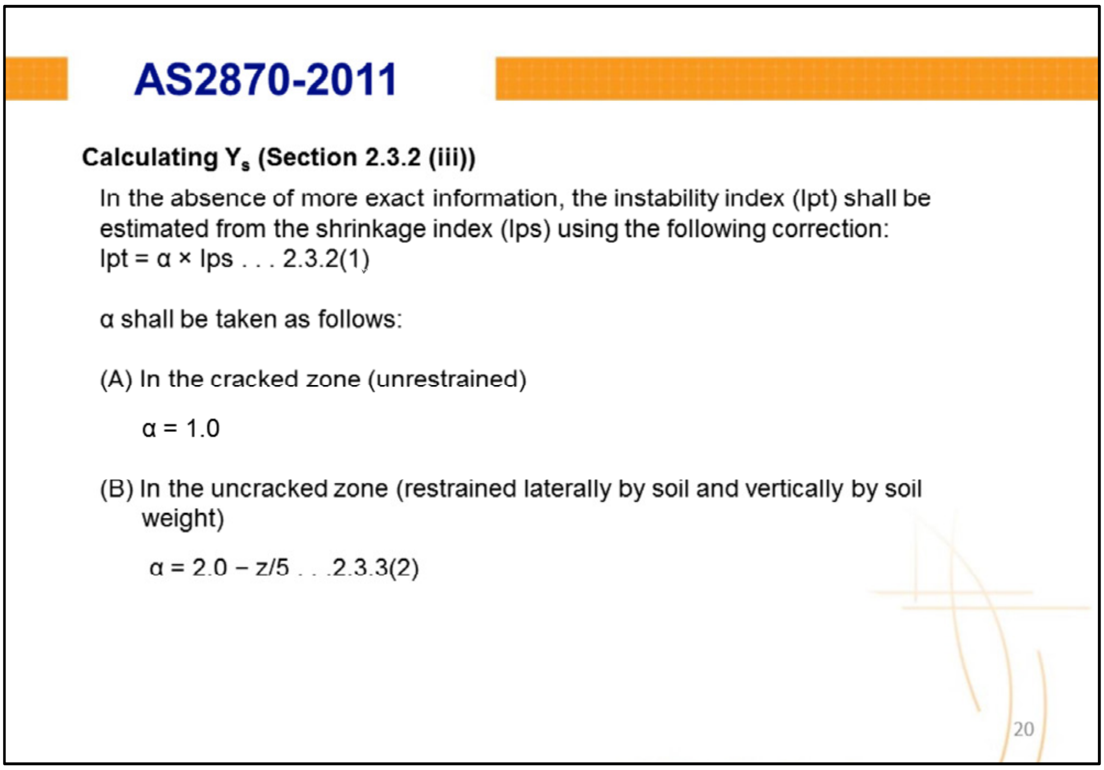
For those of you who are up to date with the latest lithology nomenclature you could read Neo instead of Qvn



AS2870 Section 2.3.1

I am not going to teach you to calculate Y_s. But I am going to demonstrate that if the Standard was being applied diligently, y values of 85 to 130mm would be common in these Basaltic Clay areas.

$I_{pt} = \alpha \times Ips$ . . . 2.3.2(1)



AS2870 Section 2.3.2 (iii)

Alpha varies from around 1.6 to 1.7 for a virgin site and only adds 2 to 10mm to ys (because it is applied below a depth of $0.75H_s$). But for controlled fill it's a different story.

AS2870-2011

Calculating Y_s

where z = the depth from the finished ground level, in metres, to the centroid of the area defined by the suction change profile and the thickness of the soil layer under consideration in the uncracked zone.

In the absence of more exact information, the depth of the cracked zone shall be taken as —

(1) 0.5Hs to Hs where Hs is as given in Table 2.4.

(2) 0.75Hs in Adelaide and Melbourne; and

(3) 0.5Hs in other areas.

For reactive clay in controlled fill placed less than 5 years prior to building construction, the depth of the cracked zone shall be taken as zero. Where a site has been cut less than two years prior to building construction, the depth of the cracked zone shall be reduced by the depth of the cut.

NOTE: The cracked zone relates to the zone in which predominantly vertical shrinkage cracks exist seasonally

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Discuss:

For virgin sites around Melbourne the depth of cracked zone is $0.75H_{s.}$

But for the subdivisional controlled fill alpha is going to vary from 2 at the surface adding significantly to y_s if reactive clays used for the fill.

We need careful logging of the soil layers and assigning of the instability index to allow Engineers in the design supply chain to assess the level of risk.

CALCULATING Y_S

AS2870-2011 Section 2.3.2

REALITY CHECK USING I_{PS}

Typical I_{ps} & Original Engineers' Borelog

HOLLOWS CIRCUIT TARNEIT

Description	Depth (m)	I_{PS} %	Thickness	Y_s (mm)
Subdivisional fill	0 - 0.6	7.0	0.6	
Clayey silt	0.6 - 0.7	4.5	0.1	
Stiff from silty clay	0.7 - 1.5	6.5	0.8	
Presumed to be as above	1.5 - 2.3	6.5	0.38	
			y_s1 =	95
			y_s2 =	135

Surface Suction Change = 1.2pF

Depth of design soil suction change (Hs) = 2.3m

Y_s1 - Crack Zone Depth 1 = 1.7m

Y_s2 - Controlled fill uncracked

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Calculation in accordance with AS2870-2011

Discuss:

Let's go back to Tarneit.

This is a calculation (estimate) of ys using typical bench mark I_{ps} values for the Tarneit area.

The subdivisional fill was less than 5 years old.

If the subdivisional fill layer was assumed to be "uncracked" then you would apply $\alpha = 2.0 - z/5 = 2 - 0.3/5 = 1.9$ to I_{ps}.
This would add around 40mm to first layer => total ys ~ 135!

CALCULATING Y_S

AS2870-2011 Section 2.3.2

Pembridge Ave, WILLIAMS LANDING, VIC				
Description	**Depth (m)**	**I_{PS} %**		**y_s(mm)**
Fill, Silty CLAY: (CH)	0 – 1.8	5.5		
Silty CLAY: high plasticity, brown, blocky	1.8-2.5	4.5		
			y_s1	80
			y_s2	125

Surface Suction Change = 1.2pF

Depth of design soil suction change (Hs) = 2.3m

Y_s1 - Crack Zone Depth 1 = 1.7m

Y_s2 - Controlled fill uncracked

Calculation in accordance with AS2870-2011

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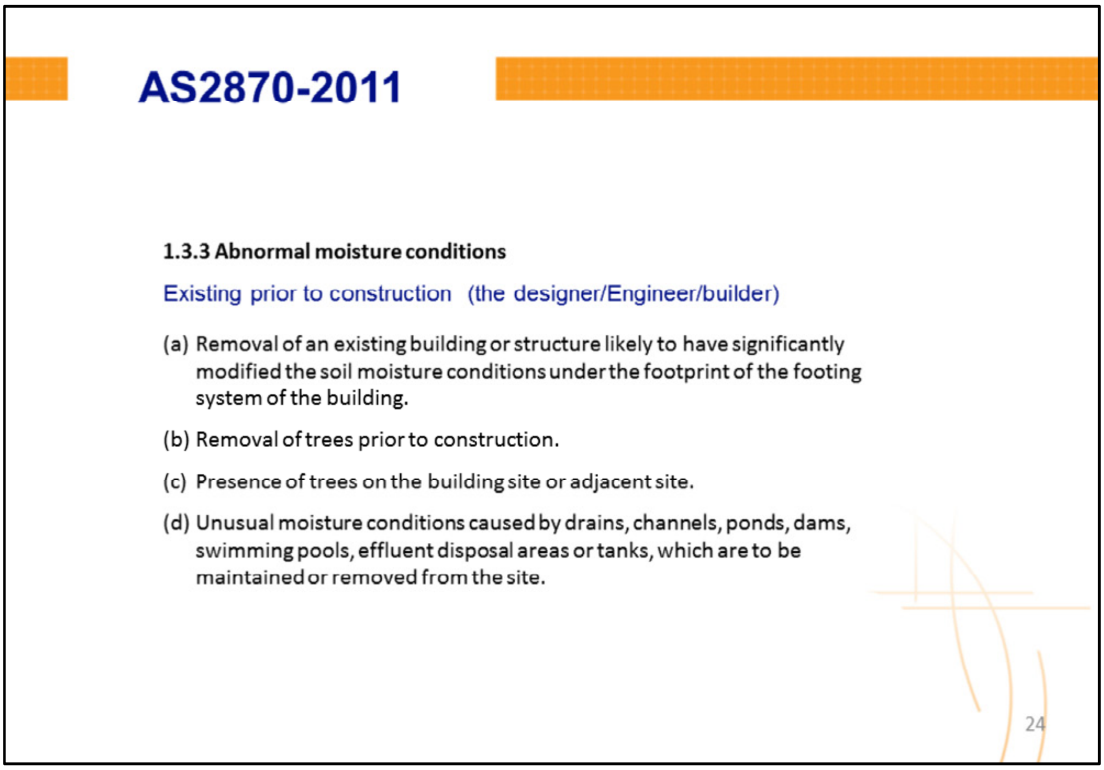
A typical recent borelog again based upon measured I_{ss} values. Whichever way you look at it, the site was E but a substantial footing increase results due to controlled fill.

A 900mm deep conventional raft may be appropriate for these two sites.

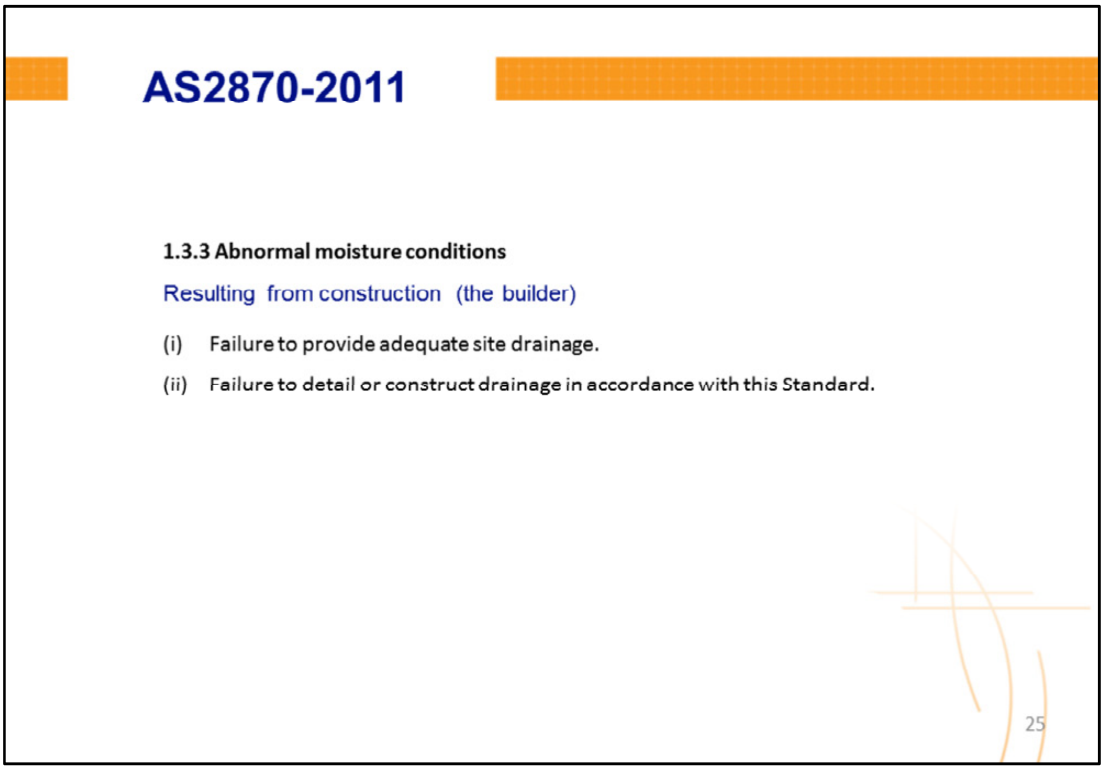


Discuss:

Abnormal moisture conditions mean a P site.

