A review and comparison of design methods for raft substructures on expansive soils

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Abstract

Shrink-swell movements of soils cause angular distortion to substructures leading to significant damage to lightweight structures. The built environment of lightweight structures, particularly single-detached dwellings, may compromise the structural performance and cause unforeseen maintenance that may expedite deterioration of the entire build. Due to the importance of damage minimisation in the design phase of single-detached dwellings, this paper aims to review and compare existing design methods for raft substructures on expansive soils through parametric comparison. The comparison considered parameters related to soil properties, environmental factors and stress conditions, including substructure configuration, affecting the shrink-swell potential of expansive soils. The comparison observed that PTI method calculated beam depths with most proximate values to the overall median, while Lytton and Briaud method calculated beam depths closest to the overall third quartile with respect to all considered design methods. WRI and BRAB method obtained larger values of beam depths, specifically for scenarios with higher plasticity index, liquid limit and longer span, which can be considered as outliers. AS 2870, Walsh and Mitchell method were in the less conservative range based on the range of beam depths calculated. Calculated required beam depths ranged from 300 to 815 mm neglecting outliers with higher dispersion of values when the active depth zone was deeper, the plasticity index and liquid limit were higher, applied uniform load was higher and span of the substructure was longer. This review paper presents the range of probable values, variability and degree of central tendency depending on the values of beam depths calculated by different current design methods that are useful for designers.

Keywords: Raft substructures, expansive soils, lightweight structures, beam design depths

1. Introduction

Expansive soils cause significant damage to lightweight structures due to differential movements of the ground affecting substructures and subsequently superstructures (Teodosio et al., 2019a, Li and Cameron, 2002, Karunarathne, 2016). The shrink-swell movement of a ground is due to the decrease and increase of soil moisture, which is severe in sites with finer soils and higher plasticity (Briaud et al., 2016, Kodikara et al., 2013). The damage induced by expansive soils cost significant global financial loss (Jones and Jefferson, 2012, Miao et al., 2012, Skinner et al., 1998, Cameron et al., 1987, Krohn and Slosson, 1980).

The difference of the rate of soil moisture evaporation and infiltration between the free soil and paved area leads to centre heave, shown in Figure 1a, and edge heave, presented in Figure 1b (Adem and Vanapalli, 2015, Rajeev et al., 2012, Masia et al., 2004). The critical centre heave due to shrinking soil at the open ground causes a raft substructure, with a length L, to behave as a double cantilever supported by the soil in contact at the centre of the covered ground while developing separation at the edges over a certain distance e due to soil vertical movement y_m shown in Figure 1a (Tran et al., 2019, 2018, Kodikara and Costa, 2013). Contrarily, the critical edge heave due to swelling soil at the open ground causes a substructure to act as a simply-supported beam while the centre of the covered ground develop separation (Figure 1b). Depending on the design method used, most available methods assume a predefined soil mound profile for edge heave and centre heave (e.g. using shape factor W_f in Walsh method). The traditional assumption of a raft substructure over a distorted soil mound permits iterations to find the corresponding internal forces and required stiffness based on the permissible soil movement considering the Ultimate Limit State (ULS) and Serviceability Limit State (SLS).

Different design methods in the literature are available for practitioners (Teodosio et al., 2018, 2019b). One of the early proposed design methods is developed by Building Research Advisory Board (1968), recognised as BRAB method, and was afterward modified by Snowden (2008) as WRI method. Another established design method uses a beam-on-mound equation with Winkler or coupled springs, which was initially applied by Lytton (1970), known as Lytton method, and was then modified by Walsh and Walsh (1986), as Walsh method, and Mitchell et al. (1984), as Mitchell method. Subsequently, Standard Australia (2011) adopted the methods of Lytton (1970), Walsh and Walsh (1986) and Mitchell et al. (1984) and developed the AS 2870 method. Another approach used non-linear empirical equations derived by Post Tensioning Institute (2008a,b) recognised as the PTI method.

Some design methods used numerical simulations, for instance, Fraser and Wardle (1975) that modified the Winkler and coupled springs in Lytton method and Walsh method. Holland et al. (1980) developed the Swinburne method by improving the approach of Fraser and Wardle (1975). Other numerical methods include Sinha et al. (1996) who applied the equations proposed by Lytton (1970) to conduct parametric simulations with varying stiffness of slab systems in relation to free soil heaving. Li (1996) developed a coupled thermo-mechanical model to emulate the hydro-mechanical properties of expansive soils. Totoev and Kleeman (1998) proposed an infiltration model to determine pore pressure distribution to predict shrink-swell movements, while Bulut (2001) analysed slabs using a plate theory. Other design methods consider soil-structure interaction by using three-dimensional models for soil volume changes considering change in ψ_w (El-Garhy and Wray, 2004, Masia et al., 2004, Wray et al., 2005, Fredlund et al., 2006). Coupled hydro-mechanical models were also developed using climatic, soil and structural data as inputs (Teodosio et al., 2020, Zhang, 2005, Abdelmalak, 2007, Zhang and Briaud, 2010, Weerasinghe et al., 2015, Shams et al., 2018, 2019). Dafalla et al. (2011) and Briaud et al. (2016) proposed new design methods for raft substructures based on parametric simulations using finite element method.

Current design methods have different parameter considerations and theoretical formulations that affect the design process of raft substructures on expansive soils. Hence, practitioners and researchers should understand the assumptions and considerations incorporated in a design method being used for raft substructure on expansive soils. Due to the importance of damage minimisation in the design phase of single-detached dwellings, this paper aims to review and compare existing design methods for raft substructures on expansive soils through parametric comparison. The parametric comparison considers soil properties, environmental factors, stress condition and structural configurations affecting the shrink-swell potential of expansive soils and the movement of substructures. The comparison involves AS 2870, Walsh, Mitchell, BRAB, Lytton WRI, PTI and Briaud methods used to calculate required beam depths.

2. Design methods

Design methods consider essential parameters for design specifications of raft substructures on expansive soils. These parameters are related to soil properties, environmental factors and stress conditions that affect required structural specifications.

The parameters related to soil properties are one of the primary considerations to determine the reactivity of soils. Soil properties being used in design methods, influencing shrink-well potential of ground include, clay mineralogy, amount of fine clay, plasticity and permeability.

The soil condition and environmental factors greatly affect the shrinking and swelling of expansive soils mainly through water infiltration and evaporation causing soil moisture ingress and egress (Karunarathne et al., 2018). The primary parameters influencing the soil environment include initial and current soil suction, soil moisture, climate, drainage, groundwater and presence of vegetation.

The stress history and in-situ conditions have a significant influence on the shrink-swell potential of expansive soils. Expansive soil with an overconsolidated state is more critical than normally consolidated soil with comparable void ratio (Gould et al., 2011). Even though swell pressure increases depending on the period of load application on compacted clays, the magnitude of shrink-swell movements under a lightweight structure has been shown to reach stabilisation due to wetting and drying cycles (Tripathy and Rao, 2009, Tripathy et al., 2002).

2.1. BRAB Method

Building Research Advisory Board (1968) is one of the early design methods for substructures considering expansive soils. This approach assumes a rectangular mound shape, which disregards the effect of slab thickness along an unsupported distance (National Research Council , U.S.). The entire irregular slab is divided into overlapping regular rectangles of length, L, and width, B. The effective load for each rectangle is calculated corresponding to its aspect ratio using the uniformly distributed load applied on the slab-on-ground. The maximum internal moment,force and deflection can now be calculated correspondingly using

$$M_{max} = \frac{wL^2B(1-C)}{8}, \tag{1}$$

$$V_{max} = \frac{wLB(1-C)}{2}, \tag{2}$$

$$\Delta_{max} = \frac{wL^4B(1-C)}{48EI}. \tag{3}$$

where M_{max} is the maximum internal bending moment, V_{max} is the maximum internal shear force, Δ_{max} is the maximum differential deflection of a substructure and C is the support index.

