UNIVERSITY OF SOUTHERN QUEENSLAND

INVESTIGATION INTO RELOCATABLE HOME DESIGN AND CONSTRUCTION

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ABSTRACT

Relocatable homes (also known as manufactured or prefabricated homes) exist all over Australia, especially in Queensland where relocatable home parks are popular in the form of tourist parks and elderly estates, and they are also commonly found in mining towns where the demand for new housing exceeds the limitations of local builders. Although not as popular here in Australia as they are in the USA, they are however becoming more and more common all around the globe as they are ideal for certain lifestyles and budgets. The homes undergo significant rough behaviour in their journey from their construction to final positioning including loading on and off of the transporting truck, wind loads and general jolting on the roads. Even once in place, cracking can often occur due to foundation movements. This thesis outlines an analysis of the dry stack block method.

Key words: Relocatable homes, wind loads, foundation movement, dry stack block footing, short pier footing

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CHAPTER 1 – Introduction

The substructure elements of any buildings including manufactured homes across Australia and throughout the world, have long posed problems for engineers aiming to minimise defects like deflections and cracking on the structure. Due to certain unavoidable issues such as mining subsidence or expansive soils in the foundation, or other natural causes such as wind/snow loads or earthquakes, the design of the support system, which spreads the structure's loads over a large ground area in the foundation, becomes vital in prolonging the existence of the structure.

It is the author's opinion that the support system is by far the most important element to any structure. The support system is the combination of footings and piers that support the home. Whether it forms the basis of a house, high rise building or bridge, it is the support system that will bear the full load of the structure and its strength will determine the overall safety. When it comes down to people's lives being at stake, its importance is second to none and special attention is often drawn to the engineers involved to ensure the required safety standards are met.

Concrete footings can be separated into shallow or deep footings, according to their depth embedded into the soil foundation. Shallow foundations include spread footing, mat-slab, slab-on-grade and pad foundations and are generally anything less than a metre deep. They are used to transfer the building's loads to an earthen layer close to the surface, whereas deep foundations such as piles, piers, drilled shafts and caissons transfer their loads to subsurface layers at greater depths. The structural design of the footing is based on several aspects including design loads, soil conditions and constraints on site. Houses and other small structures will always have shallow footings provided there are no constraints preventing their use. The advantages of using shallow footings are that they are cost effective, simple to construct, are made mainly of concrete and generally doesn't require any expertise in labour. Deep footings can be more advantageous as they are subject to less torsion, moment and pullout forces, can be used on irregular ground surfaces, and suffer less effects of foundation settlement. As the relocatable homes of interest use only shallow foundations, they will form the focus for that section of this thesis.

The pier and ground anchor support system has been the common and accepted relocatable homes anchorage and support system. The most common pier types are steel jack stands, hollow concrete blocks stacked one on top of the other, and short piers. The focus will be on the analysis of the dry stack blocks.

The focal points of this research are illustrated in Figure 1.1.