The Standard looks at abnormal moisture conditions prior to, during and after construction. The conditions at investigation and design stage are the responsibility of Engineer / Builder and are to be designed for. There is more advice in Appendix F2 of the Standard.

We must design for the abnormal moisture conditions that exist including street trees that are currently 1 metre high.



Discuss:

Responsibility of the builder.

Notice this gives the builder the option to provide landscaping and drainage plans for the owner to construct.

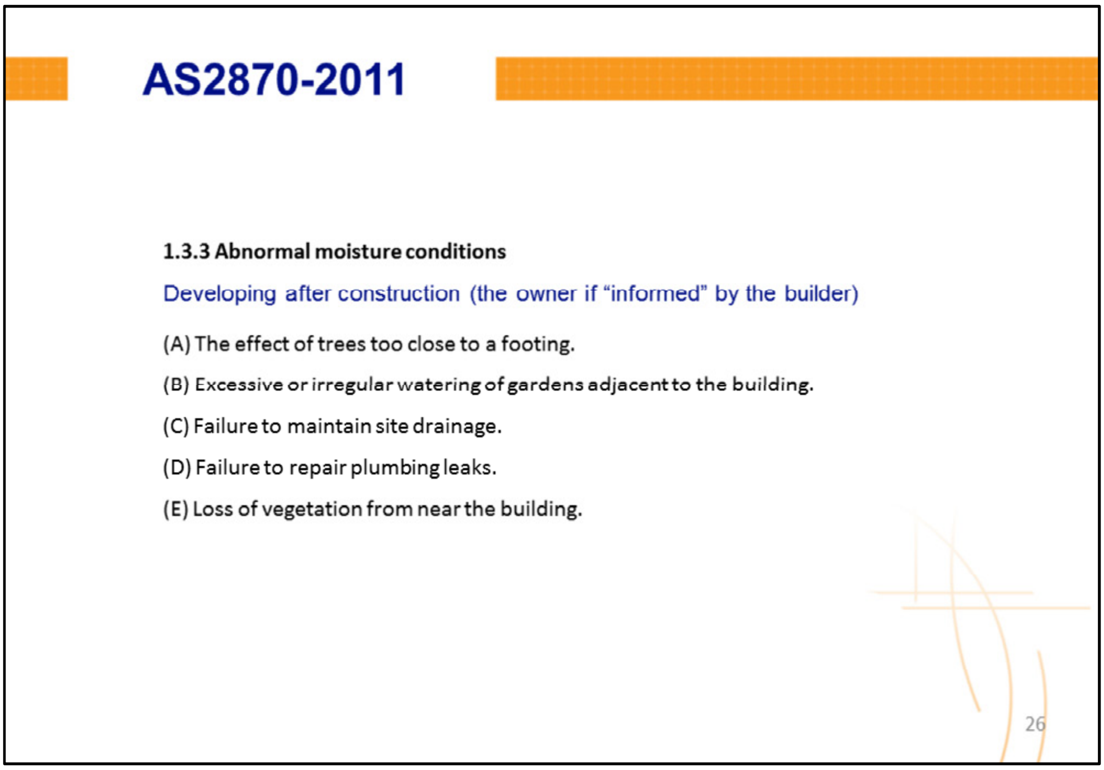
We must provide construction specification that directs the builder how to manage the site construction:

- **Cut and fill plan**
- **Site gradings**
- **Bench levels and floor levels**
- **Temporary drains**

This was why the Tribunal held the builder responsible.

The failed sites all had abnormal moisture conditions triggering the actual failure event:

- some during construction; and
- some post construction



Discuss:

AS2870 says that the site drainage and protection of the footings becomes the owner's responsibility, but only if we give the owner a specification about how to do it for the particular site. (Note: in designing the footings in accordance with AS2870 the Engineer is assuming abnormal site conditions will not be allowed to occur. On what basis?)

Provide the owner (or builder) with plans and specification (a drainage plan)
- **Site grades**
- **Paving protection**
- **Surface drains and sumps**
- **Stormwater disposal**
- **Deal with potential boundary issues on small or zero lot line lots**

The owner contributed, the builder blamed the owner but the Tribunal left the blame with the builder. Why?

…. and found that the Engineer was not commissioned by the builder to advise on drainage so the Engineer carried no responsibility.

AS2870-2011

3.1 (Standard Designs) SELECTION OF FOOTING SYSTEMS

3.1.1 Selection procedure

Standard deemed-to-comply designs shall be in accordance with Clauses 3.2 to 3.6. These designs shall not apply to—

(a) Class E or Class P sites;

(b) and etc

So the engineer needs to design (calculate) the footings for E and P sites (reference VCAT decision)

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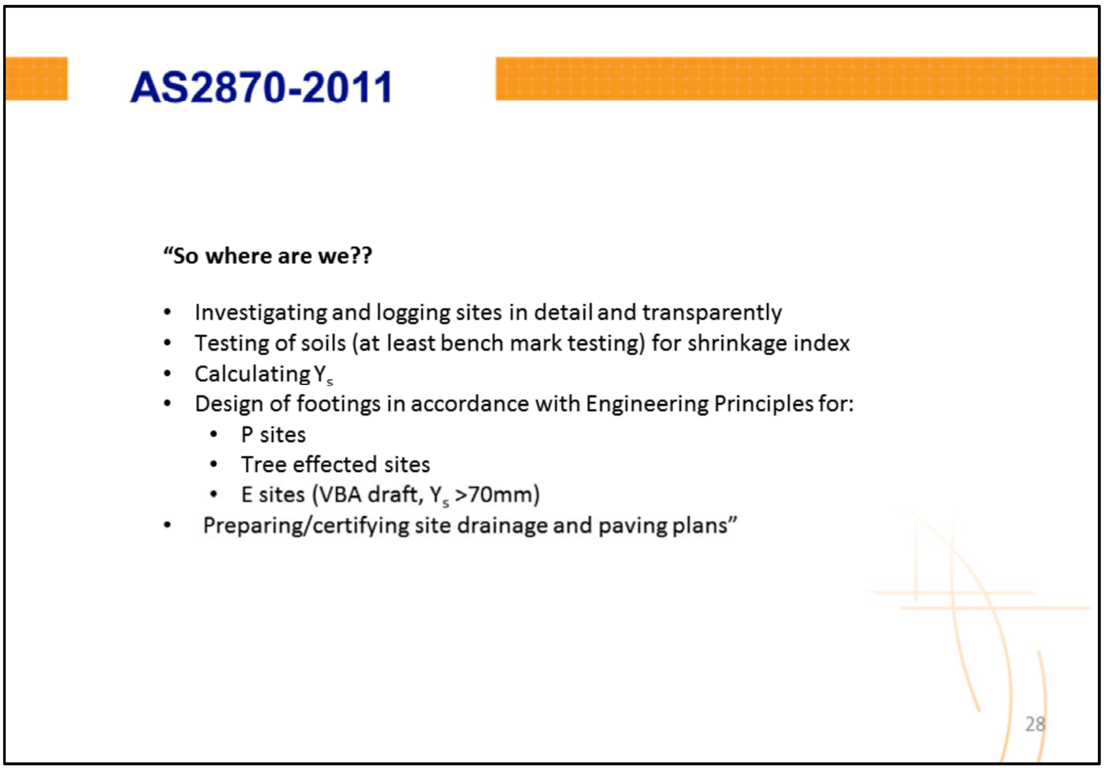
Discuss:

The Standard states that a site with abnormal moisture conditions is a P site.

So its slab design by Engineering Principles for E, P and tree effected sites.

This means we have to do a design in response to the particular site conditions not just go to another lookup table of standard designs.

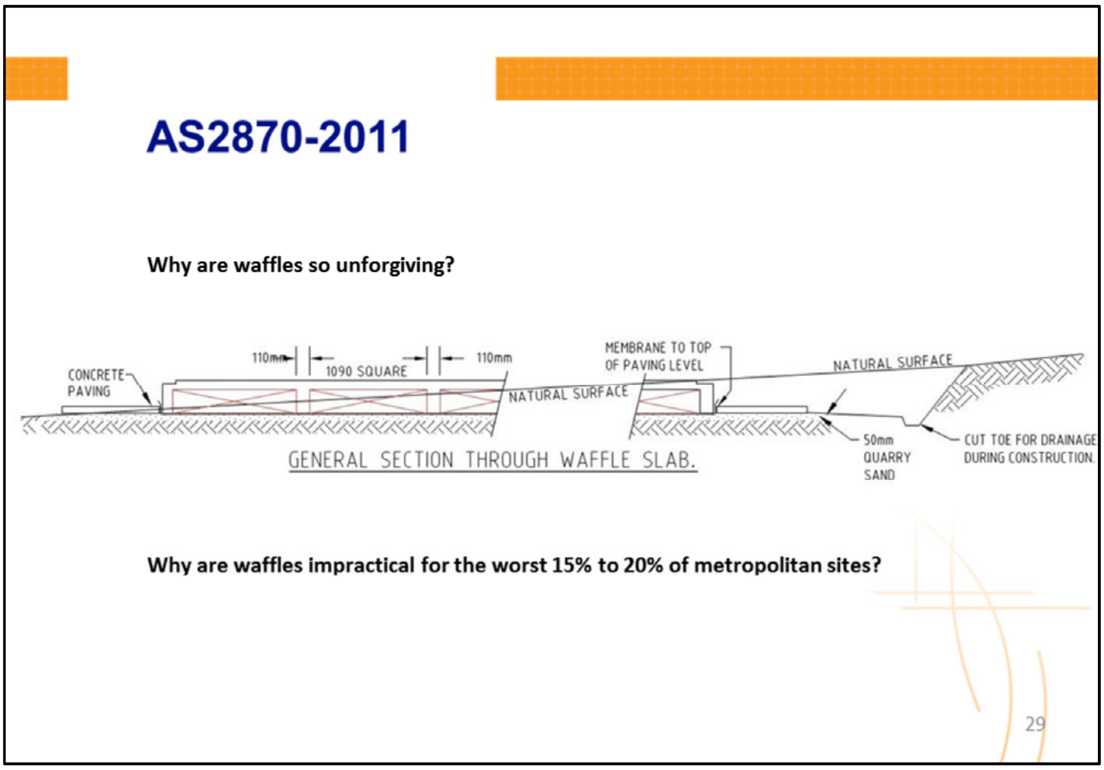
Standard designs are for simple well behaved sites. Engineers are paid to carry out thoughtful analysis and design and model the particular abnormal moisture conditions to be accounted for..



Discuss:

If we are to make the design assumption that we are designing for a "normal site" then site drainage and paving plans the will ensure "normal site drainage" need to be specified.

Before we look at "Design by Engineering Principles" it's instructive to draw some lessons from the outcomes of these failures.



I'll just talk to this cross section which is out of the original 1989 James Hardie brochure.

You can design waffles for E sites but you are talking 600 to 800mm thick slabs. For these slabs, you have the market's expectation that internal floor levels are approximately the same as for the external alfresco areas etc. **If the waffles slab is dug into a hole in the ground, abnormal moisture conditions will be triggered. If the site is filled around the waffle slab, the risk of triggering abnormal moisture conditions is extremely high and cannot be monitored.**

These issues are the reason waffles are not used for H and E sites in some jurisdictions. Not poor performance but rather lack of market acceptability. A 700mm high slab with protective paving around it and steps up to it to give access is not acceptable.

The hearings documented how abnormal moisture conditions were caused on these sites both during and after construction.

There are not negotiable details shown on this cross-section. Firstly, the cut surface needs the natural soil cut sloping away from the house as shown above and it needs to drain. Temporary drains and stormwater connections for construction and protective paving etc for owner occupier.