The parameters related to soil properties used in this method can either be I_{ss}, PI or Potential Volume Change or PVC-reading. The support index, C, can then be determined using Figure 6 in Building Research Advisory Board (1968), which is influenced by a parameter related to environmental factors denoted as C_W. This parameter is based on the fluctuation of climate in an area dependent on the total annual precipitation, uniformity and distribution of precipitation, number of precipitation occurrence, duration of each occurrence and amount of precipitation per occurrence. This parameter simplifies the effect of the difference in u around free field areas, u_{ff}, and under substructures, u_e. If wide variations in soil moisture is experienced by expansive soils due to precipitation dynamics, substructures will be subjected to sequence of shrink and swell movements; having larger movements if precipitation occurrences are intermittent.

The parameters related to stress condition of soils include the dimensions of substructures, L and B, and the uniform load, q, applied on the substructures. Eq. 1 and 2 are then used to assess if the design of substructure dimensions suffice the ULS or strength requirement, while Eq. 3 is used to comply with the SLS or deformation and cracking requirement.

The magnitude of surcharge load and the area of application contributes to the magnitude of soil volume change of expansive soils (Masia et al., 2004, Bishop, 1959). A substructure with length, L, and width, W, affects the soil-structure interaction through structure continuity. Uniform load pressure, p, and line load, q, play an important role to resist the edge heaving effect of swelling soils shown in Figure 1b (Mokhtari and Dehghani, 2012, Abdelmalak and Briaud, 2016). Substructures should have sufficient stiffness, EI, depending on the type of construction (i.e., clad frame, masonry veneer or full masonry) to carry these loads specifically in a centre heave scenario when substructures are acting as cantilever like in Figure 1b (Standard Australia, 2011, Masia et al., 2002).

BRAB Method has a relatively simplistic and empirical process in designing substructures on expansive soils compared to other approaches. However, this design method calculates more conservative design of substructure cross sections, specifically when L is longer due to the direct proportionality of the cantilever length leading to soil-substructure separation, l_c, to the corresponding substructure dimensions (Abdelmalak, 2007).

2.2. WRI Method

Snowden (2008) presented the WRI Method by testing slabs and modifying equations based on Building Research Advisory Board (1968). The bending moment, shear force, and deflection are calculated using

$$M = \frac{pB(k_l l_c)^2}{2}, \tag{4}$$

$$V = pBk_l k_c, \tag{5}$$

$$\Delta = \frac{p(k_l l_c)^4 B}{4E_c I}, \tag{6}$$

where k_l and k_c are design factors, l_c is the cantilever length of a substructure, E_c is the concrete elasticity for creep and I is the moment of inertia of a cross section of the substructure.

The parameter related to environmental factors used in this method is C_w, which is the same parameter used in BRAB Method. A parameter related to both soil properties and environmental factors called soil-climate support index $(1-C)$ was introduced and can be determined using PI and C_w. The cantilever length, l_c, is then obtained using the effective plasticity index (PI_{eff}) from the underlying ground considering a depth of around 4.6 m. This parameter, l_c, can be modified using multiplier k_l based on the dimension of substructures. The parameters related to stress conditions of soils used in WRI method are p, L, B, E_c and I.

WRI method is based on the empirical method of Building Research Advisory Board (1968), which calculates conservative dimensions of substructures. Both BRAB method and WRI method use C and $1 - C$, parameters reflecting the relationship between the soil properties and environmental factors, where there are no available data or documentation on the rationality of this interrelation. This relationship appears to be based on experience of a number of people with less probable independent evaluation of its derivation (Abdelmalak, 2007). The parameters related to soil properties, PI, I_{ss} and PVC-reading, are the solely basis of these methods to determine the reactivity; however, these parameters are sensitive to their initial conditions and are dependent to the variation of weather and time that can be more accurately reflected by soil matric suction, u.

2.3. Lytton Method

Lytton (1970) proposed a curved mound using elastic mathematical models of beam and slab based on the beam-on-mound equation, Eq. 17. The centre heave uses Winkler spring model and the edge heave uses a coupled spring model. The one-dimensional moment for centre heave (M_c) and edge heave (M_e) is given by

$$M_c = \frac{pLB}{2} + \frac{L^2}{8}(2p + q' + qB) - C'\frac{rL}{8}, \text{and} \tag{7}$$

$$M_e = \frac{q'LB}{4} + \frac{L^2}{8}(2q + pB) - C'\frac{rL}{8}, \tag{8}$$

where q' is the line load acting through the centre of a building, r is the total load acting on the slab and C' is the modified support index. Eq. 7 and 8 are adjusted for the two-dimensional moment in the long direction (M_L) and short direction (M_S) given by

$$M_L = M_c\left(1.4 - 0.4\frac{L}{B}\right) = M_e\left(1.4 - 0.4\frac{L}{B}\right), \text{and} \tag{9}$$

$$M_S = M_c\left[1 + 0.9(1.2 - C')\left(\frac{L}{B} - C'\right)\right] = M_e\left[1 + 0.9(1.2 - C')\left(\frac{L}{B} - C'\right)\right], \tag{10}$$

and the shear force, V, and the deflection, Δ, is given by

$$V = \frac{4M_L}{L} = \frac{4M_S}{L}, \tag{11}$$

$$\Delta = \frac{M_L L^2}{12EI} = \frac{M_S L^2}{12EI}. \tag{12}$$

Lytton (1970) improved the methodology of Building Research Advisory Board (1968) by suggesting an equation for the support index given by

$$C' = \frac{m+1}{m+2}\left(\frac{m+1}{m}\frac{1}{ky_m}\frac{r}{A}\right)^{\frac{1}{m+1}}, \tag{13}$$

where A is the slab area.

The parameter C', which is similar to the support index suggested by the Building Research Advisory Board (1968), is now related to both soil properties and environmental factors affecting the shrink-swell potential of soils. The parameters related to stress conditions of expansive soils considered in this method are p, q, q', r, L and B.

This design method was developed using a closed-form solution using a finite difference analysis using a beam-on-mound equation with coupled spring and Winkler spring, which is a more mathematical approach compared to other methods that are mostly empirical. It is also important to note that this method assumes an infinitely-stiff concrete substructure.

Lytton (1970) provided the spark to develop the current Australian design methods discussed in the succeeding sections.

2.4. AS 2870 Method

The performance criteria and design specifications of substructures for Australian soil conditions are stipulated primarily in the Australian Standard (AS) 2870-2011 (Standard Australia, 2011). This standard focuses on design of substructures on expansive soil susceptible to ground movement for class 1 and 10a buildings such as single-detached dwellings and garages (Australian Building Codes Board, 2016).

AS 2870 Method (Standard Australia, 2011) specifies standard deemed-to-comply designs (Section 3) and a customised design using engineering principles (Section 4) for slab substructures and strip substructures of residential structures and garage. Deemed-to-comply designs of stiffened and waffle rafts shall conform with the limitations stated in Clause 3.1.1. If the requirements and limits in Clause 3.1.1 were not satisfied, Section 4 should be used for a custom-made design of substructures. A simplified method for raft designs presented in Section 4.5, which can also be used to design raft substructures provided that the parameters are within the required limits. This alternative design to extend the validity of Clause 3.1.1 was based on interpolated design calculations using Walsh method (Section 2.5).