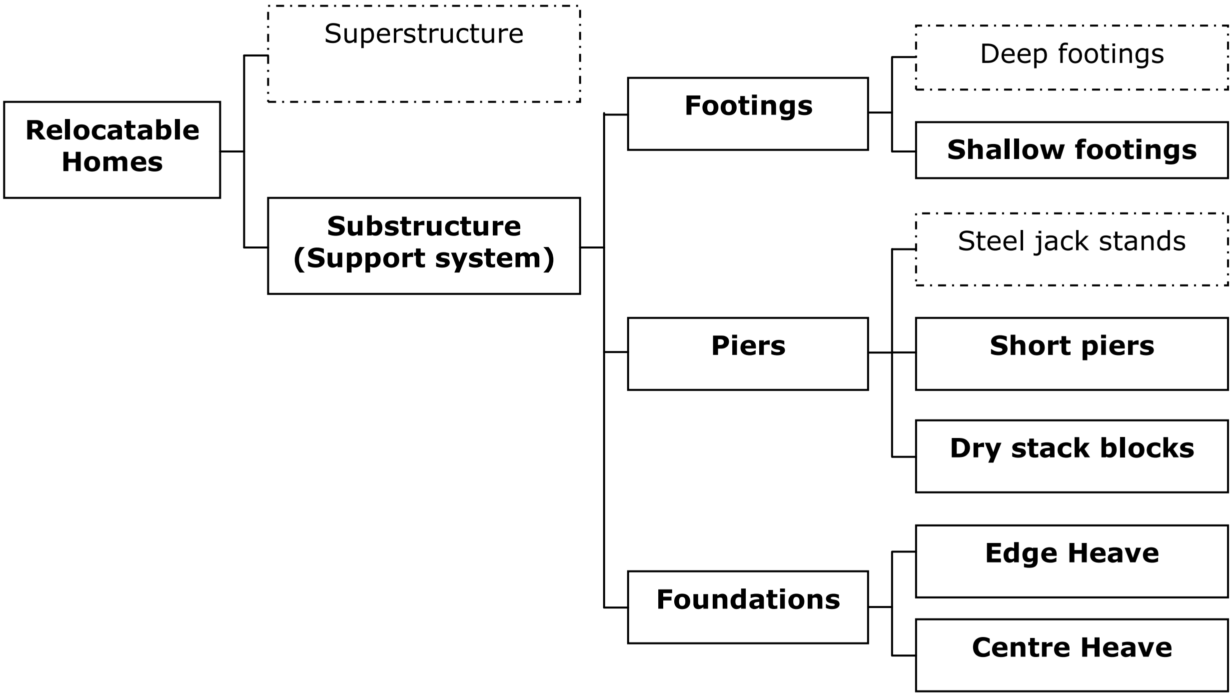

Figure 1.1 - Flowchart Illustrating the Focus of the Thesis

My current employer Hunt Michel & Partners Pty Ltd were this year given a job of approving a set of drawings developed by a company who wishes to continue to manufacture and sell relocatable homes similar to what is shown in Figure 1.2. The company has been using dry stack blocks as their support system. As the typical pier system across Australia appears to be short concrete, timber or steel columns, the author felt it necessary to determine the quality of performance of the dry stack blocks with regards to maximum heights possible while maintaining its structural safety. It is also of particular interest, the tie down and bracing used with the dry stack support system.

Figure 1.2 – Typical Manufactured Home (Norfolk Homes, 2010)

Although a cost comparison of dry stack block with traditional methods could be taken into consideration for the company wishing to build the homes, it will not be a part of the research. Design and construction aspects for transportation of relocatable homes including loading and unloading, and crainage will also not be addressed, meaning these remain available topics for future research.

Finally, recommendation tables are proposed in the later chapters should time permit. This recommendation aims to achieve a high quality design that could be used sufficiently throughout Australia.

CHAPTER 2 – Literature Review

2.1 Home Classification

Manufactured Homes (Residential Parks) Act (2003) defines a manufactured home as:

1. A structure, other than a caravan or tent, that-
    a) has the character of a dwelling house; and
    b) is designed to be able to be moved from one position to another; and
    c) is not permanently attached to land.
2. A manufactured home does not include a converted caravan.

Manufactured homes today range widely in quality, price, size and style, with the most limiting characteristics being size and shape. In the early years of the manufactured home, they were being designed to market the lower class of citizen but now can price upward of $250,000. Most are still located in "parks", which are specially designed and situated for their purposes (Sigfusson, 1997).