The waffle is unforgiving:
- If the designer sites the house low because of owner preference or "ResCode" roofline requirements
- If the builder cuts the site low, doesn't grade the cut natural surface away or uses too coarse a rubble under the slab (Ref: VCAT) (NCC Section 3.2.2 (a)(i) "Sand used in controlled or rolled fill must not contain any gravel size material …..") or the site falls towards the house, or downpipes are not connected early enough;
- If the landscaping and paving by the owner falls the wrong way or allows water to pond or there is no edge paving protection for the slab.

AS2870-2011

Design by Engineering Principles

Design in Accordance with Engineering Principles

Section 4.1 General

Section 4.2 Design Criteria

Section 4.3 Design of Footing Systems

Section 4.4 Stiffened Raft Systems

Section 4.6 Design of Footing Systems Other Than Stiffened Rafts

Appendix F: Soil Structure Interaction Analysis for Stiffened Rafts

Appendix H: Guide to Design of Footings for Trees

AS2870 discusses the interactive soil mound/structural model in terms of the “Walsh Method” and “Mitchell Method” in Appendix F. These are generally represented by the commercially available software programs CORD and SLOG.

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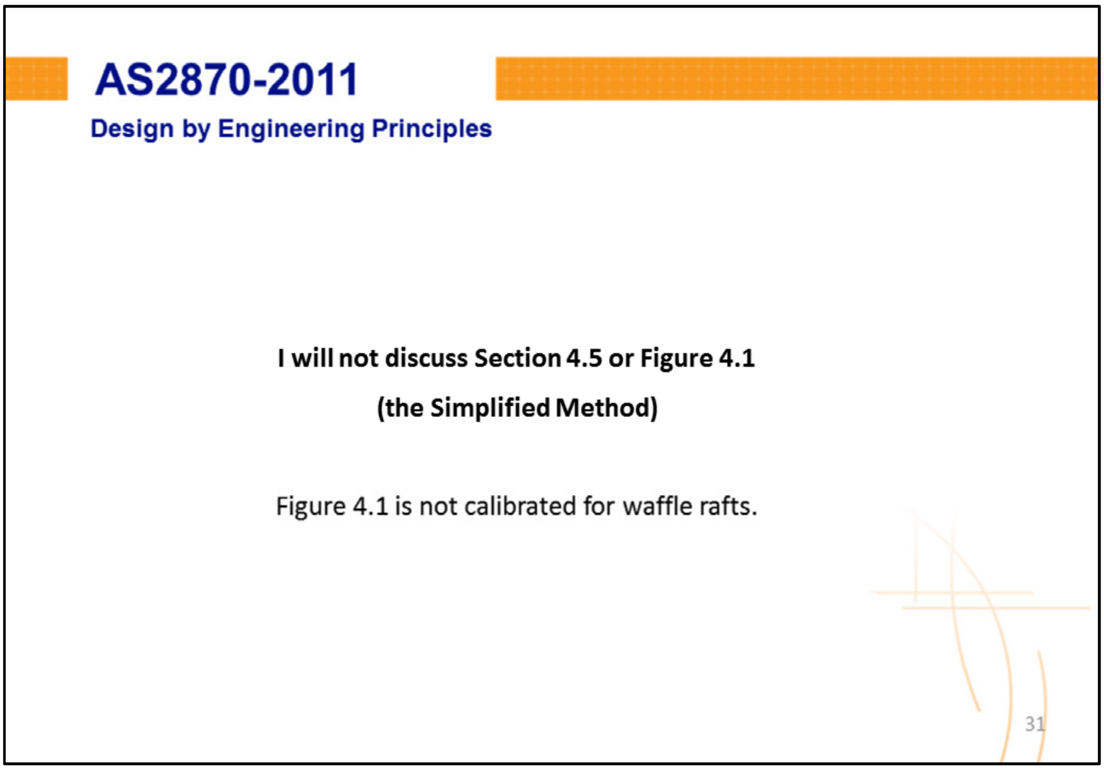
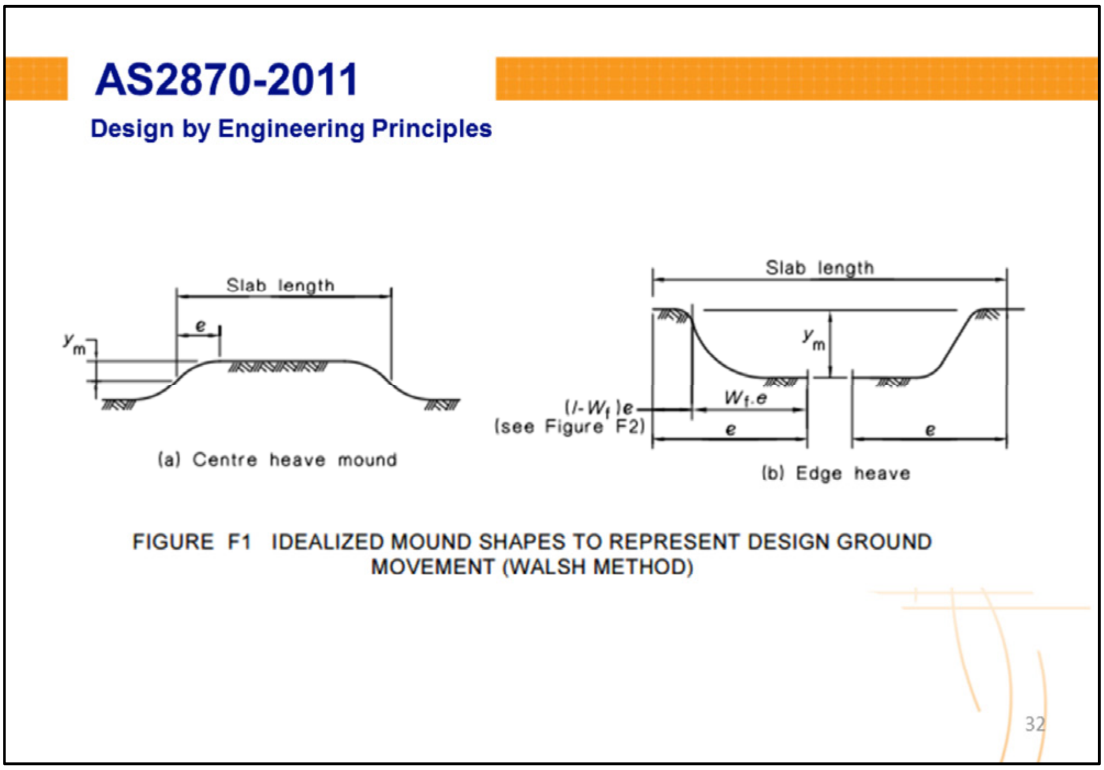
Discuss:



Discuss:

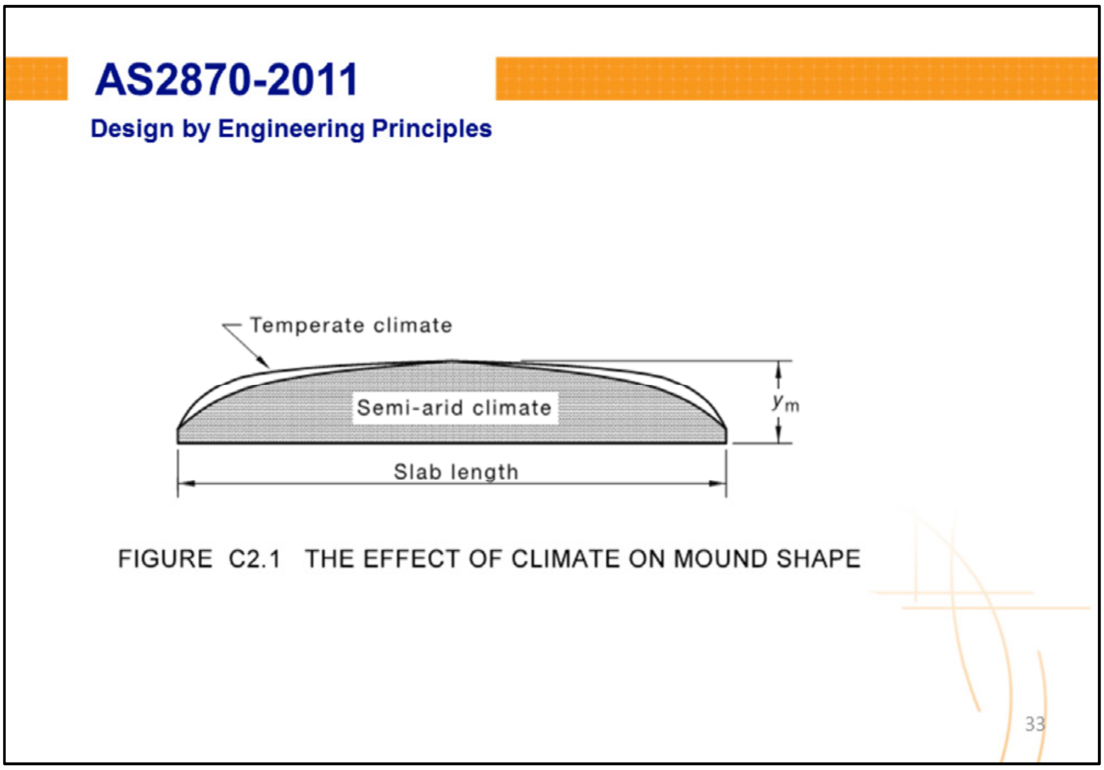
There is a firm belief that the structural analysis for E & P sites needs to be more thoughtful and site customized design specific to the abnormal conditions being modelled rather than simply going to anther look up table. We will be operating at different risk levels depending upon the borelogs, the past history of the site (controlled fill, trees removed) and how he site is to be used.



Discuss:

What are we trying to do?

The design of a slab to accommodate ground movements requires the provision of sufficient overall strength and stiffness. Whereas a very flexible slab could deform in the same way as the foundation, the stiffness of a properly designed slab limits the differential movement as a result of interaction of the foundation and structure. This interaction utilizes the mass of the slab and structure and its flexural stiffness and strength. Some contribution may be made by tensile membrane action of the slab. The stiffness of the slab not only reduces the deformations, but also transfers load to the relatively high areas of the foundation, and thereby tends to suppress heave at those locations.



Discuss:

AS2870-2011, C2.1.1

The site classification process requires a secondary classification based on the regional climate and, accordingly, the expected depth of soil moisture change or depth of movement, (Hs). Experience has shown that slightly stiffer footing systems are required in semi-arid areas than in more temperate regions for sites of the same level of classification. This experience suggests that it is not only the magnitude of the movement that dictates the design of the footing; the shape of the distorted ground, as represented by the design parameters of edge distance or mound exponent, also plays an important part in the design. It is proposed that the shape is dependent on the depth of movement, with the most severe distortions occurring in semi-arid areas. This dependency has been expounded in Appendix F of the Standard. Figure C2.1 illustrates the effect of depth of movement on mound shape.

The standard applies climate impacts to only the centre heave mound shape. I will discuss this later.