The parameter related to soil properties and environmental factors being considered in AS 2870 is the active depth zone, H_s, which classifies the site whether it has a normal profile having $H_s < 3$ (Site Class M, H_1 and H_2), or a deep-seated movement profile having $H_s \geq 3$ (Site Class M-D, H_1-D or H_2-D). Sites with normal profile require smaller values of unit stiffness presented in Figure 4.1 of AS-2870, denoting reduced design stiffness (EI) requirements leading to reduced design beam depths. On the other hand, sites with deep-seated movement profile require large values of unit stiffness resulting into higher EI requirements. Since the plots in Figure 4.1 of AS 2870 are linear, these can be simplified and presented using the following equations to calculate the overall beam depth (D, in mm),

$$\frac{y_s}{\Delta} = \frac{45}{22}\left[log\left(\frac{\sum \frac{B}{D^3}}{L}\right)\right] - \frac{689}{44} for H_s < 3, \text{and} \tag{14}$$

$$\frac{y_s}{\Delta} = \frac{15}{7}\left[log\left(\frac{\sum \frac{B}{D^3}}{L}\right)\right] - \frac{481}{28} for H_s \geq 3, \tag{15}$$

where Δ is the allowable substructure deflection depending on type of construction (i.e, clad frame, masonry veneer or full masonry), B is the effective width of the web of a beam, D is the depth of a beam and L is the overall length of a slab. The parameter y_s is defined as the surface characteristic movement used to classify the site reactivity (Table 1) calculated using

$$y_s = \frac{1}{100}\sum_{n=1}^{N}(I_{pt}\Delta\overline{u}h)_n = \frac{1}{100}\sum_{n=1}^{N}(\alpha I_{ss}\Delta\overline{u}h)_n \tag{16}$$

where I_{pt} the instability index, $\Delta\overline{u}$ is the average soil suction change over the layer thickness, α is the lateral restraint factor (Al-Shamrani and Dhowian, 2003), I_{ss} is the soil shrinkage index (Fityus et al., 2005, Zou, 2015), h the individual soil layer thickness and N is the number of distinct soil layers.

Parameters affecting the stress conditions such as B and L are necessary to determine the required beam depths in Eq. 14 and 15. Uniform area load and line load, p and q, are not part of the input; however, these factors may already be intrinsically considered due to the interpolated calculations using Walsh method with typical loadings applied to residential substructures.

2.5. Walsh Method

Walsh and Walsh (1986) method is an approach based on computer analysis considering the total slab cross-section of overlapping rectangles. The mound in this method is assumed to be a flat section with movement occurring over an edge distance, e, shown in Figure 1. This method adopted the beam-on-mound equation by Lytton Method (Payne and Cameron, 2014) given by

$$\frac{d^2}{dx^2}\left(EI\frac{d^2\delta}{dx^2}\right) - \frac{d}{dx}\left(GB\frac{d}{dx}(\delta - y)\right) + kB(\delta - y) = p, \tag{17}$$

where x is the distance along the beam, y is the distance below the highest point of the mound, G is the shear stiffness, δ is the beam deflection, k is the soil stiffness and B is width of substructures. The first term links the beam curvature, beam stiffness and applied loads; the second term shows the behavior of the soil with shear coupling; and the third term represents the shrinking or swelling of the soil. Walsh Method extended Eq. 17 by using a mathematical representation of a reinforced concrete beam instead of an infinitely-stiff concrete substructure. The revised equation requires finite element method to solve for the required EI. The shape of the mound is represented by the edge distance, e, that can be calculated using

$$e = \frac{H_s}{8} + \frac{y_m}{36}, \text{for centre heave; and} \tag{18}$$

$$e = min\left[0.2L, 0.6 + \frac{y_m}{25}\right], \text{for edge heave,} \tag{19}$$

where y_m is the differential mound movement, a parameter related to soil properties being considered in this method which can be calculated as $0.7y_s$ for the centre heave and $0.5y_s$ for the edge heave on initially dry site. On a site with wet profile during the initial construction stage, a reduction of y_m for edge heave not exceeding 40% can be applied. If L is less than $2e$, the value of y_m can be linearly reduced using L/2e. Another parameter related to soil properties is the mound stiffness (k), for substructures in contact with swelling soils ranges from 400 kPa/m to 1500 kPa/m. Alternatively, a value of k equal to $100q'$ but not less than 1000 kPa m^{-1} may be used, where q' is the total building applied load over the slab plan area. For Melbourne basaltic clays, k should be taken as 400 kPa m^{-1} or $50q$, whichever is smaller. For substructures in contact with stable or shrinking soil, k should be taken as at least 5000 kPa m^{-1} (Standard Australia, 2011).

The parameters related to environmental factors being considered in this method is H_s, which is similar with AS 2870 Method and used to calculate e for centre heave in Eq. 17. For edge heave, a mound curvature factor denoted as W_f is multiplied to e (Figure 1b). This factor is also influenced by H_s with different functions for a normal profile having $H_s < 3$ (Site Class M, H_1 and H_2) and for a deep-seated movement profile having $H_s \geq 3$ (Site Class M-D, H_1-D or H_2-D) shown in Figure F2 of AS 2870-2011.

Parameters affecting stress conditions of soils considered in this approach are L, p and q to suffice the beam-on-mound equation in Walsh and Walsh (1986). The required stiffness, EI, is the output of the calculations corresponding to an amount of soil movement (y_m) to comply with the strength and serviceability limit requirements indicated in AS 2870. The experience of practitioners with this method was generally reported as satisfactory, with an exception of conservative dimensions of designed substructures calculated for edge heave scenarios. It is also important to mention that the shape of an edge heave mound and W_f has been subjected to scrutiny since it was released due to relatively higher calculated design requirements.

2.6. Mitchell Method

Mitchell et al. (1984) modified the methodology of Walsh and Walsh (1986), by calculating y_m for both centre and edge heave scenarios using $0.7y_s$. The required stiffness, EI, is a function of variables represented by

$$\frac{EI\delta}{EI\Delta} = f(x', L, B, p, q, C_M, \delta, \Delta, y_s, t, m), \tag{20}$$

where C_M is the support ratio calculated using finite difference explained in more detail in Mitchell et al. (1984), x' is the critical location assumed to be the quarter of L and t is the beam-on-mound exponent. The mound exponent, m, used to reflect the shape of the mound and its separation with the substructure is given by

$$m = \frac{1.5L}{D_{cr} - D_e}, \tag{21}$$

where D_{cr} is the critical depth given by

$$D_{cr} = \frac{H_s}{7} + \frac{y_m}{25}, \tag{22}$$

and D_e is the depth of embedment of an edge beam from the finished ground level.

The parameters related to soil properties, y_m, and environmental factors, H_s, affect the calculation of m through D_{cr}. These parameters are similar with the parameters used in AS 2870 method and Walsh method.

The parameters related to stress conditions of soils include L, p and q to suffice the equations presented in Mitchell et al. (1984). The required stiffness, EI, is the output of a calculation corresponding to a specific soil movement (y_m) to ensure relative substructure deformations are within the acceptable limits depending on the type of construction specified in Table 4.1 of AS 2870-2011.

2.7. PTI Method

Post Tensioning Institute (2008a,b) is based on an analysis of a plate resting on a semi-infinite elastic conducted by Wray (1979). This method uses equations derived from non-linear regression of parametric study to calculate critical moments for centre (M_{ch}) and edge heave (M_{eh}) given by

$$M_{ch} = \frac{1}{727}(L^{0.013}S^{0.306}D^{0.688}W^{0.534}y_m^{0.193})(Be^{1.238} + C) \text{ for centre heave,} \tag{23}$$

$$M_{eh} = \frac{S^{0.1}(De)^{0.78}y_m^{0.66}}{7.2L^{0.0065}W^{0.04}} \text{ for edge heave.} \tag{24}$$

where S is the spacing of beams of a substructure.