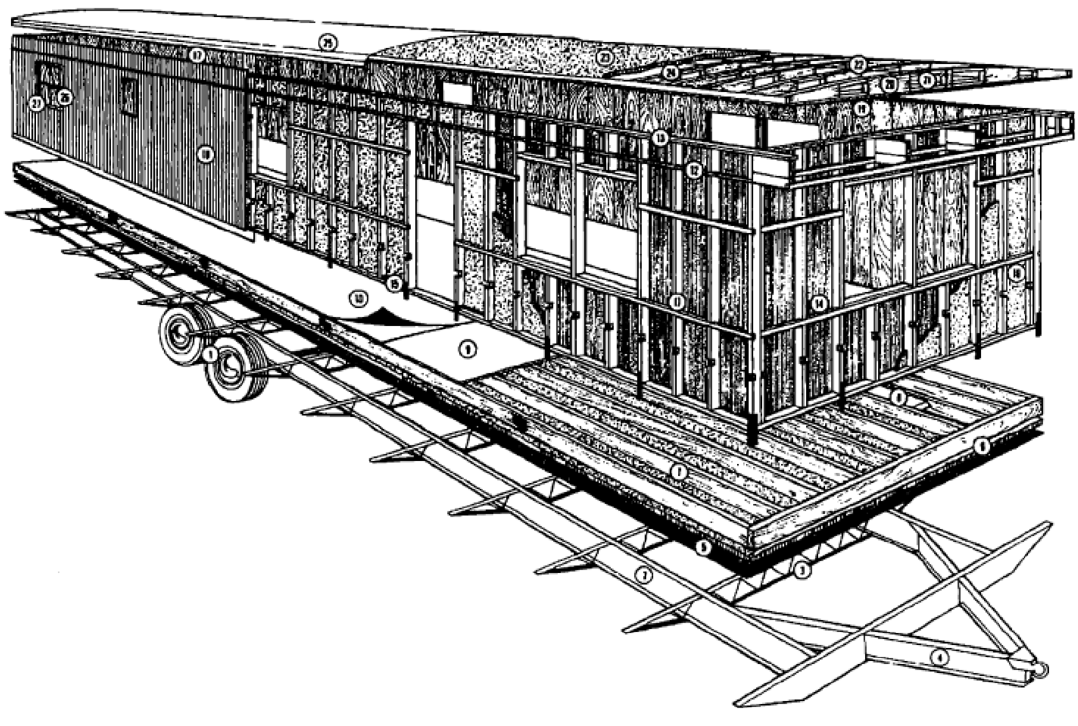

Figure 2.1 - Exploded View of a Manufactured Home (Sigfusson G., 1997)

2.2 Preface

In recent years, the popularity of manufactured homes has increased with a public realisation that they are a very affordable option to becoming a homeowner, as well as the mining boom. With this increasing popularity, one would assume that there would be an equivalent increase in the research of manufactured homes, whether it be a cost analysis, structure design or soil effects however this has appeared not to be the case.

Several manufacturers across the USA have produced handbooks (MHRA, 2002) detailing such things as guideline to foundation and support systems, and floor frame assemblies, however in Australia the author has found that we lack these important guides that are readily available with other construction materials.

The designs in Australia are based around what has proven to work for years rather than actual engineered designs. There is a major problem with this level of thinking. At the base level, homes in Australia are designed to bare wind loading summarised in the Australian Standards (AS1170.2, 2002), from which an importance factor is required for calculation. A manufactured home park would be of importance level 2 (BCA, 2007), which means that it should be designed based on a wind speed from a 1 in 500 year event. It is of the author's opinion that perhaps manufactured homes in Australia have lasted the test of time simply because they have not encountered such a strong wind event, and not because they are adequately designed. As the importance levels are a function of both hazard to human life and public impact of building failure, it is therefore vital that current design and construction methods be examined and checked to ensure that they will perform safely during such an event.

2.3 Current Construction

Manufactured homes have to be designed and constructed with the ability to be transported, and because of this they are usually single or double-sectioned. The major difference between manufactured homes and modular homes is the metal chassis, which is a permanent part of the manufactured home. The chassis must be designed for transport and must support the entire load of the superstructure at the points of wheel assembly. The wheels and chassis are removed once installed on site.

The manufacturer of the homes (Park Homes, 2010) offer several standard construction features:

- Heavy-duty engineered steel chassis
- Engineered footing plans
- Built to comply with local standards
- Council-approved construction
- Transit insurance to site
- N1-4 structural timber frame (depending on location)
- 10mm plaster board lining
- Colorbond roofing, guttering, flashing and capping
- Engineered tie-downs
- Manufactured engineered timber truss system

With the option for several extras such as:

- Verandah area
- Air-conditioning, gas heating, wood fires etc
- Tiled roof
- Raked ceiling
- Eaves

The metal chassis is the same dimension of the floor system of the home. The typical dimension that will be used in this thesis will be 12.5m long by 7m wide by 3.5m high, which is a double-section that is built in halves for transportation and connected on site (each section is 3.5m wide). As they can be built in one, two or occasionally three sections, a wide variety of floor plans are available.

The home is built from the inside out. Once the floor panels are in place, the interior walls and all appliances are next, with the exterior walls last to be erected. After they are in place, electrical wiring and insulation can be installed and finally, the exterior siding is applied. The walls are generally made from 2″ x 4″ timber members, with thinner members available for interior walls if space is an issue. The roof, which is built separate to the rest of the home is then hoisted in place and bolted into position. Apart from the chassis and wheel assembly, these homes are constructed almost identical to regular housing, with the exception that they are built in a factory meaning there are no weather delays meaning they can be completed faster, and that they are built in a controlled environment (Sigfusson, 1997).