AS2870-2011

Design by Engineering Principles

TABLE 4.1

MAXIMUM DESIGN DIFFERENTIAL FOOTING DEFLECTION (Δ) FOR DESIGN OF FOOTINGS AND RAFTS

Type of construction	Maximum differential deflection, as a function of span, mm	Maximum differential deflection, mm
Clad frame	L/300	40
Articulated masonry veneer	L/400	30
Masonry veneer	L/600	20
Articulated full masonry	L/800	15
Full masonry	L/2000	10

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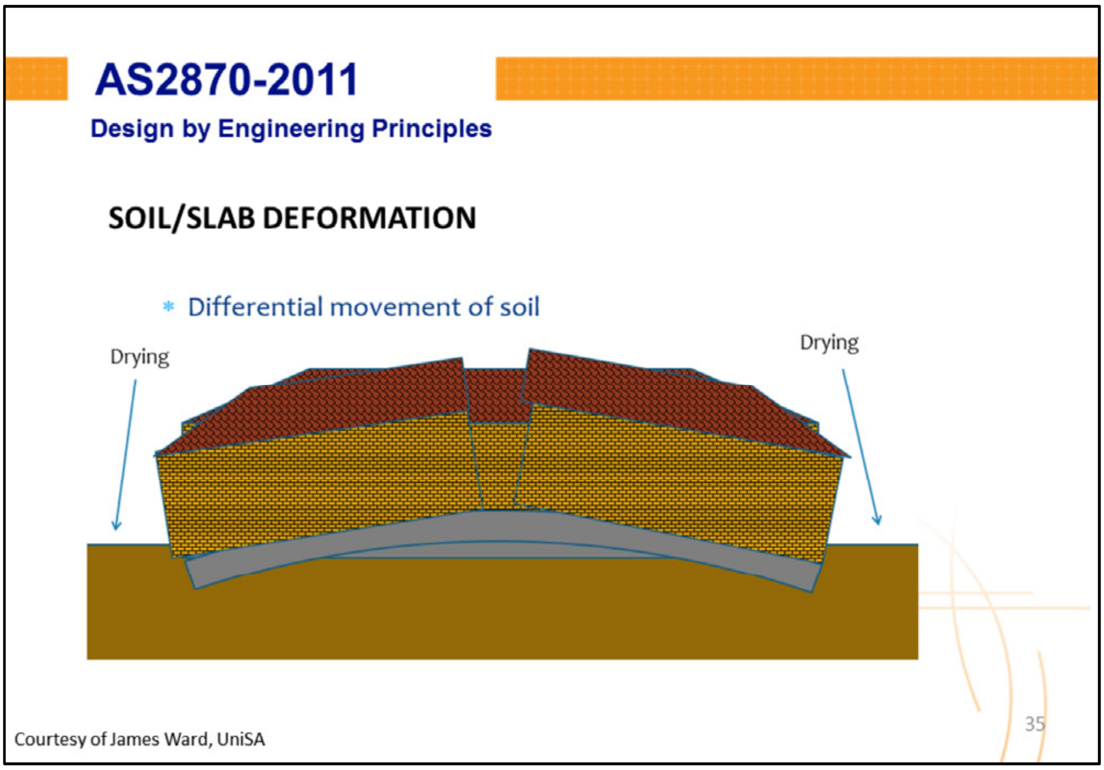
Discuss:

We are designing the footing so the building will achieve the performance requirements set out in Section 1.3.1 and Appendix B of this Standard.

These deflection ratios are not building performance standards. They are a structural design criteria applied to the simplified cylindrical design model we are using to set the target stiffness criteria for the design of the structural elements in that model. If these targets are met in say CORD analysis then the building should achieve the performance standards.

They are not a pass or fail for slabs,


AS2870-2011
Design by Engineering Principles
SOIL/SLAB DEFORMATION
* Differential movement of soil
Drying
Drying
Courtesy of James Ward, UniSA
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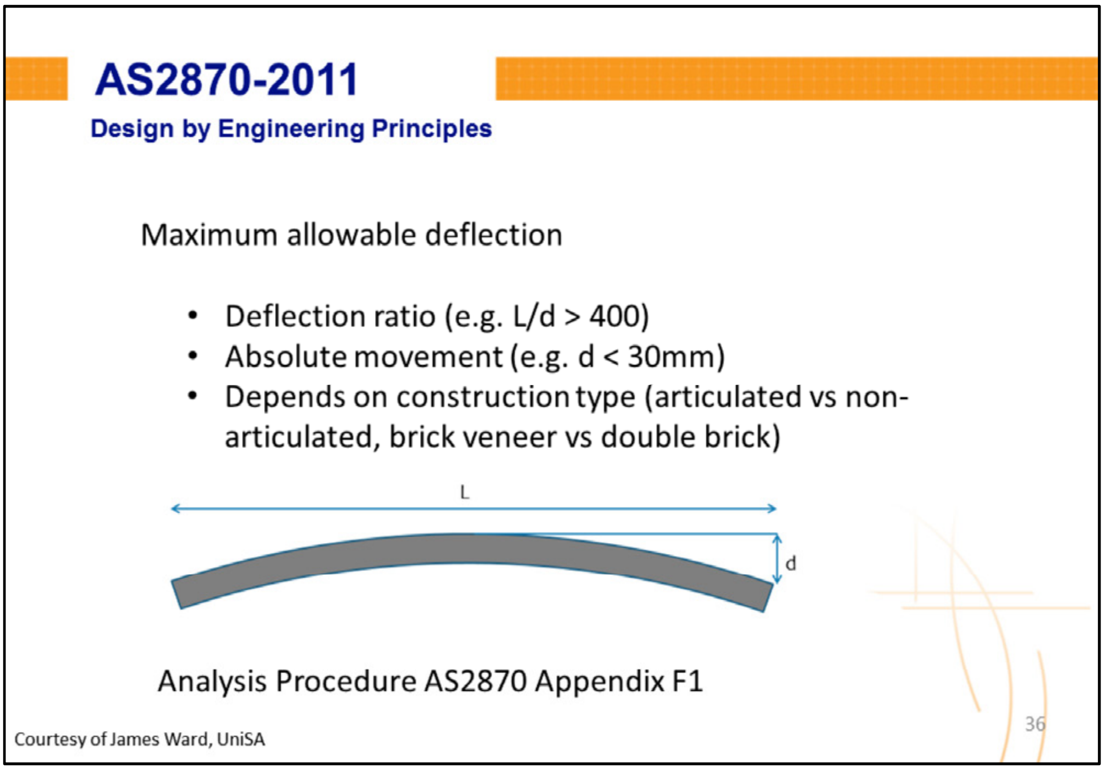
Discuss:



Discuss:

AS2870-2011, F1 ANALYSIS PROCEDURE
Design parameters may be determined by an analysis that allows for interaction of the structure with the foundation over a design range of soil moisture conditions. Generally, the raft should be proportioned to resist positive and negative moments of approximately the same magnitude. The recommended procedure is a computer analysis for the actual loading pattern in accordance with the Walsh or Mitchell methods (Refs 1 and 2, Appendix I).

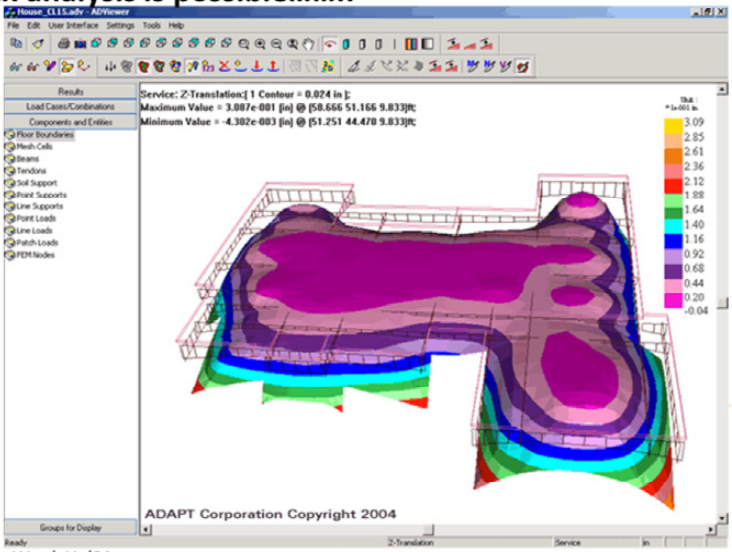
The analysis of non-rectangular buildings is commonly on the basis of overlapping rectangles.

The analysis and design may be based on the total slab cross-section, modified if applicable to incorporate the effective flange widths as defined in Clause 4.4(e).

Section 4.4 gives the structural design rules for proportional raft footings systems including:
- Effective flange widths
- Strength
- Ductility (Mu ≥ 1.2 Mcr)

Design by Engineering Principles

Complex analysis is possible........



Courtesy of James Ward, UniSA

http://www.adaptsoft.com/shots_sog.php

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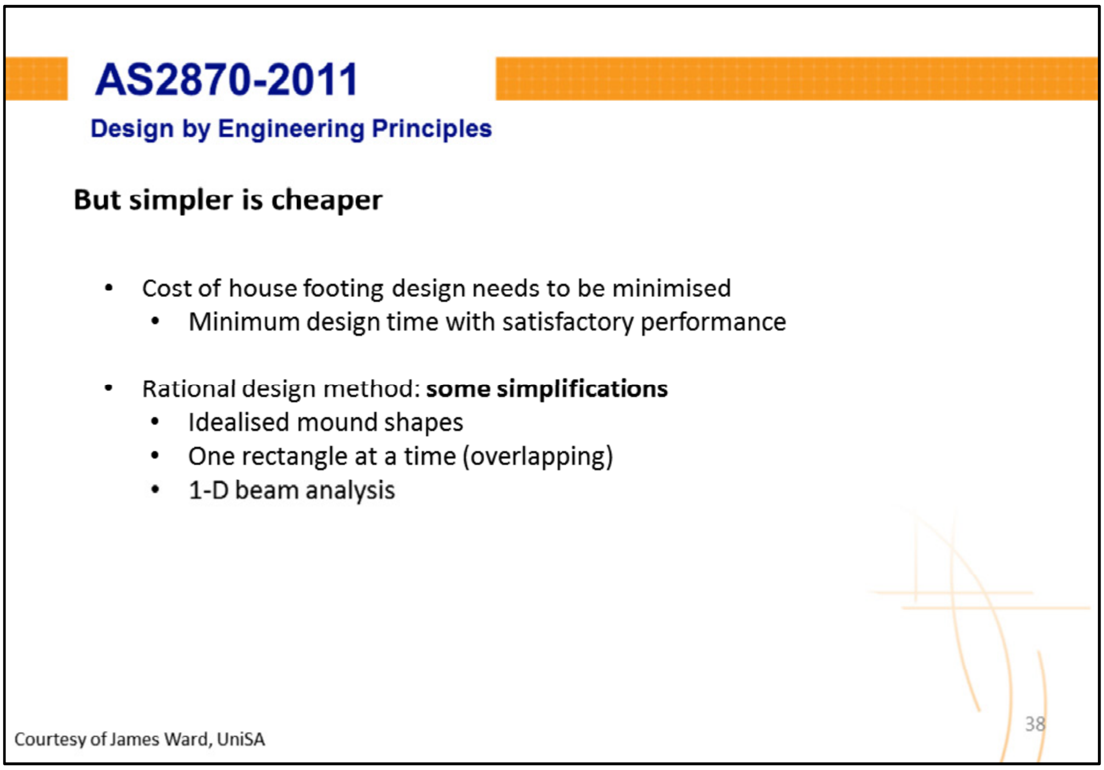
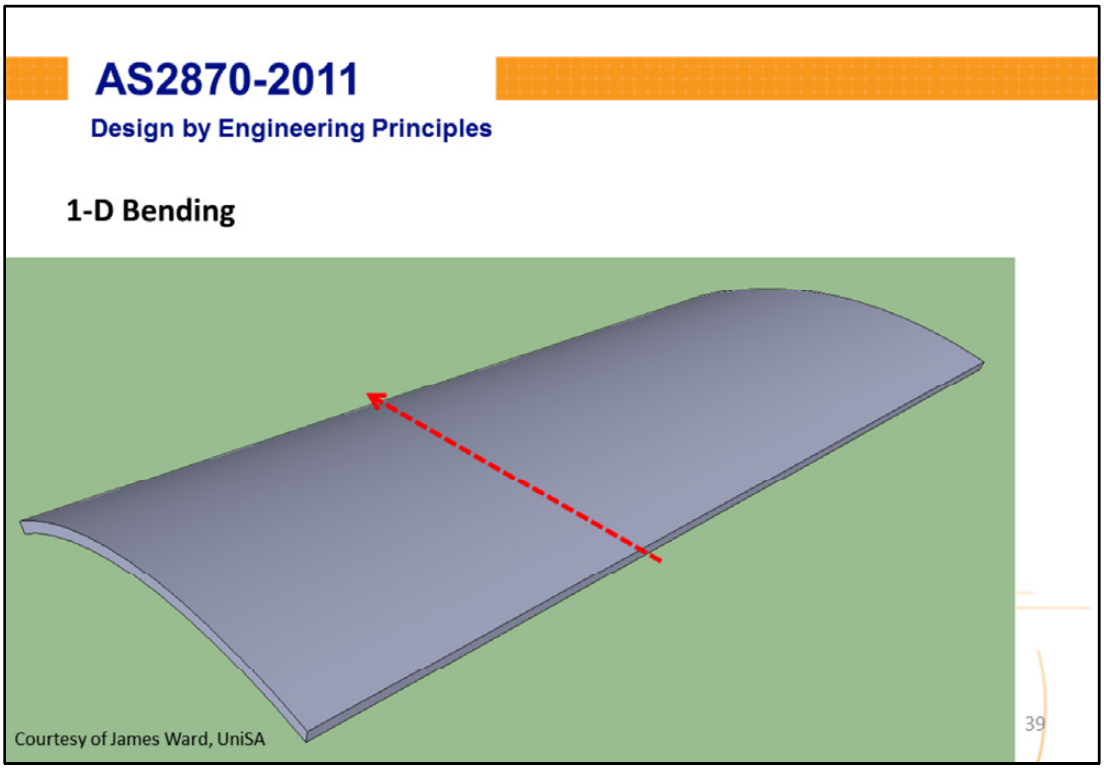
Discussion:

Complex 3D finite element analysis is possible



Discuss:

But the simple cylindrical models using 1D analysis to envelope the more sophisticated results works quite well enough.