The primary parameters related to soil properties used in this method are PI, percent fine clay ($\%fc$), soil diffusion (α') and clay classification to determine the mineral type and configuration (Post Tensioning Institute, 2008b). Using these parameters, e is determined (Post Tensioning Institute, 2008a). However, there is a concern that e is underestimated due to high empirical dependence of this method (Durkee, 2000).

The parameter related to environmental factors being used in this method is the Thornthwaite Moisture Index, I_m (Karunarathne et al., 2016, Fityus and Buzzi, 2008, Thornthwaite, 1948). This parameter estimates the annual potential evaporation of an area using air temperature based on catchment data and controlled experiments. A positive value of I_m indicates an average annual runoff, while a negative value denotes water deficiency. This parameter is a useful method to calculate the water balance of an area, however, it does not consider the fluctuation of precipitation (i.e., total yearly annual precipitation, uniformity and distribution of precipitation, number of precipitation occurrence, duration of each occurrence and amount of precipitation per occurrence) that affects the behavior of shrink-swell movements. Furthermore, this parameter tends to underestimate evapotranspiration in cooler periods by relying only on air temperature and neglecting the effects of wind and relative humidity. In effect, y_s may have lower values for the centre heave calculation.

The parameters related to stress conditions of soil being used in the calculations of this method are L, S and W.

Since this method is based on the parametric study of Wray (1979), the applicability of this method may be limited to: (1) D between 450 to 750 mm, (2) S between 3.6 to 6.1 m, (3) W between 8.9 to 21.6 kN/m, (4) L equal to 7.3, 14.6, 29.3 and 43.9 m and (5) constant B equal to 200 mm and constant p equal to 0.6 kN/m. PTI Method cannot consider interior loads (i.e., load bearing walls, column loads and additional uniform loads). Further study is required to assess the applicability of this method beyond the limits mentioned.

2.8. Briaud Method

Briaud et al. (2011, 2016) proposed a new method to design stiffened raft substructures based on numerical simulations of change in soil suction considering several cities in the United States. This method was based on parametric simulations using finite element model of shrink-swell movements.

The parameters related to soil properties used in this method include soil modulus (E_s), a significant parameter to determine the soil-swell pressure. The decrease in E_s due to soil moisture increase and the increase in E_s due to soil moisture decrease have significant impact on the soil-structure interaction. Another parameter related to soil properties is I_{ss}, a parameter referring to the capacity of soils to retract when dry and expand when wet. Additional parameters related to soil properties are k_p or k_s. expansive soils have higher clay content with lower k_p or k_s, affecting the migration of fluid through the soil. This lag in water migration leads to spatial difference in soil moisture causing differential settlement due to shrinking and swelling.

The parameter related to environmental factors considered in this method is the time series of evapotranspiration (ET) measurements.

One of the factors to represent the soil condition in an area is the initial value of and change in soil suction, defined as the required energy for extracting unit volume of water in a soil medium (Saadeldin and Henni, 2016, Cameron, 1989). Suction is comprised of matric suction and osmotic suction. The matric suction is the negative pressure exerted by soils to equalise the soil moisture content in a soil block, which is in part related to the capillary phenomena due to the soil water surface tension. Matric suction varies with the inherent soil condition in the site. The osmotic suction is the pressure due to the dissolved solutes (i.e. salt) in the soil pore water. Osmotic suction influences the movement of water from a lower solute concentration region to a higher solute concentration region. The presence of suction in an unsaturated medium creates a highly transient environment, which is more prone to shrink-swell cycles (Sun et al., 2017, Karunarathne et al., 2013). Soil moisture is the water stored in the soil medium considering soil density, structure and fabric (Weerasinghe, 2018, Huat et al., 2005). The relationship between soil moisture and soil suction is represented by a Soil-Water Characteristic Curve or SWCC (McDowell, 2004). In designing substructure systems on expansive soils, the suction at the free field (u_{ff}) and the suction at the edge of a substructure system (u_e) are observed since their difference can determine the potential differential ground movements affecting raft substructures (Briaud et al., 2011, 2016). This factor determines the distorted mound shape due to shrinking and swelling of soils, presented in Figure 1.

This affects the change in u_{ff} and u_e. The soil volume change in the free field is more frequent since this area is exposed to precipitation and evaporation. The soil volume underneath the substructure system shrink and swell depending on k_p or k_s. Since expansive soils have high $\%fc$ and very low k_p or k_s, soil water dispersion underneath the substructures is delayed. Due to this, the difference of u_{ff} and u_e can determine the potential differential soil settlement, y_s, and the edge penetration distance, e.

The parameters related to stress conditions of soils are L, EI, p and q. A parametric study was conducted to identify influences of each parameter to the soil movements and raft substructure design. An automated design spreadsheet (TAMU-SLAB) based on Briaud method can be used to design raft substructures. Further verification of design calculations is necessary to investigate the applicability of this method.

A summary of the parameters considered in each method is grouped based on their relationship to soil properties, environmental factors and stress conditions (Table 2). These parameters affect the shrink-swell properties of expansive soils and subsequently the design of raft substructures for ULS and SLS.

3. Comparison of design methods

The comparison of design methods for raft substructures on expansive soils were performed through parametric analysis considering parameters related to soil properties, environmental factors and stress conditions, including substructure configuration, affecting the shrink-swell potential and subsequently influencing the structural design calculations. The approach undertaken for this investigation was divided into two phases, the first phase involved parametric calculations varying parameters related to soil properties, environmental factors and stress conditions for each method, and the second phase involved a non-parametric statistical analysis and clustering considering the individual results of each method and the overall results of all methods.

3.1. Parametric design comparison

The required beam depths were calculated using the design methods with varying parameters related to soil properties, environmental factors, and stress conditions including substructure configuration, presented in Section 2. The design methods have different approaches and inputs presented in Table 2. To effectively compare these design methods, the following assumptions for the relationships between different parameters used in each approach were correlated for effective comparison. A total of 90 scenarios were calculated and compared.

Climatic condition of an area influences the shrink and swell movements of soil. The active depth zone, H_s, represents the soil depth susceptible to shrink and swell movements influenced by both soil properties and environmental factors specified in Standard Australia (2011). To consider the effect of H_s, different values were varied in the calculation of beam depths based on the common values in Australia equal to 4.0, 3.0, 2.3, 1.8 and 1.5 m. The total suction change, $\Delta\bar{u}$, was 1.2 pF as stated in AS 2870 reflecting the average changes in suction. To effectively compare the design methods, the correlation between H_s and I_m was used given by the equation (Abdelmalak, 2007)

$$H_s = 1.387 + 0.939e^{\frac{-I_m}{24.843}}. \tag{25}$$

The calculated values of I_m for H_s of 4.0, 3.0, 2.3, 1.8 and 1.5 m were -22, -12, 1, 25 and 46, which will be used for the calculations using PTI Method. The parameter, C_w, was not considered in the comparison, however the support index, C, was directly calculated using Eq. 13.

Three ideal and hypothetical soil types were chosen with varying PI and LL. PI were assumed to be 30%, 45% and 60% and the LL were assumed to be 50%, 70% and 90%, which will be used for the calculations using WRI method and PTI method. For all soils, the % fine clay was assumed to be 70% and the % passing sieve no. 200 was assumed to be 100%. Using PTI method, the suction compression indices (γ_h) were estimated as 0.028, 0.077 and 0.133 pF^{-1}, respectively. These values of γ_h represent slopes between volumetric strains and matric suctions, which is comparable to I_{pt} reflecting the slope between vertical strains and matric suctions. The values of I_{pt} were assumed to be one-third of the values of γ_h since vertical strains are one-third of volumetric strains (Fityus 2004). Hence, the calculated values of I_{pt} are 0.0093, 0.0257 and 0.0443.