The height of the manufactured home is limited as well as the width due to transportation issues (see Section 2.6).

The full assembly can take as little as a single day up to a few weeks to complete depending on the manufacturer and size of the home. Once the home is finished it is transported to location on the back of a flatbed truck, where it is either rolled into position, or in difficult circumstances (such as the last home in a cul-de-sac) craned into position. Its connection to the foundation depends entirely on the footing system used. This thesis will discuss the tie-down methods of the dry stack block support system.

2.4 Foundations

The foundation is loosely defined as the soil that supports all components of the support and anchoring system (that might include such features as piers, footings, slabs, walls, ties, anchoring equipment, ground anchors, or any other material or equipment) that supports a home and secures it to the ground (Porter, 1996).

There are many types and varieties of manufactured homes now produced, with an equal number of varieties of support systems (MHRA, 2002). No foundation system is the single best, however there is a way to determine the best solution for any location depending on such things as soil conditions, available funds, wind zones, climatic zones etc. This thesis aims to determine in which situations dry stack blocks are more beneficial, and which situations short piers are. If neither method proves appropriate, it will be mentioned accordingly.

2.4.1 Expansive Soils

Expansive clays are classified as those which respond to a change in moisture level by changing in volume – shrinking as well as swelling. They are also known as reactive or swelling clays (Krack, 2008). There are many factors that cause either shrinking or swelling of soils, as well as laboratory and field procedures available to identify expansive soils and predict volume changes, however this thesis will not go into any depth regarding these topics. It will however consider the conditions edge heave and centre heave conditions caused by expansive soils.

2.4.1.1 Shrink Swell Index

The shrink-swell test is a measure of the reactivity of the soil. It is predominantly used in Australia, particularly AS2870 Residential Slabs and Footings Code, which enables vertical surface movement to be calculated using the equation:

$$y_s = \frac{1}{100} I_{pt} \Delta u \Delta dh$$

Eqn 2.1

where y_s = characteristic surface movement

I_{pt} = effective instability index including allowance for lateral restraint and vertical load

Δu = change in suction

Δh = thickness of soil layer in metres

From there, the soils are classified according to their degree of reactivity as per the following table:

Classification	Degree of Reactivity	y_s (mm)
S	Slight	< 20
M	Moderate	20 - 40
H	High	40 - 70
E	Extreme	> 70

Table 2.1 - Soil Classification (AS2870, 1996)

As these values are required for footing design in Australia, it is the author's intention to supply a foundation design recommendation for each of the soil classifications.

2.4.1.2 Structure–Soil Interaction

Expansive soils can have drastic effects on anything they surround, and likewise the structure has an influence on the soil characteristics. By covering the soil with an impervious member, the infiltration and evaporation cycle is interrupted. Moisture contents heads towards equilibrium and becomes relatively stable under the structure's centre, while the edges are still subject to seasonal weathering.

The moisture will of course depend on whether or not the soil was wetter or drier than equilibrium prior to construction. The site's climate is a major factor controlling the magnitude of the differential soil movement, and thus will determine the wet and dry soil suction profiles (Fig. 2.2).