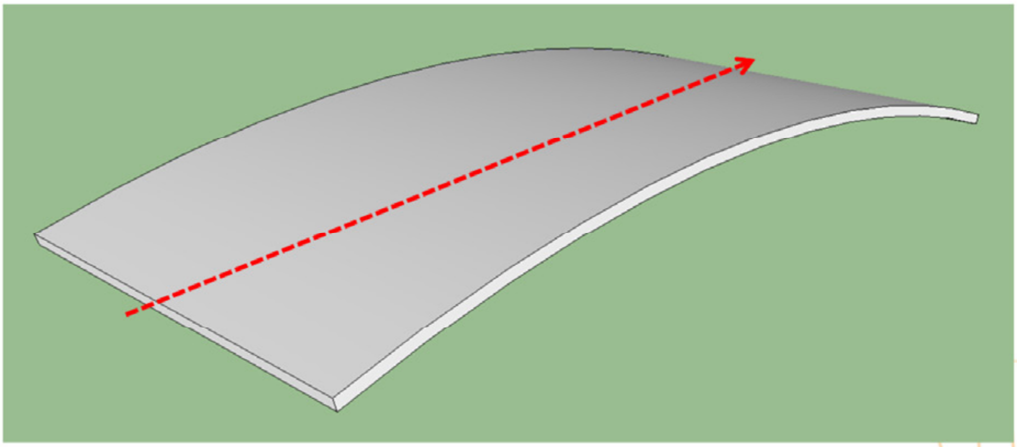
Discuss:

Analysis in the short direction.

AS2870-2011

Design by Engineering Principles

1-D Bending



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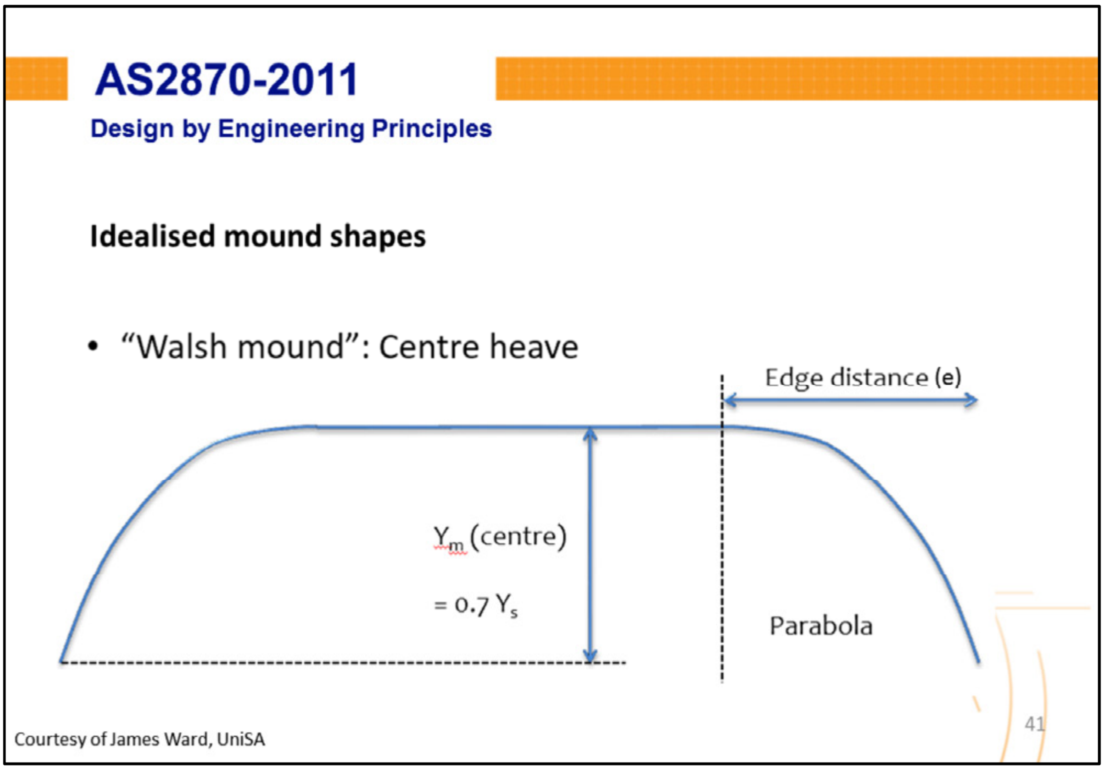
Discuss:

Analysis in the long direction.



Discuss:

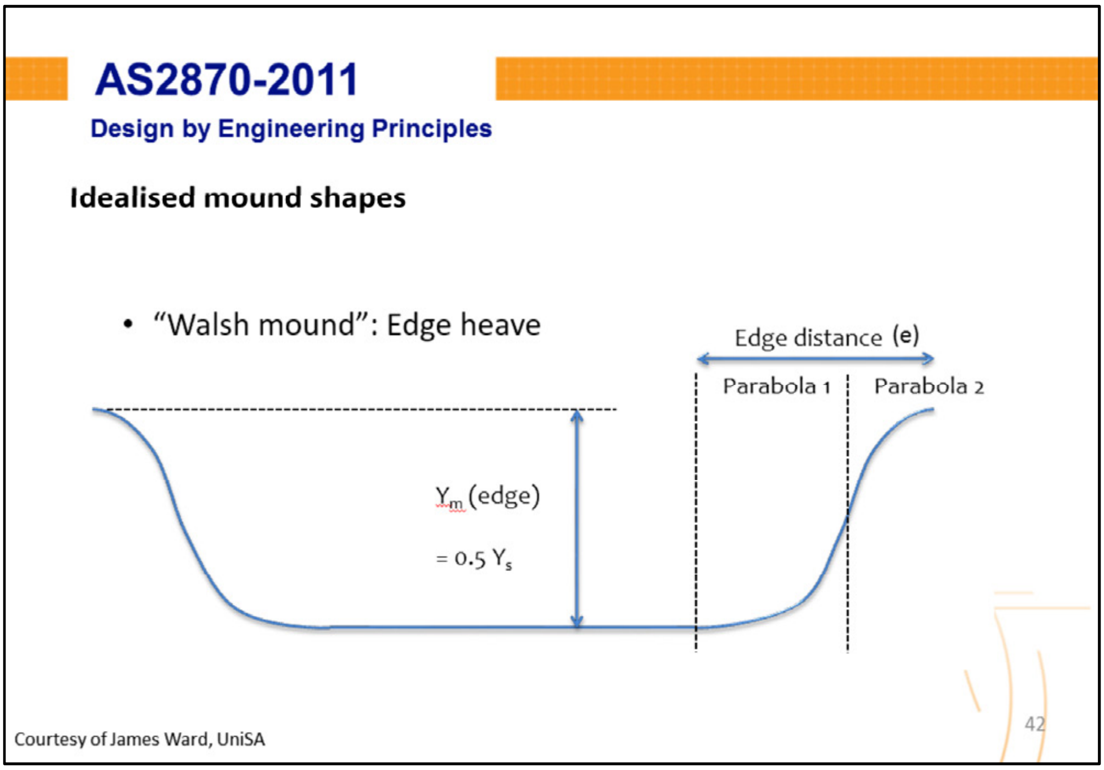
AS2870-2011, F1
The Walsh mound shape is taken as a flat section with a parabolic movement occurring over an edge distance "e".

AS2870-2011, F2
The design differential movement is represented by this idealized mound.

	WALSH	MITCHELL
Centre Heave	y_m=0.7ys	y_m=0.7ys
Edge Heave	y_m=0.5ys	y_m=0.7ys

Appendix F: y_m is estimated taking account of the moisture conditions at the time of construction and the influence of the footings system and paths on the design moisture conditions.

Issues to be accounted for: initially wet site y_{me} can be reduced 40%, other issues eg gilgais and particular geological or drainage features should be used to modify the model.



Discuss:

AS2870-2011, F1

The edge heave shape is a compound parabola and the shape factor for this is given in Figure F2 in Appendix F.

AS2870-2011

Design by Engineering Principles

(b) *Edge distance* The edge distance (e), is taken as:

(i) For centre heave, in metres:

$$e = \frac{H_s}{8} + \frac{y_m}{36}, \text{ where } y_m \text{ is in millimetres and } H_s \text{ is in metres} \qquad \ldots \text{F4(1)}$$

(ii) For edge heave, in metres:

$$e = 0.2\text{L} \leq 0.6 + \frac{y_m}{25}, \text{ where } y_m \text{ is in millimetres} \qquad \ldots \text{F4(2)}$$

For the Mitchell method:

$$\text{Mound exponent } (m) = \frac{1.5L}{D_{cr} - D_e} \qquad \ldots \text{F4(3)}$$

where

D_{cr} = critical depth

$= \frac{H_s}{7} + \frac{y_m}{25}$, where y_m is in millimetres and H_s is in metres

D_e = depth of embedment of edge beam from the finished ground level

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Discuss:

AS2870-2011, Appendix F2:
Edge heave has been taken to be a transitory phase. It may occur before centre heave has been established. The depth of moisture change causing edge heave is likely to be associated with the surface soil effects such as site drainage and certainly no deeper than the depth of seasonal movement. The design suction depth change Hs is usually much greater than seasonal movement particularly in semi arid regions.

In recognition of these differences, the formulae for edge distance (e) and mound exponent (m) depend on both y_m and Hs for the case of centre heave, but only on y_m in the case of edge heave.

So, in case of the **centre heave**, the form of the mound shape depends on climate, whereas in **edge heave,** the mound shape depends on only y_m (surface soil effects and drainage impacts that can be managed away).

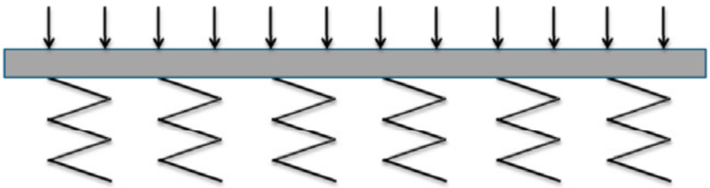
These rules highlight that **the designs were not meant to be** carried out for the abnormal moisture conditions that triggered the subject failures. A normal site is an assumed pre-requisite and the rules in AS2870 for site management are set up to ensure this occurs. I did a paper some years ago which reviewed edge heave failures for raft footings (triggered by abnormal moisture conditions). The conclusion was that to account for these edge heave abnormal moisture conditions the footings would need to be 70% stiffer and stronger than the designs from AS2870. In many jurisdictions, eg Qld and SA, site management, drainage, designs are required as part of the building application so the Engineer can make that assumption.