The lengths, L, of substructures were chosen to be 6 m, 12 m and 18 m with a constant width, B, of 6 m to represent different aspect ratios. The spacing and the width of the ribs of the hypothetical raft substructure were assumed constant, which are equal to 1.2 m and 0.11 m. The uniform load, p, was 2.5 kPa and 6.5 kPa, while the line load, q, was taken as constant equal to 10 kN/m. An articulated masonry veneer was assumed to be the type of construction.

3.2. Statistical analysis

The main aim of the statistical comparison of the design methods is to determine the central tendency and dispersion considering all the considered approaches. To consider the central tendency of the values of beam depths for each scenario, the result of each method were compared to the overall calculated median ($\overline{M}$) and the overall calculated third quartile ($\overline{Q_3}$) considering all design methods depending on the varying parameters related to soil properties, environmental factors and stress conditions. To consider the dispersion of calculated values of beam depths using the design methods, values of interquartile ranges (IQR) considering varying depths of H_s, values of PI and LL, magnitude of p and span od L. A total of 90 combinations were calculated and compared.

Non-parametric statistics was used in the analysis since the probability distributions of the calculated beam depths using different design methods might not follow a normal probability distribution (Gibbons and Fielden, 1993, Saltelli and Marivoet, 1990, Siegel, 1957). A five-number summary of datasets was used, which is comprised of the lower limit (LL), the first quartile (Q_1), the median (M), the third quartile (Q_3) and the upper limit (UL). This five-number summary is similar to a common box plot shown in Figure 2. Outliers were determined and then neglected through the upper limit ($UL = Q_3 + 1.5IQR$) and the lower limit ($LL = Q_1$ - $1.5IQR$) of each design method, where IQR is calculated as

$$IQR = Q_3 - Q_1. \tag{26}$$

The design methods were assessed by calculating the normalised cumulative absolute difference ($\sum |M - \overline{M}|$) between the calculated beam depth of a specific combination and $\overline{M}$ considering all the design methods. Similarly, a more conservative comparison of the design methods were performed by calculating the normalised cumulative absolute difference ($\sum |Q_3 - \overline{Q_3}|$) between the calculated beam depth of a specific combination and $\overline{Q_3}$ considering all the results of the design methods of a specific combination. This comparative analysis shows which methods are closer to the overall $\overline{M}$ and $\overline{Q_3}$ for specific scenarios.

The variability of each dataset was reflected using IQR. The calculated values of variability for each combination were clustered based on H_s, PI and LL, p and L. The different clusters are: Cluster 1 presents the sole variation due to the active depth zone, H_s; Cluster 2 presents the combined effect of variation due to H_s and soil reactivity; Cluster 3 presents the combined effect of variation due to H_s and p; and Cluster 4 presents the combined effect of variation due to H_s and L. The comparison of design methods through input variation aims to further understand the design models used in practice and to determine critical inputs that affect design variability.

4. Results of the comparison

The review and parametric comparison of AS 2870, Walsh, Mitchell, BRAB, Lytton, WRI, PTI and Briaud method calculated different values and range of beam depths. The parametric comparison used statistical parameters to show the range of values of calculated beam depths depending on varying inputs for each design method. This also determine the design method closer to $\overline{M}$ and $\overline{Q_3}$ reflecting the central tendency of the considered desifn methods in this paper. Furthermore, the variability of the calculated beam depths were quantified using IQR and was clustered depending on values of H_s, PI and LL, p and L.

The calculated beam depths ranged from 140 to 1750 mm shown in Figure 3. Neglecting outliers using the limits UL and LL, the acceptable range for the calculated beam depths were 300 to 815 mm.

The design methods were compared by calculating the normalised cumulative absolute difference ($\sum|M-\overline{M}|$) between the calculated beam depth of a specific combination and $\overline{M}$ considering all the design methods. The comparison observed that PTI method had calculated beam depths with most proximity to the overall median, $\overline{M}$, with a value closer to zero considering all design methods (Figure 4). This method is highly empirical, which is dependent on I_m, e and the limits stated in Wray (1979). The beam depths calculated by this method may have been affected by the limits (450 to 750 mm) in Wray (1979) that falls mostly around the median values (350 mm to 710 mm). These values proximity to $\overline{M}$ may be due to combined effects of the empirical limits, the eventual underestimation of I_m during cooler periods, and the underestimation of e. Further validation of the applicability of PTI method to designs outside the specified limits should be investigated to assure that the calculated beam depths were due to rational relationship and not due to the combined effects of assumptions, estimations and limitations of the approach. Alternatively, a method based on physical models (i.e., rational methods for infiltration of water through expansive soils) will be easier to validate and will give more confidence to designers and researchers on the rationale of calculated beam depths.

A more conservative comparison of the design methods were performed using the normalised cumulative absolute difference ($\sum|Q_3-\overline{Q_3}|$) between the calculated beam depth of a specific combination and $\overline{Q_3}$ considering all the design methods. The comparison observed that Lytton and Briaud methods have beam depths with most proximate to $\overline{Q_3}$, with values closer to zero considering all design methods (Figure 5). Unlike PTI method, Lytton and Briaud method are mathematical approaches. Lytton (1970), Lytton et al. (1985) proposed a curved mound using elastic mathematical models of beam and slab based on the beam-on-mound equation (Equation 17). However, this method has some assumptions such as an infinitely-stiff concrete substructure, which was improved by Walsh and Walsh (1986). Briaud method is a new method to design stiffened raft substructures based on numerical simulations considering change in soil suction due to parameters related to environmental factors (e.g., precipitation and evapotranspiration). However, further verification of this method is necessary to investigate the applicability of this design method.

Calculations using WRI and BRAB method obtained larger dimensions of beam depths, specifically for more expansive soils (i.e., Class H_1 and H_2) with longer L, where some of the results were considered as outliers. This supports the findings of Abdelmalak and Briaud (2006), where these design methods calculated overly conservative beam depths, specifically when L is longer due to the direct proportionality of the cantilever length, l_c, to the required substructure EI affecting design calculations.

AS 2870, Walsh and Mitchell methods had beam depths falling in the less conservative range based on the range of beam depths calculated (Figure 3). AS 2870 method assumed a common p applied in substructures, where higher p cannot be considered in the design process due to predefined interpolation considering common dead and live loads applied to single-detached dwellings. Thus, the change in p for the parametric study did not affect the calculated beam depths.

Clustering of IQR was performed to investigate the effect of variation of H_s, PI and LL, p and L. Four different clusters were identified; these were Cluster 1 that presents the sole variation due to H_s, Cluster 2 that presents the combined effect of variation due to H_s and site reactivity, Cluster 3 that presents the combined effect of variation due to H_s and p, and Cluster 4 that presents the combined effect of variation due to H_s and L. Cluster 1 reflects the effect of H_s to IQR (Figure 6a). Locations that are drier with deeper H_s had higher IQR of calculated beam depths (e.g., Adelaide and Hobart). Cluster 2 shows the effect of H_s and soil to IQR (Figure 6b). The clustering observed that beam depths with higher PI and LL (i.e., Class H_1 and H_2) had larger IQR than soils with lower PI and LL (i.e., Class M). Cluster 3 reflects the effect of H_s and p to IQR (Figure 6c). This shows that calculations with higher applied uniform load (w=6.5 kPa) had larger IQR than lower applied uniform load (w=2.5 kPa). Lastly, Cluster 4 presents the effect of H_s and L to IQR (Figure 6d). Calculations with longer span, L, had higher IQR. In summary, a direct proportional relationship was observed among IQR and H_s, PI and LL, p and L. The findings of this variability analysis suggest that when higher input values of H_s, PI and LL, p and L were specified, it is critical a design method that best applies to a specific project since selecting a selecting a design method that does not support the limit assumptions in the local site will lead to over or underestimation of beam cross sections.