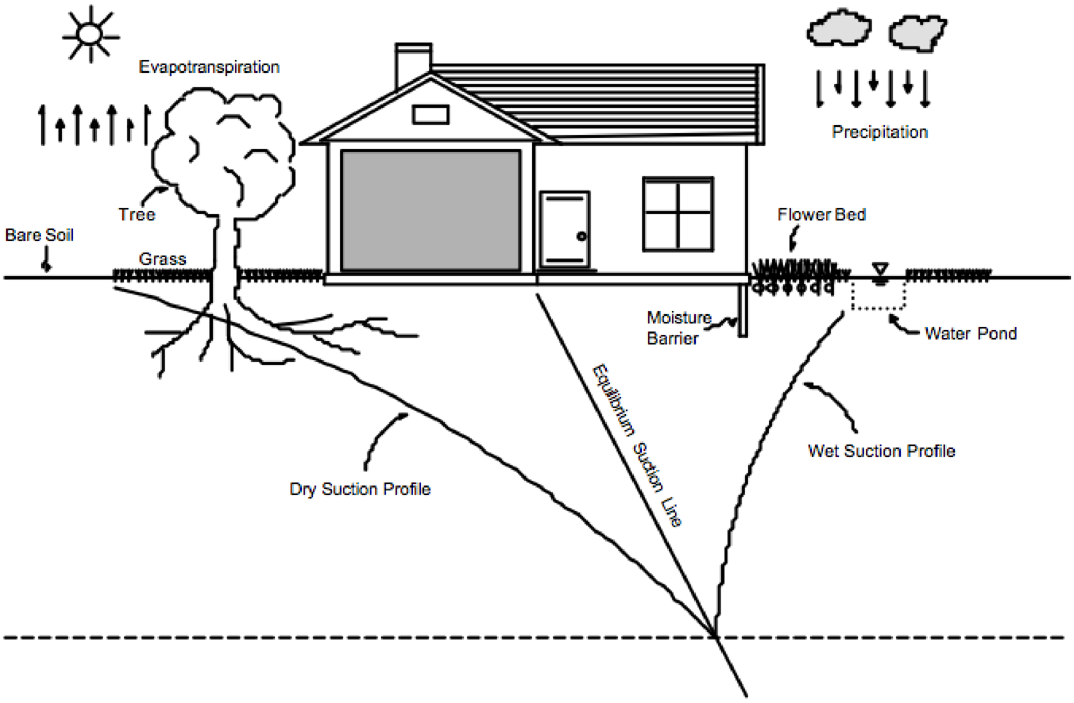

Figure 2.2 - Climate Effects on a Home Foundation (Bulut R., 2001)

If the structure's foundation is on reactive soils, the soil responds with a change in volume, and is responsive not only to structural loading, but change in soil conditions. Expansive soils swell when they absorb moisture from the environment and shrink when they lose moisture to the environment (Bulut, 2001). The moisture is not uniformly distributed across the foundation's underlying soil and thus results in differential soil movement. It is this movement that causes major distresses in the slab foundations.

Lightly loaded structures such as houses and pavements have been affected by reactive soils all over the world (Fig 2.3).

Figure 2.3 - Global Distribution of Expansive Soils (Bulut R., 2001)

When a lightly loaded structure is built on reactive soils, two types of distortion cases exist, known as centre-heave and edge-heave (Fig 2.4). If the structure is constructed on a soil, which is drier than equilibrium moisture content, moisture will tend to travel to the centre and a centre-heave profile. In this case, the edges tend to move up and down with seasonal moisture fluctuations. If the site is wetter than equilibrium moisture content, the moisture tends to travel outwards, resulting in the edge-heave profile, however this is usually only short-term, with the long-term profile heading towards the centre-heave shape (Krack, 2008).

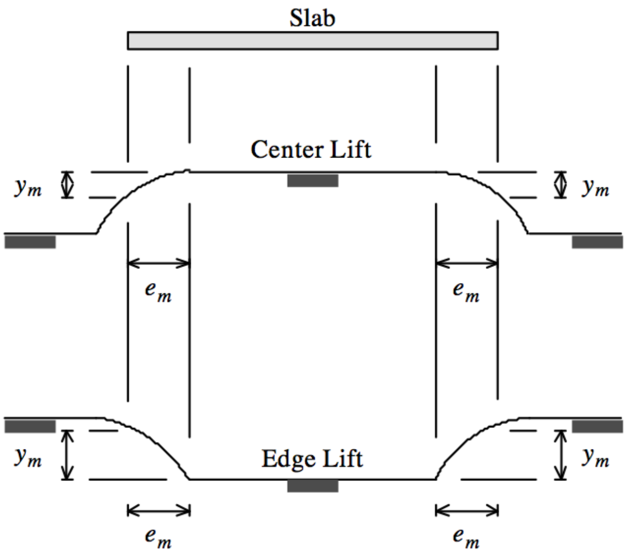

Figure 2.4 - Soil Distortion Cases (Bulut R., 2001)

Regardless of whether or not the long-term mound profile fully develops, the moisture under the centre of the structure will remain fairly constant, while the edge moisture will fluctuate seasonally. The edge distance that the movements occur can be calculated using the following equation:

$$e = \frac{H_s}{8} + \frac{y_m}{36}$$

Eqn 2.2

where Hs = depth of design suction change

y_m = differential mound movement = $0.7y_s$ (mm)

2.4.2 Structural Cracking

There are engineering and standard building tolerances where most minor cracks may not be defects. These are termed hairline cracks and are considered only slight damage (Kozlowski C. & Mazzone B. 2002). Cracking is the most common serviceability issue in most structures, and sizes can vary dramatically in length, width and depth with most developing at window and door openings. As a general rule, widths of greater than 5mm should be of great concern to the homeowner.