Design by Engineering Principles

"CORD"

The Basic Theory

- Approximate soil as a set of springs



The "Walsh Method" and "Mitchell Method" are generally represented by the commercially available software programs CORD and SLOG

Courtesy of James Ward, UniSA

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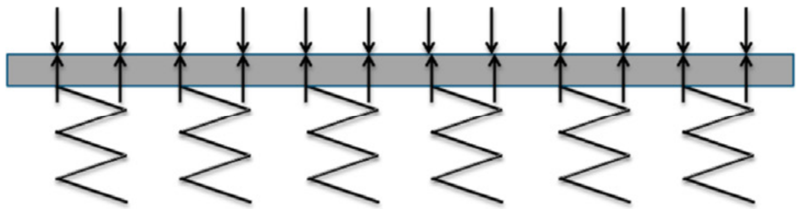
Discuss:

Design by Engineering Principles

“CORD”

The Basic Theory

- Beam squashes springs until spring force balances load on beam



- No differential movement, so no “failure”

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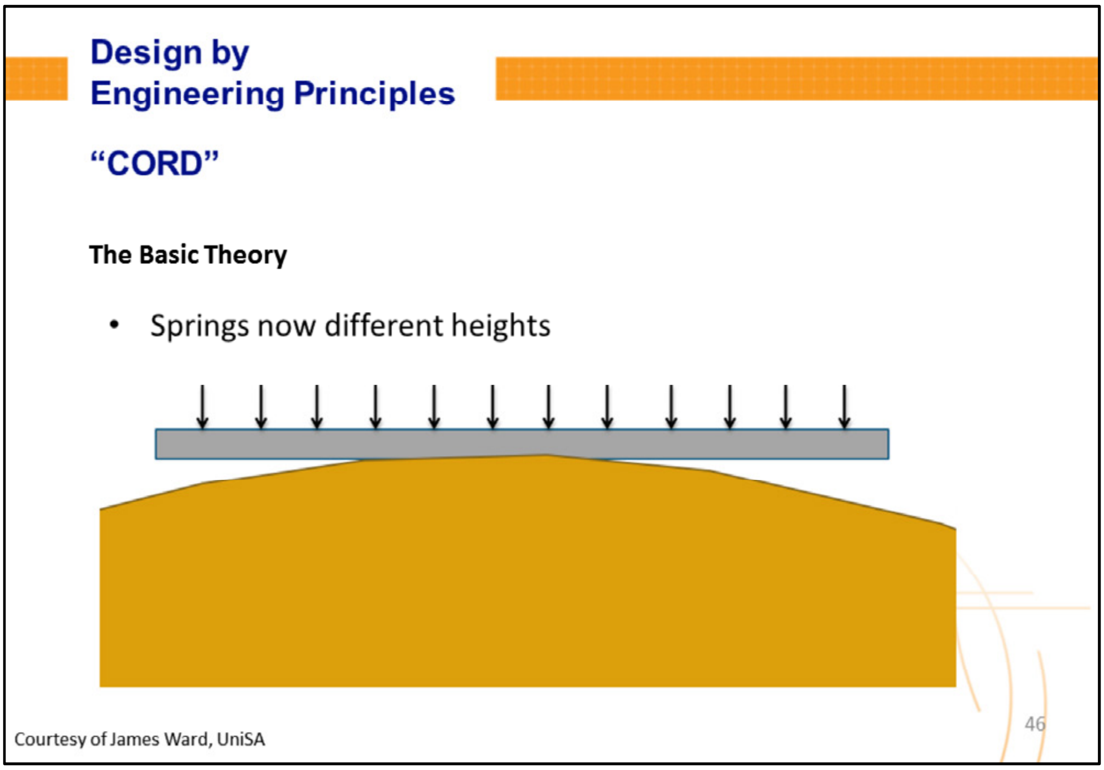
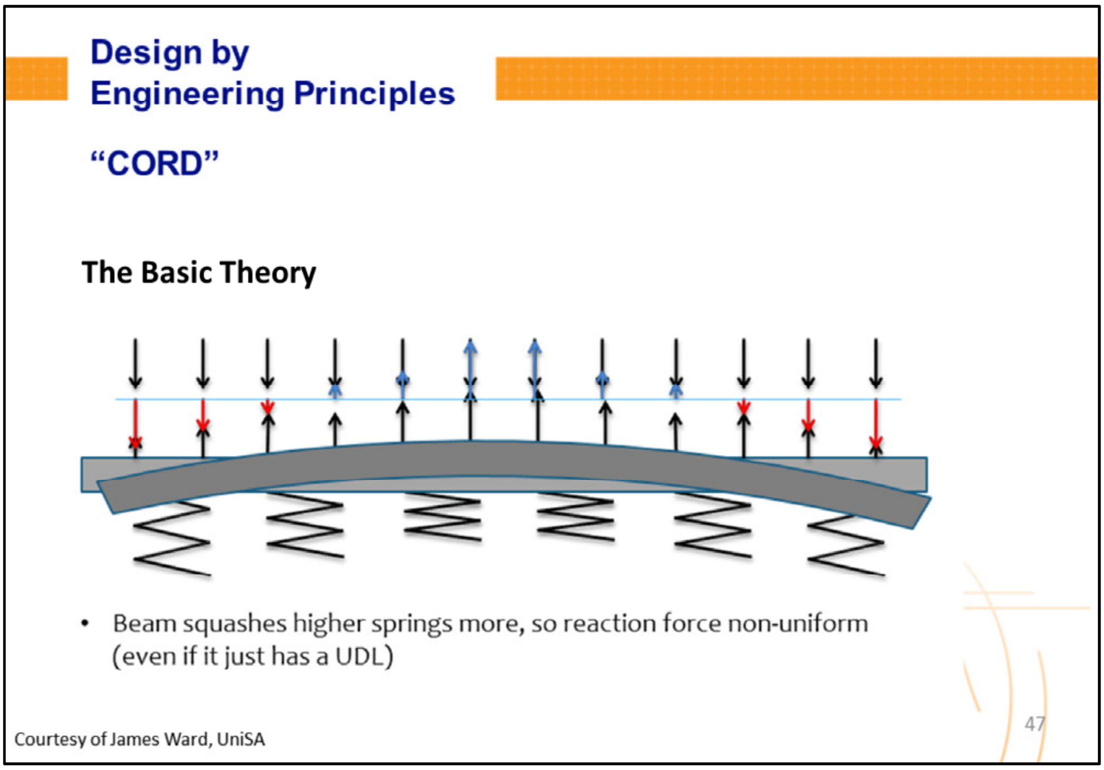
Discuss:



Discuss:

For a mound formation, the springs are now different heights.

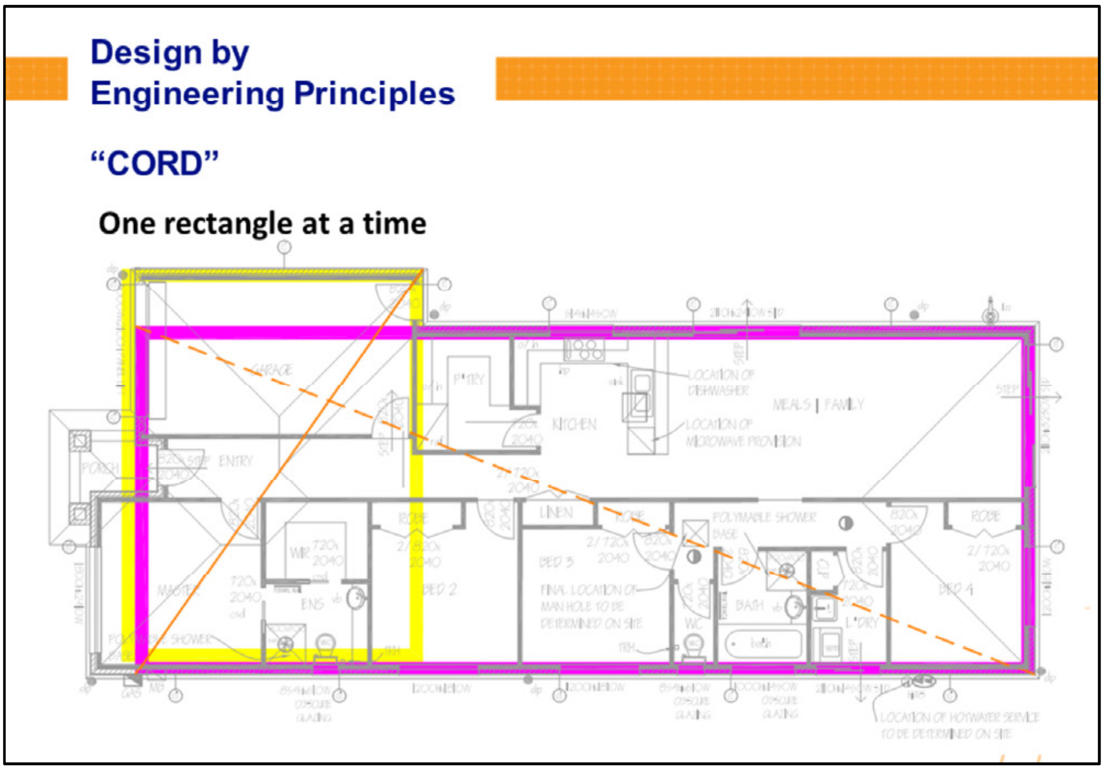


Discuss:

Guidance on the swell stiffness to be used for both swelling clay and for shrinking or stable soil is given in Appendix F2 (c).

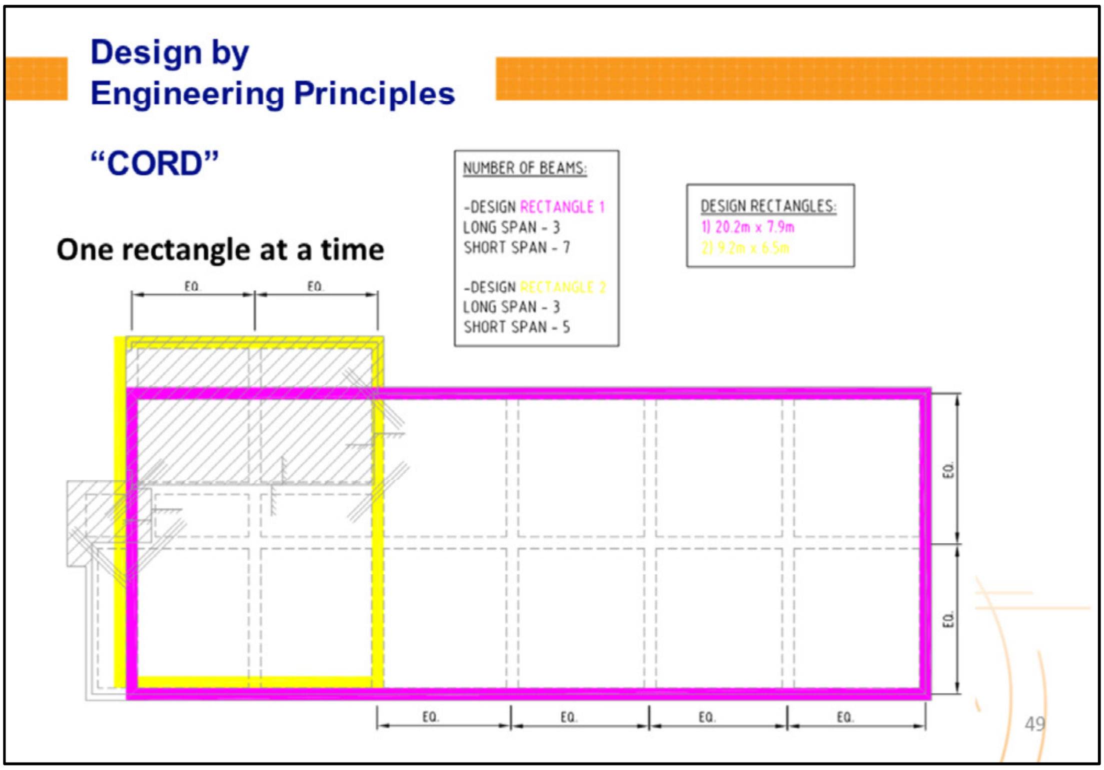
The computed forces and displacements are not particularly sensitive to the value of k except in certain edge heave situations.