5. Conclusion

Thorough review of different design methods with varying approaches were performed. Different parameters related to soil properties, environmental factors and stress conditions were discussed and varied for parametric comparison of the current design methods. The design methods considered were the Australian Standard 2870-2011 or AS 2870 method, Walsh method, Mitchell method, Building Research Advisory Board or BRAB method, Lytton method, Wire Reinforcement Institute or WRI method, Post-Tensioning Institute or PTI method and Briaud method.

The parametric comparison of the design methods determined the central tendency and dispersion considering all the considered design methods. The parametric analysis considered: the varying hypothetical active depth zone, H_s; the plasticity and liquid limits, PI and LL; the magnitude of area load p; and the lengths of substructure. A total of 90 combinations were calculated and compared. The calculated beam depths of each method were compared to the overall calculated mean, represented by $\overline{M}$, and the overall calculated third quartile, represented by $\overline{Q_3}$, considering all design methods depending on the varying parameters. The dispersion of calculated values of beam depths using the design methods was represented using interquartile ranges, IQR.

The values of the calculated beam depths ranged from 140 to 1750 mm, which narrowed to a range from 300 to 815 mm when outliers were neglected. PTI method had values of calculated beam depths closest to $\overline{M}$, however, further validation is necessary due to high empiricism. Alternatively, Lytton and Briaud methods were more conservative design options with values of calculated beam depths closest to $\overline{Q_3}$. The variability of calculated beam depths, which were presented using the ranges of IQR, were higher when the location had deeper H_s, higher PI and LL, greater p and longer L.

In summary, designers and practitioners shall consider more conservative approach when designing substructures with deeper H_s, higher PI and LL, greater p and longer L since the calculated values of beam cross section have wider range than more stable sites with smaller and lighter structures. Approaches such as Lytton method, Briaud method or PTI method may be advisable, however, these design methods still have opportunities to be improved through additional experiments and field observations of built substructures. Future considerations to improve design methods for raft substructures on reactive soils include approaches based on simplified multiphysical derivations (i.e., hydro-mechanical) using practical parameters commonly used in practice.

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Table 1: Site classification dependent on the surface characteristic movement, y_s, presented in AS 2870-2011 Standard Australia (2011).

Site class	Soil foundation	y_s in mm
A	rock and sand	0
S	slightly reactive silt and clay	0 – 20
M	moderately reactive silt and clay	20 – 40
H_1	highly reactive clay	40 – 60
H_2	very highly reactive clay	60 – 75
E	extremely reactive clay	> 75
P	filled, soft silt or clay, loose sands, sandslip, mine subsidence, collapsing	varying

Table 2: Summary of parameters related to soil properties, environmental factors and stress conditions affecting the shrink-swell potential of expansive soils.

Notation	Parameter	Design Methods
Parameters related to soil properties		
$H_s{}^a$	Active depth zone	AS 2870
y_s	Surface characteristic movement	AS 2870, Walsh, Mitchell
I_{ss}, $I_{pt}{}^b$	Shrink-swell and instability index	AS 2870, Briaud
h, N	Layer thickness and number of layers	AS 2870
e^a	Edge penetration distance	Walsh, Mitchell, PTI
$W_f{}^a$	Mound curvature factor	Walsh
y_m	Differential mound movement	Walsh, Mitchell, Lytton
m	Mound exponent	Mitchell, Lytton
$D_{cr}{}^a$	Critical depth	Mitchell
PI, PI_{eff}	Plasticity and effective plasticity index	BRAB, WRI
PVC	Potential Volume Change reading	BRAB
C, C'^a	Support index and modified support index	BRAB, Lytton, WRI
$1 - C^a$	Soil-climate support index	WRI
$\%fc$	Percent fine clay	PTI
α'	Soil diffusion	PTI
k_p, k_s	Permeability or hydraulic conductivity	Briaud
Parameters related to environmental factors		
$\Delta\bar{u}$	Average suction change	AS 2870
C_W	Climatic index	BRAB
I_m	Thornthwaite Moisture Index	PTI
u_{ff}	Suction at free field or open grounds	Briaud
u_e	Suction underneath substructures or covered grounds	Briaud
ET	Evapotranspiration time series data	Briaud
Parameters related to stress conditions		
p, q, q'	Uniform area and line loads	All methods
r	Total slab load	Lytton
B, D, L	Width, Depth, Length of substructure	All methods
α	Lateral restraint factor	AS 2870
G, k	Soil shear and mound stiffness	Walsh, Mitchell, Lytton
D_e	Embedment Depth	Mitchell
t	Beam-on-mound exponent	Mitchell
k_l, k_c	Design factors	WRI
l_c	Cantilever length (empirical)	WRI
E, E_c	Concrete elasticity	All methods
I	Moment of inertia	All methods

[a]This parameter is also related to environmental factors.
[b]This parameter is also related to stress conditions.

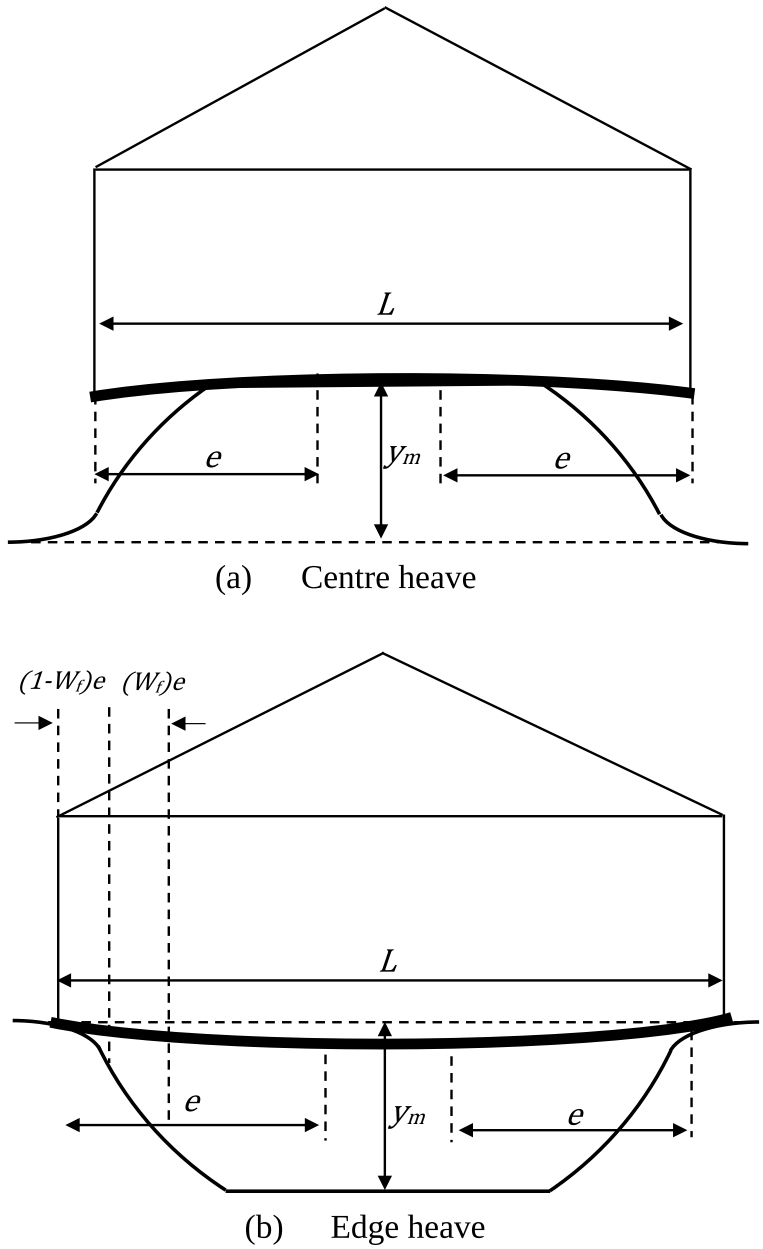

Figure 1: Mound for (a) centre heave and (b) edge heave to represent the soil shrink-swell ground movement of Walsh and Walsh (1986).