Cracking can result from a number of influences including:

- Thermal movement of building elements.
- Moving/sagging of supports.
- Moving/sagging of cantilevered elements.
- Large tree roots drawing moisture out of the soil.
- Foundation movement.
- Inadequate tie-down/anchor system.

When examining a typical defected building, all of these influences may be affecting the cracking, however it is the foundation movement and tie-down/anchor system that will be of particular interest in this thesis.

2.5 Transportation Issues

Manufactured homes can be transported very long distances, between states even, with the only factor affecting the transport length being cost. A problem with relocating to a remote location is that there is a lack of tradesmen specifically trained in the area in manufactured home assembly. If you know of qualified tradesmen in the area you are wishing to relocate to, you have the option of travel via road on the back of a truck which is the preferred method, however other common forms of transport include train, barge or in extreme cases, helicopter.

Relocatable homes are designed to the safety and strength requirements of the Building Code of Australia, with extra reinforcing in case of several moves during their lifetime. Each home is built on a heavy-duty galvanised chassis designed to withstand unsealed roads, difficult undulating surfaces, high winds and stress from lifting and lowering by use of a crane (see Figure 2.5) or hydraulics (Modular Home, 2011).

Figure 2.5 – House Craned into Position (MHRA, 2002)

With difficulties being raised during transportation, such as load limits and sight restrictions, special consideration has to be taken to ensure the home is transported safely and legally. Department of Transport and Main Roads (2010) state that for a truck of excess dimension in Queensland to be transported during the day without the need for a police escort, it is limited to 4.6m high and 3.5m wide (Table 2.2). It is because of this that the homes are built as "double-sectioned", meaning they are built in two separate halves and joined/married together once in their final location.

Type of Vehicle or Combination carrying indivisible article	**Dimension**	**Limits**
Trucks	Length Height Width	12.5m 4.6m 3.5m
Trucks hauling one trailer (includes a pig trailer, dog trailer or tag trailer)	Length Height Width	19.0m 4.6m 3.5m
Prime mover semitrailer combination (includes drop decks, low loaders, low loader dollies, jinkers, extendable trailers and platform trailers)	Length Height Width	25.0m *5.0m 3.5m *see note
Type of trailers being towed by a prime mover in an unladen configuration.	**Dimension**	**Limits**
Platform, jinker, low loader dolly, low loader, low loader towed in combination with a low loader dolly and a jinker towed in combination with a low loader dolly.	Length Height Width	25.0m 4.3m As per section 5.4.1

Table 2.2 - Oversize Vehicle Limits (Transport and Main Roads, 2010)

2.6 Installation Time

For all parties involved, it is important to minimise the installation time of the home. If you can become aware with common local practices, it can often greatly speed up construction time. Minimising the use of out of the ordinary components such as precast concrete beams, manufactured structural panels and prefabricated steel members can significantly reduce the installation time.

Most relocatable home installations take as little as one day to complete, and up to a week for more complex procedures. Unlike larger construction sites, relocatable home delivery dates are very accurate generally to within a day. To avoid lengthy delays, all site work and foundation construction should be completed prior to the scheduled delivery date. Special cases where typical back-of-the-truck delivery to the final location is difficult (such as at the end of a cul-de-sac) may require a crane to move the home onto the foundation and typically take longer to install.

CHAPTER 3 - Unexplored Research

In reviewing literature on the proposed topic of manufactured homes, certain gaps in current knowledge have become apparent and will be aid the research conducted. The following list contains elements of unexplored research that aim to be filled through the process of this research:

- Australia contains four different classifications of expansive soil, with the higher degrees of reactivity being very unstable and difficult for construction. AS2870 residential slabs and footings code outlines footing requirements for short piers (braced stumps) based on horizontal loading and uplift forces, but nothing for dry stack blocks. Currently, there is no advice available for their design in each soil condition, with manufacturers of the homes simply using their own standard design for all cases. It is the aim of the author to provide recommendation tables that will allow manufacturers to simply look up a table and find the wind region that the house is located in, compare that with the footing height required for the house and find out exactly what tie-down, longitudinal and cross bracing is required based on significant calculations found in the Appendices.
- It is apparent that an analysis of dry stack block footing systems has never been researched previously. Manufacturers simply use what has been working for them for years. Perhaps both types of footings are equally useful for all conditions, but it is hopeful that this is not the case so that this research will be helpful for future manufactured home designs.
- The United States of America have shown significant advances in manufactured home technology with a large population of Americans living in these homes. Their major support system is the use of dry stack blocks with appropriate anchorage and tie-downs for most purposes. In America, they have the added difficulties of frost heave, a large portion of their homes involving multiple stories and basements, and several cities being susceptible to tornadoes and other strong wind cases.
- Transportation of wide structures is common practice in Australia, and so there was nothing of interest to this topic left to research. As such, as standard sized double-wide home will be used for the analyses in this thesis.