As an example, the difference in footing size between using k=1500 kPa/m (Adelaide default value) and 400k Pa/m (Mebourne default value) is only 5-8% .



Discuss:

We model the structure using overlapping rectangles. All this is in section 4 and Appendix F of the Standard.



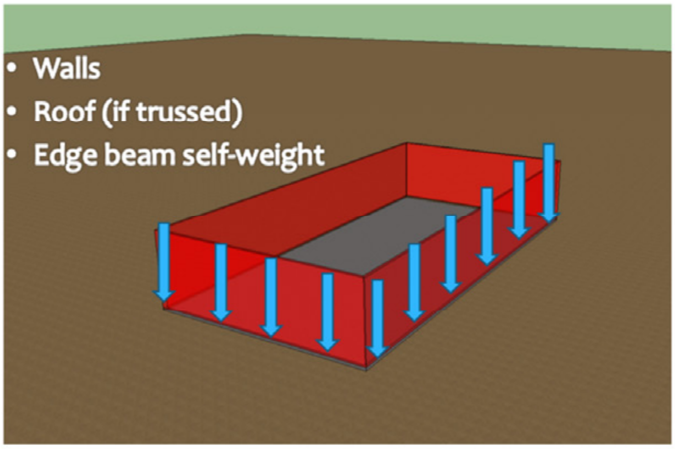
Discuss:

The structural design is based upon the total cross section and you sum the beams in the direction of interest (long or short direction).

Design by Engineering Principles

"CORD"

Edge Loads



Courtesy of James Ward, UniSA

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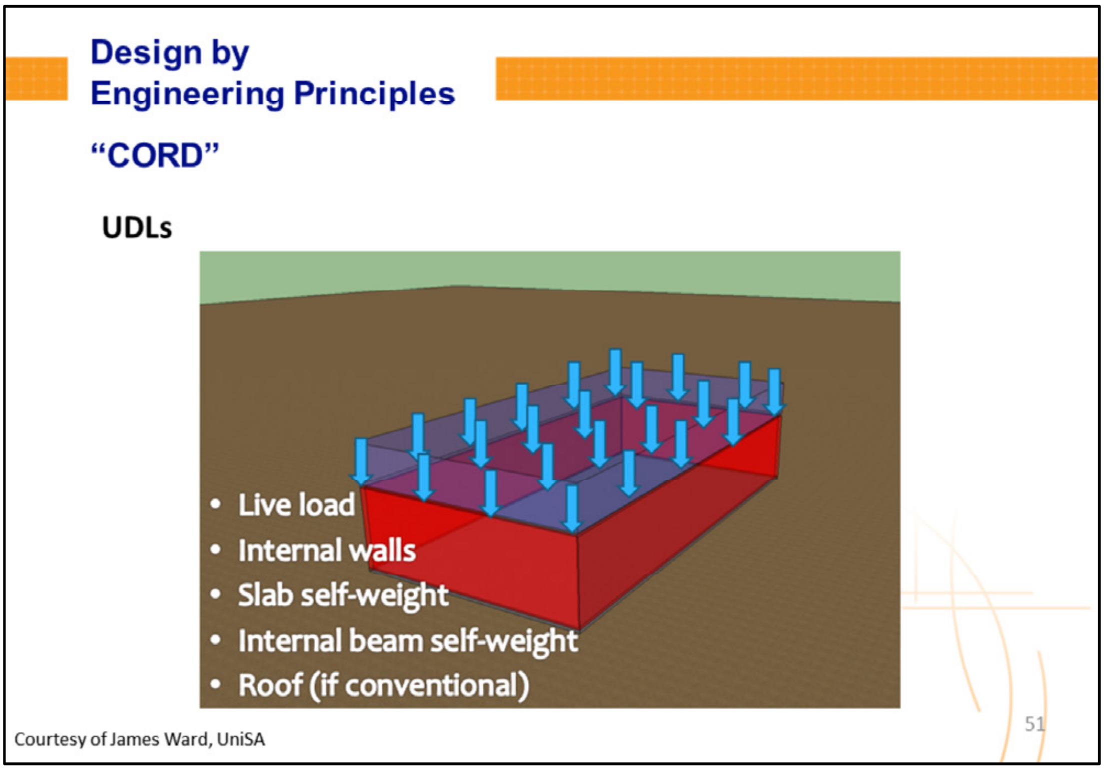
Discuss:

The loading model is simplified, for CORD there are default values based on construction type (but the designer can modify them).



Discuss:

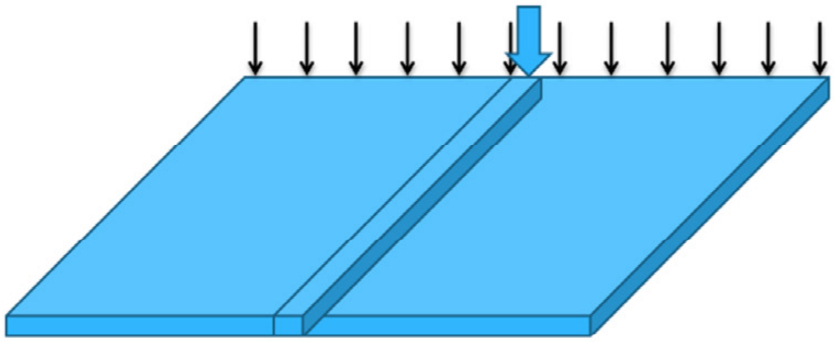
Read

Design by Engineering Principles

“CORD”

Condensing loads to 1-D

- Transverse line loads (including end beams) converted to **point loads**



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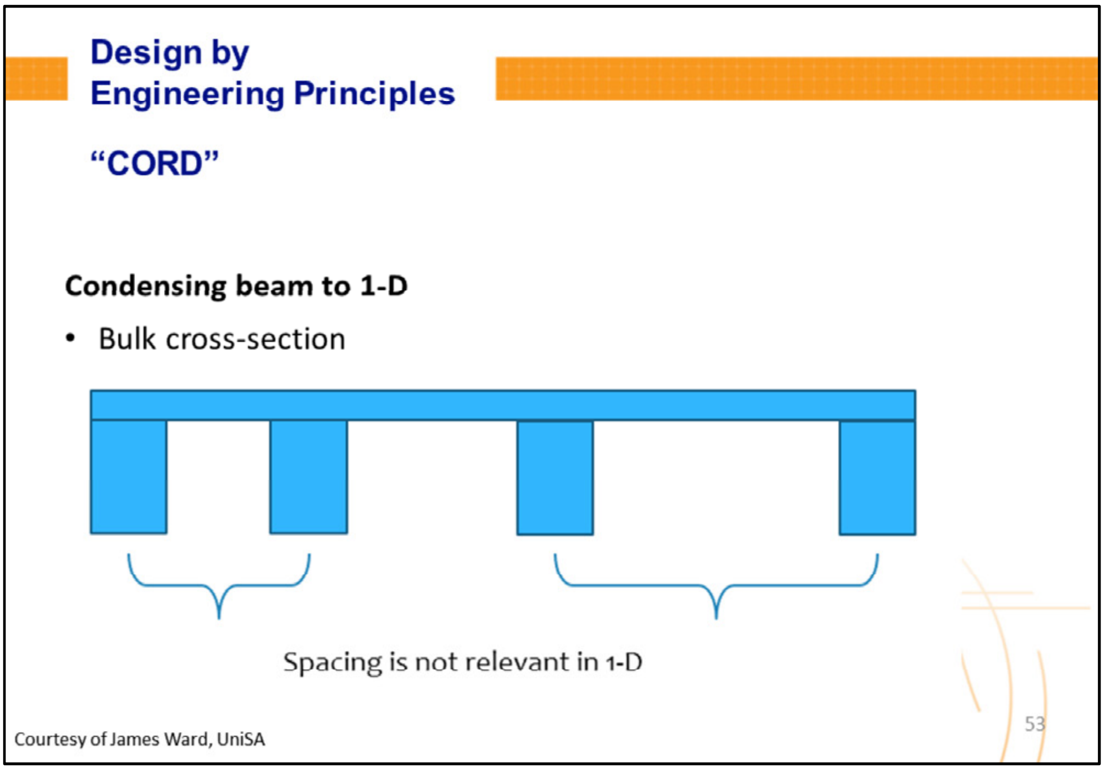
Discuss:

Transverse line loads eg beams are converted to point loads when analyzing the orthogonal direction.



Discuss:

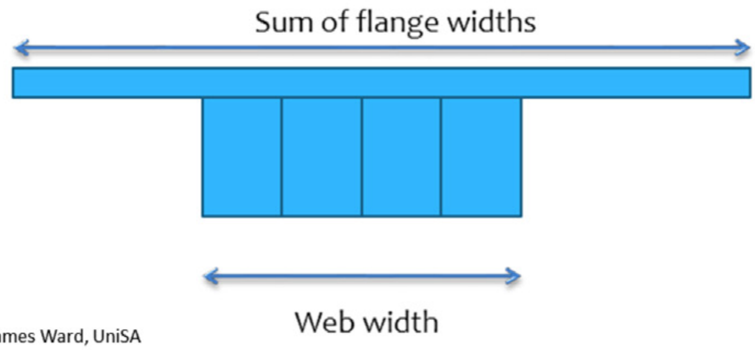
Flick this one – speak to next.

Design by Engineering Principles

"CORD"

Condensing beam to 1-D

- Bulk cross-section
- Transverse beams do not contribute strength



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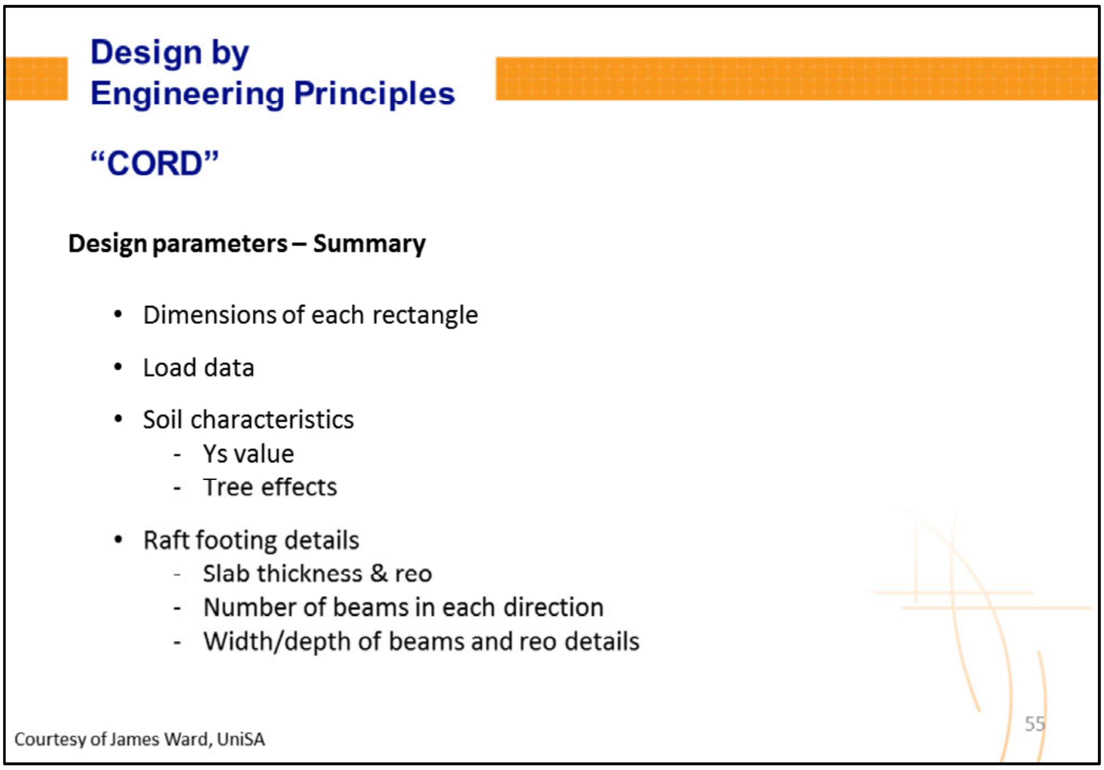
Discuss:

The analysis is based upon the total cross section and beams are summed in each direction.