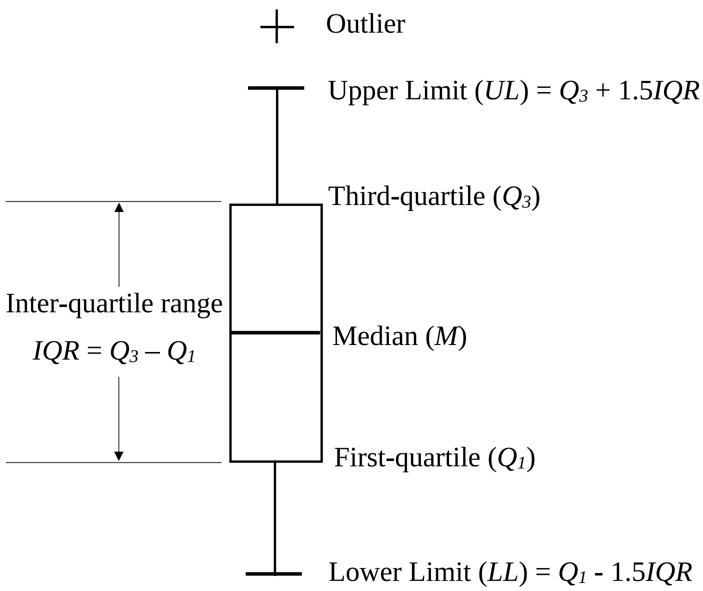

Figure 2: An example of a five-number summary of a dataset showing the lower limit LL, the first quartile (Q_1), the median (M), the third quartile (Q_3) and the upper limit (UL).

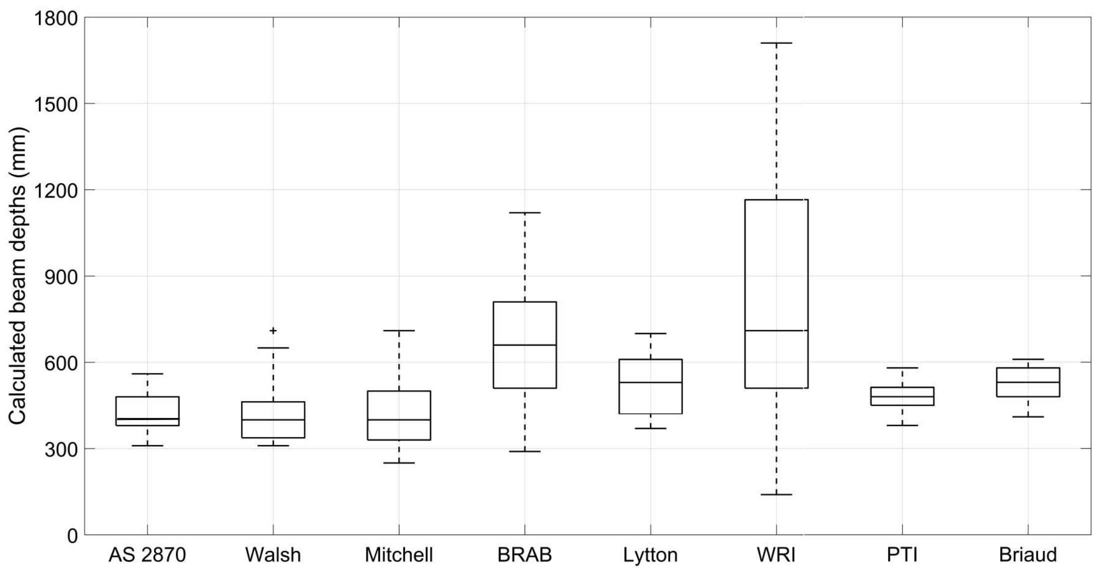

Figure 3: Boxplots for the calculated beam depth (in mm) of the design methods showing the minimum, first quartile, median, third quartile and maximum values.

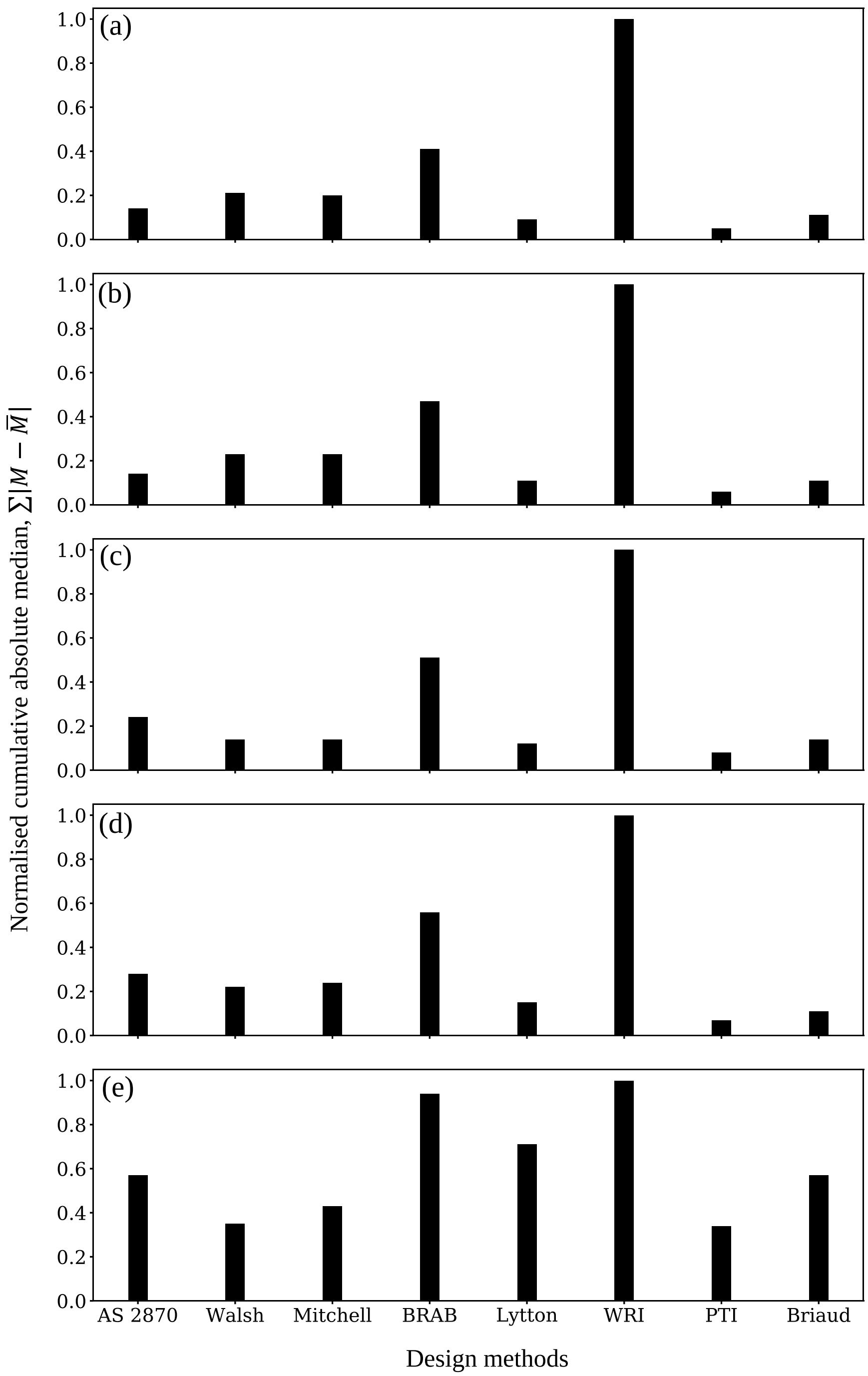

Figure 4: Cumulative absolute difference of calculated beam depths of each design method to the overall calculated $\overline{M}$ for (a) H_s=4.0 m, (b) H_s=3.0 m, (c) H_s=2.3 m, (d) H_s=1.8 m and (e) H_s= 1.5 m.