CHAPTER 4 - Research Methodology

The research involved with this thesis is based purely around the analysis of a home affected by directional loadings to determine satisfactory support systems for rising heights of dry stack blocks in each of the four wind regions of Australia. Several objectives are to be completed, each of which involving different aspects of calculations and interpretation. The steps involved in successfully completing the research are:

1. Determine design wind speeds for each wind region in Australia using AS1170.2 code.
2. Convert these wind speeds into pressures based on pressure coefficients also found in AS1170.2.
3. Calculate the weight of a typical home using AS1684.2 and convert this weight into a force applied vertically on each footing.
4. Determine friction coefficients that will act to prevent the house from sliding with wind actions.
5. Calculate the net weight of the house in a windstorm to determine if tie-down is required.
6. Using the coefficients of friction found previously, determine whether or not the bearer will slide over the footing during a windstorm. This analysis will be conducted for both longitudinal and cross winds to find out the required bracing in each of these directions.
7. Alter the height of the footing and reiterate the above procedure to create recommendations tables to be used against an existing manufactured home to determine its safety in a major windstorm. In all there are four wind regions (A, B, C and D), and possibly four or more footing heights, meaning a total of at least 16 options to be analysed.

CHAPTER 5 – Elements Involved in Footing Design

When designing relocatable home footing systems, many elements will affect its overall behaviour and therefore must be investigated in order to understand the design. This chapter involves the major considerations to be allowed for in the footing system design. It can be expected that homes will receive effects caused by seismic and snow loads, however for the purpose of this research these factors will be omitted as these situations are so rarely encountered here in Australia.

In all the factors to be considered are:

- Soil conditions
- Horizontal loading (wind actions)
- Vertical loading (weight of home)
- Termite prone regions

5.1 Soil Conditions

Australia has a variety of soil conditions, which have been categorised in the Residential Slabs and Footings (1996) code according to their reactivity (see Table 5.1). Because the soil is what supports the loads on the homes, a great understanding of their properties is required for the final selection and design of the foundation. Some soils have great ability to support the weight without manipulation, while others have very little. Some soils become more supporting while wet while others act the same when dry. Some soils expand while others shrink with the presence of moisture. Some soils are easily compacted while others aren’t. It is important when deciding on a foundation that a soil report is conducted to the relative standards to know exactly what soil classification the home will be built on as this decision can help with its overall lifetime cost and durability.

Class	Foundation
A	Most sand and rock sites with no or little ground movement from moisture changes (sand and rock).
S	Slightly reactive clay sites with only slight ground movement from moisture changes (silt & some clay).
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes.
H	Highly reactive clay sites, which can experience extreme ground movement from moisture changes.
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes.
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; land slip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which can not be classified otherwise.

Table 5.1 – General Definitions of Site Classes (AS2870, 1996)

An important measure of a soil's ability to support weight is its bearing capacity. By definition this is a value representing the load that a square metre of the ground's surface is capable of supporting without risk of yielding or displacement (Civil Engineering Terms, 2011). Problems can also be caused from an excess of organic matter in the soil. This matter should be removed and replaced with properly compacted fill.

Another group termed expansive soils significantly change in volume when they absorb water. The higher the reactivity the more they tend to shift with the rising and falling of the water table. Slab-on-grade construction is usually the preferred method when building on areas with expansive soils.

Provinces distinguish major physiographic changes across the country (Physiographic Regions of Australia, 2011). They are compiled by grouping together regions with similar landform and geological characteristics. There are 23 Provinces outlined for Australia as shown in Figure 5.1.