Discuss:

The CORD design inputs are those set out above.

Soil parameters such as swell stiffness and all structural analysis parameters are default values in CORD, but can be altered to account for site specific issues.

Design by Engineering Principles

“CORD”

What CORD and SLOG do

- Calculate beam stiffness / strength
- Generate mound profile
- 1-D finite element analysis
  - Centre heave & Edge heave
  - Long & short direction
  - Repeat for all rectangles
- Report on **whether the beam is strong enough** to keep **deflection within the AS2870 specified limits**

Courtesy of James Ward, UniSA

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Discuss:

Design by Engineering Principles

"CORD"

Rectangle 2 of 2 (6.5m x 9.2m)

BEAM DEFLECTED SHAPE	CENTRE HEAVE REQUIRED	CENTRE HEAVE ACTUAL	EDGE HEAVE REQUIRED	EDGE HEAVE ACTUAL
DIRECTION 1				
Moment of Inertia (x 10^9 mm^4/m)	0.544 (Ireq)	5.170 (Ieff)	0.385 (Ireq)	5.170 (Ieff)
Flexural Strength (kNm/m)	31.7 (M*)	81.6 (øMu)	22.5 (M*)	70.6 (øMu)
Ductility Check (kNm/m)	53.1 (1.2Mcr)	102.0 (Mu)	42.8 (1.2Mcr)	88.2 (Mu)
Flange Width (m)	External 0.95	Internal 1.6		
DIRECTION 2				
Moment of Inertia (x 10^9 mm^4/m)	1.035 (Ireq)	4.586 (Ieff)	0.547 (Ireq)	4.586 (Ieff)
Flexural Strength (kNm/m)	39.0 (M*)	73.3 (øMu)	21.4 (M*)	60.2 (øMu)
Ductility Check (kNm/m)	49.4 (1.2Mcr)	91.6 (Mu)	37.1 (1.2Mcr)	75.2 (Mu)
Flange Width (m)	External 1.22	Internal 2.14		

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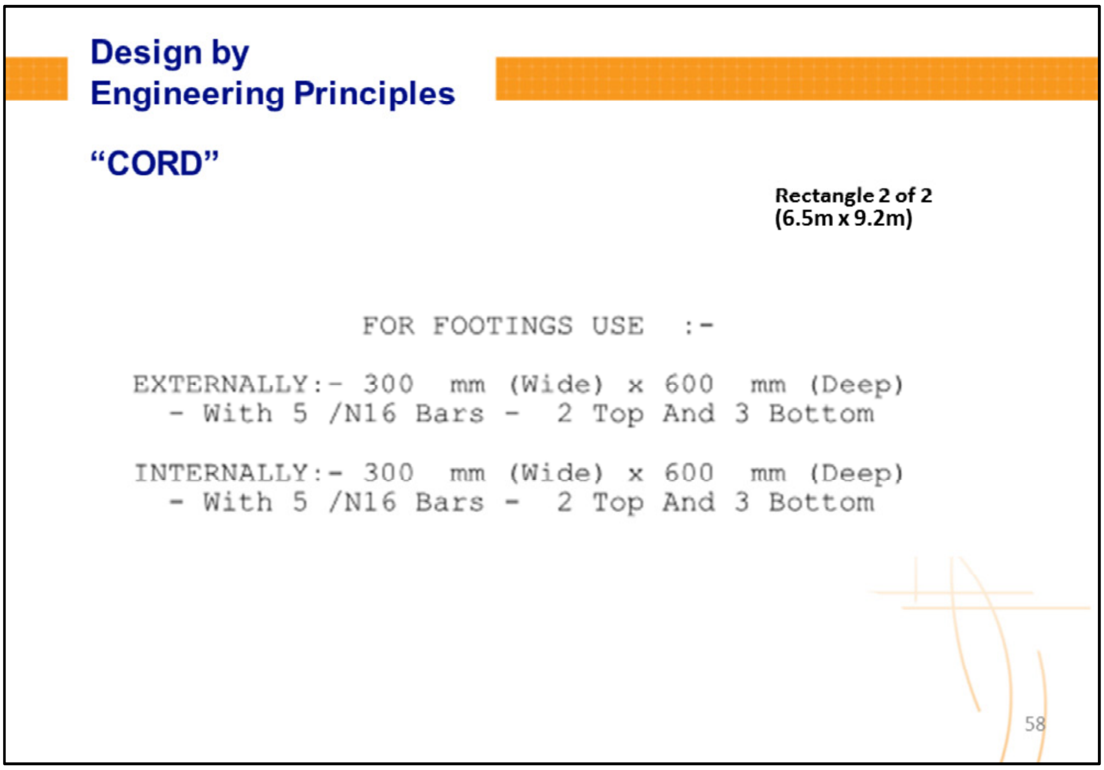
Discuss:

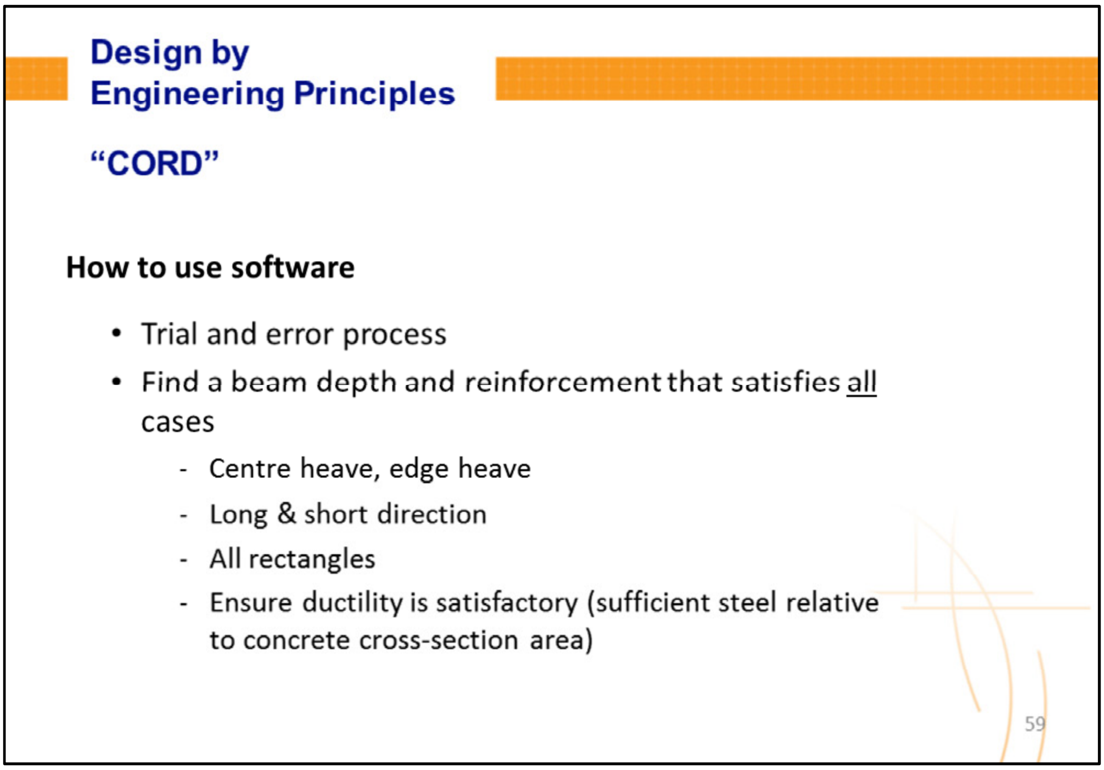
This is a typical output from CORD. It compares the "actual" (what has been designed) to the "required" for:

- Moment of Inertia
- Flexural Strength and
- Ductility



Discuss:

This was for a ys of 80mm for the layouts used earlier.



Discuss:

The CORD software is full of prompts and default values that make it easy to use. One needs to have sufficient experience to have a reasonable idea of the footings sizes that have been targeted and an understanding of the limitations of the soil interaction models and be prepared to modify the model being used for particular site conditions.

Design by Engineering Principles

"CORD"

Balance precision & practicality

- Make sure it is reasonable in terms of constructability i.e.
  - Uniform reinforcement specs
  - Adequate spacing between reo bars
  - Uniform internal beam widths **(for all rectangles)**
  - (Often) same internal & external beam widths
  - Specify reasonable reo bar sizes (e.g. 3 N16, not 1 N28)

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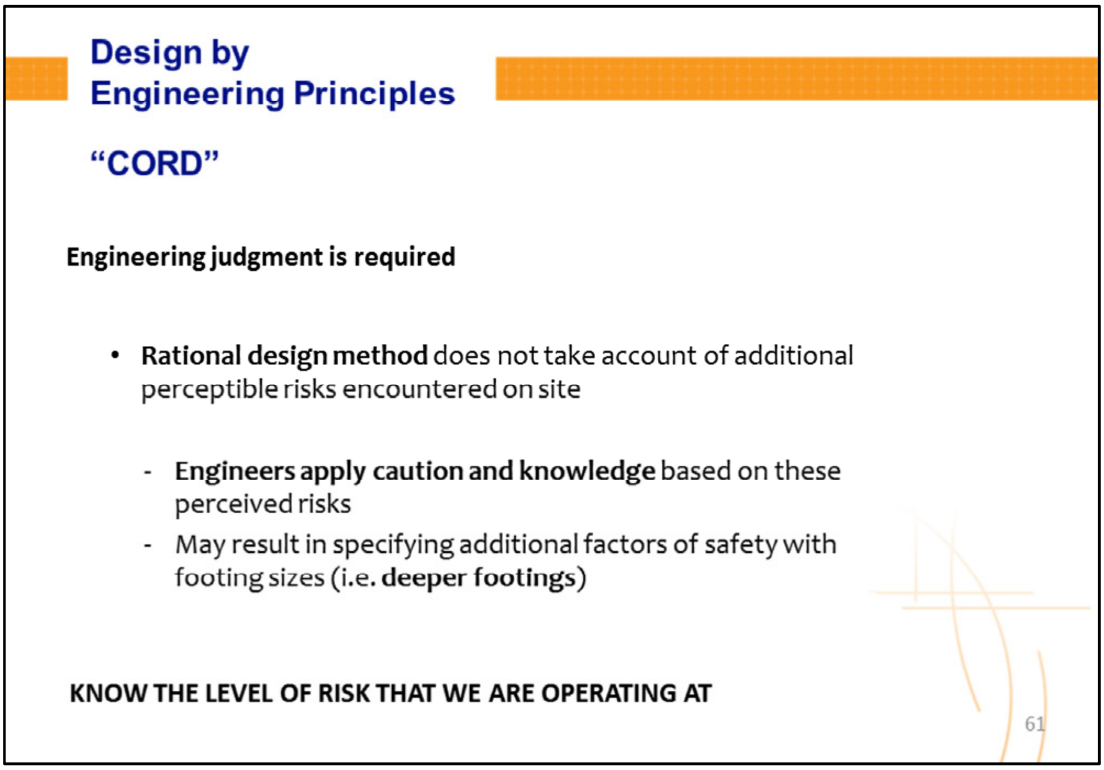
Discuss:

For E sites on Basaltic clay, particularly those where there is significant controlled fill, larger footings sizes will be designed. These may require N16 and possibly N20 bars. 8-10mm ligatures (at say around 1m centre to centre) will be required for spacers.



Discuss:

Specific abnormal moisture conditions can be modelled.

Engineering judgement is required to contextualize the level of risk being operated at and have sufficient regard for the uncertainty of measurement of the parameters being used.