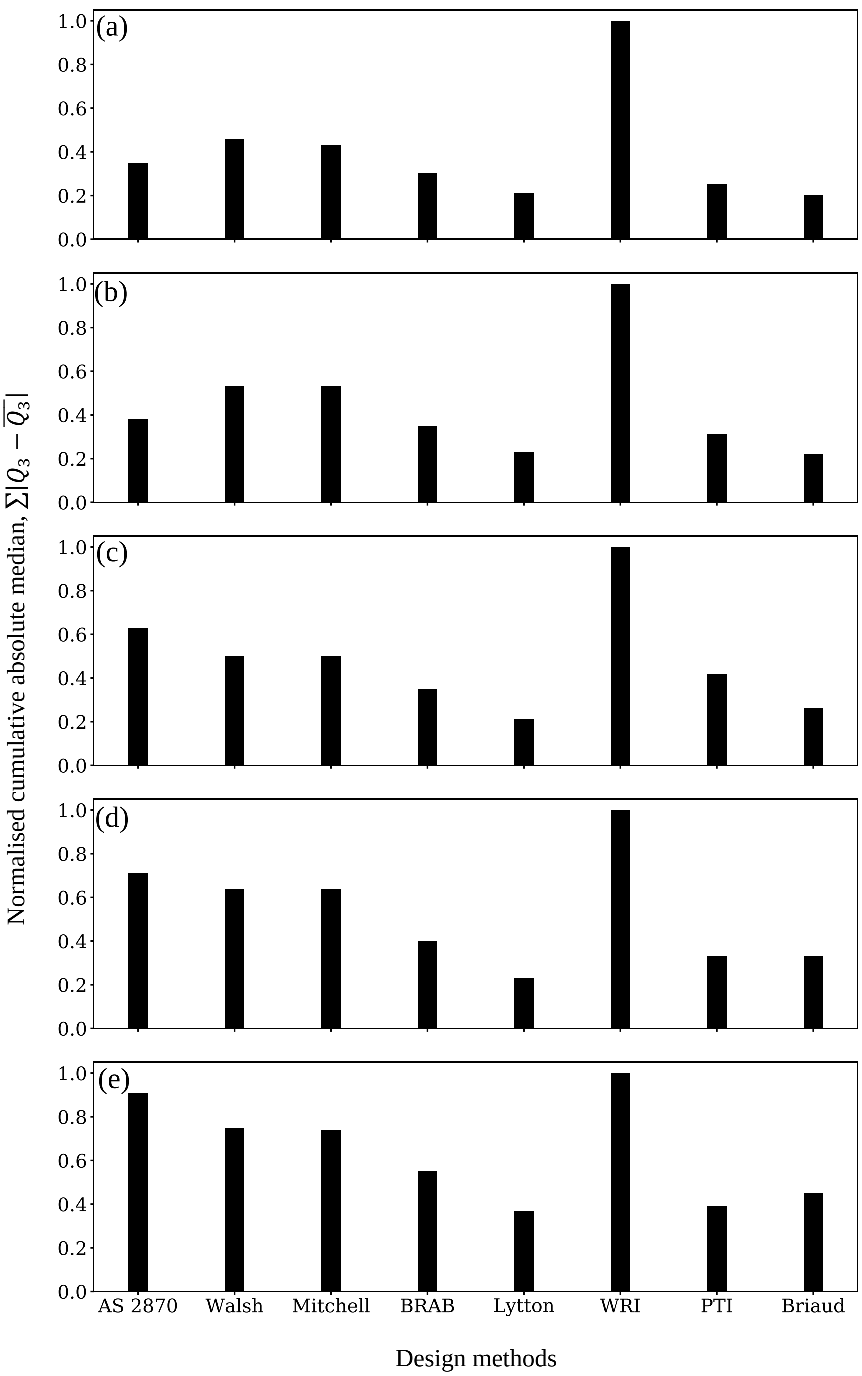

Figure 5: Cumulative absolute difference of calculated beam depths of each design method to the overall $\overline{Q_3}$ for (a) H_s=4.0 m, (b) H_s=3.0 m, (c) H_s=2.3 m, (d) H_s=1.8 m and (e) H_s= 1.5 m.

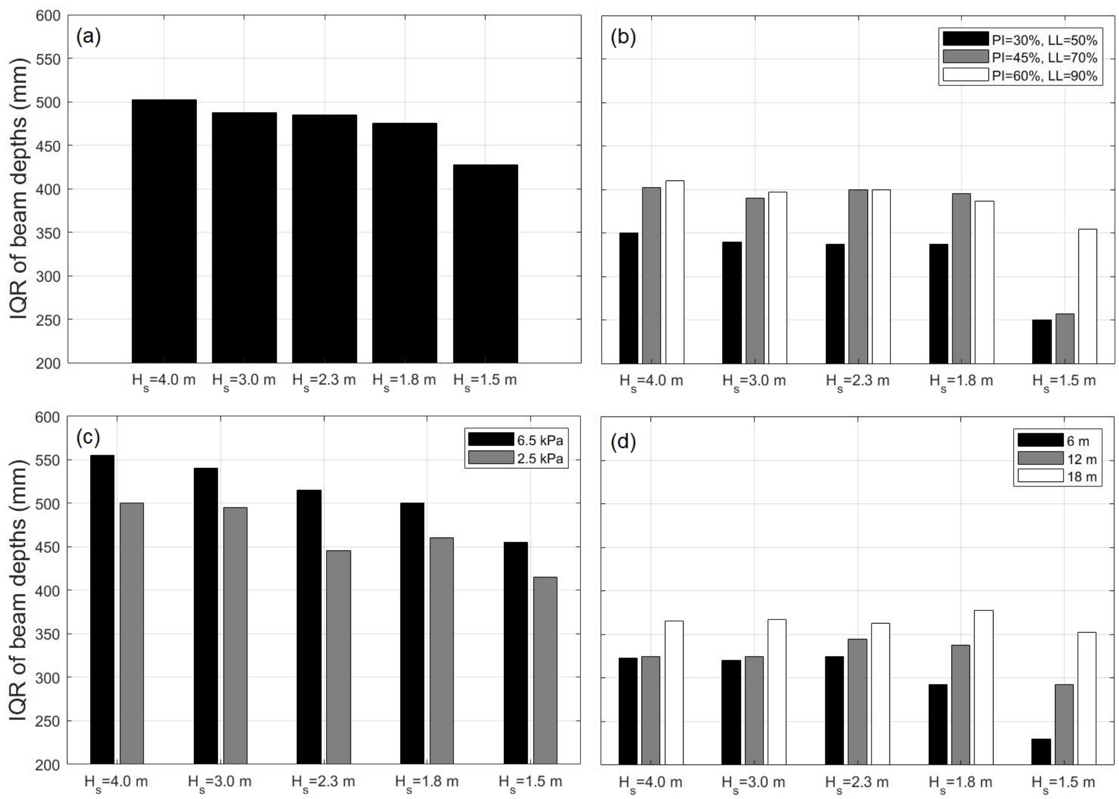

Figure 6: Comparison of interquartile range (IQR) depending on (a) depth of H_s, (b) H_s, PI and LL, (c) H_s and p and (d) H_s and L.