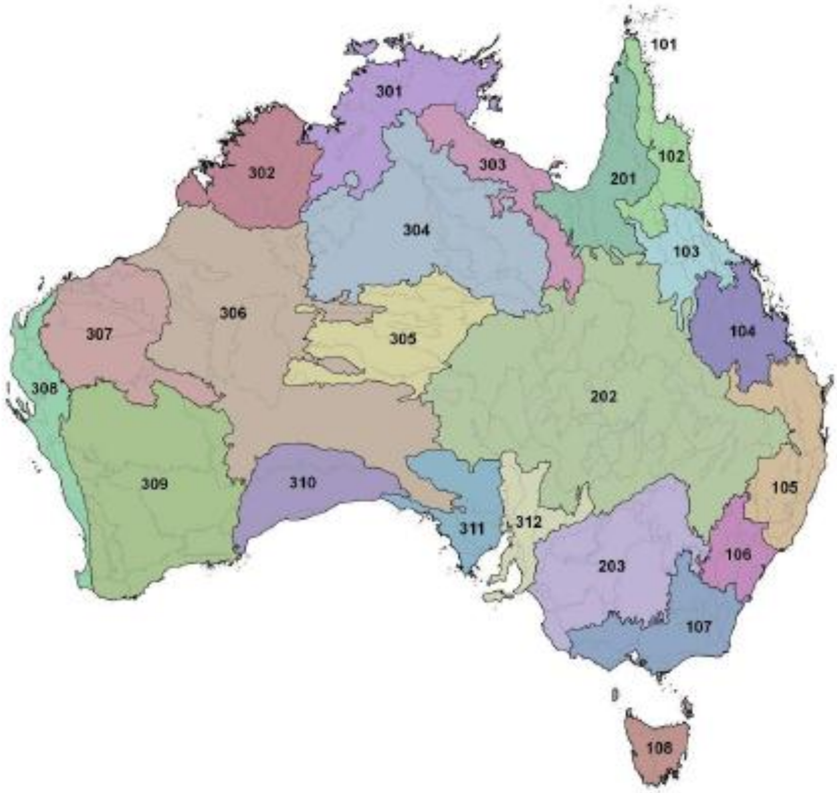

Figure 5.1 – Physiographic Provinces of Australia (Pain et al, 2011)

5.1.1 Foundation Maintenance

AS2870 residential slabs and footings code specifies the required maintenance for each of the classified soil cases. The designs and design methods in the code are based on the performance requirement that significant damage can be avoided provided that foundation site conditions are properly maintained.

Water affects all soils in one way or another. Silts are weakened, sands can settle, however most problems arise with the presence of clay foundations. Their shrinking and swelling with seasonal moisture changes have been classified in the code as per Table 5.1.

5.1.1.1 Class S Sites

Sites classified Class S can be considered non-reactive. They should be protected from extreme wetting through use of site drainage and prompt repair of plumbing leakages.

5.1.1.2 Class M, H and E Sites

Class M, H and E sites should be maintained at stable moisture conditions with extreme wetting or drying prevented. In order to achieve this, the following terms should be followed:

- No ponding should occur near or against the house. This can be achieved through proper site draining and ensuring 50mm minimum uniform fall over the first metre away from the house.
- Development of gardens should not interfere with the drainage or subfloor ventilation systems. If possible, avoid garden beds adjacent to the house and overwatering gardens close to footings.
- Trees cause damage to reactive sites as they intake moisture and hence dry out the clay at substantial distances. Planting of trees near house foundations should be avoided or at least restricted to a distance from the house of 1.5 x mature height (mh) for Class E sites, 1mh for Class H sites and 0.75mh for Class M sites. Note that removal of trees can also cause similar problems.
- Any leakages found in sewer or stormwater plumbing should be promptly repaired.

5.2 Horizontal Loading (Wind)

Often the major loading on any structure is that caused by wind. Depending on the structure, both external and internal pressures need to be calculated when designing all structural elements such. When designing structural members, worst-case scenarios should always be considered. For the case of a relocatable home footing system, downward loading caused by self-weight of the home will be opposed by negative external roof pressures created by a cross wind. Only the footing design is of interest here, therefore internal pressures can be neglected because any additional downward load on the roof created by a negative internal pressure will be cancelled out by an equal upward load on the flooring.

As shown in Figure 5.2, the wind code (AS1170.2:2002) separates Australia into four regions based on expected wind speeds. Regions A and B cover most of the map and are considered non-cyclonic meaning they encounter relatively low wind speeds. Regions C and D are coastal regions which are termed cyclonic and therefore receive much faster winds. It is noted that Region D occurs only on the coastline of Western Australia and so any research findings for this region will only be relevant for relocatable homes in that small area.

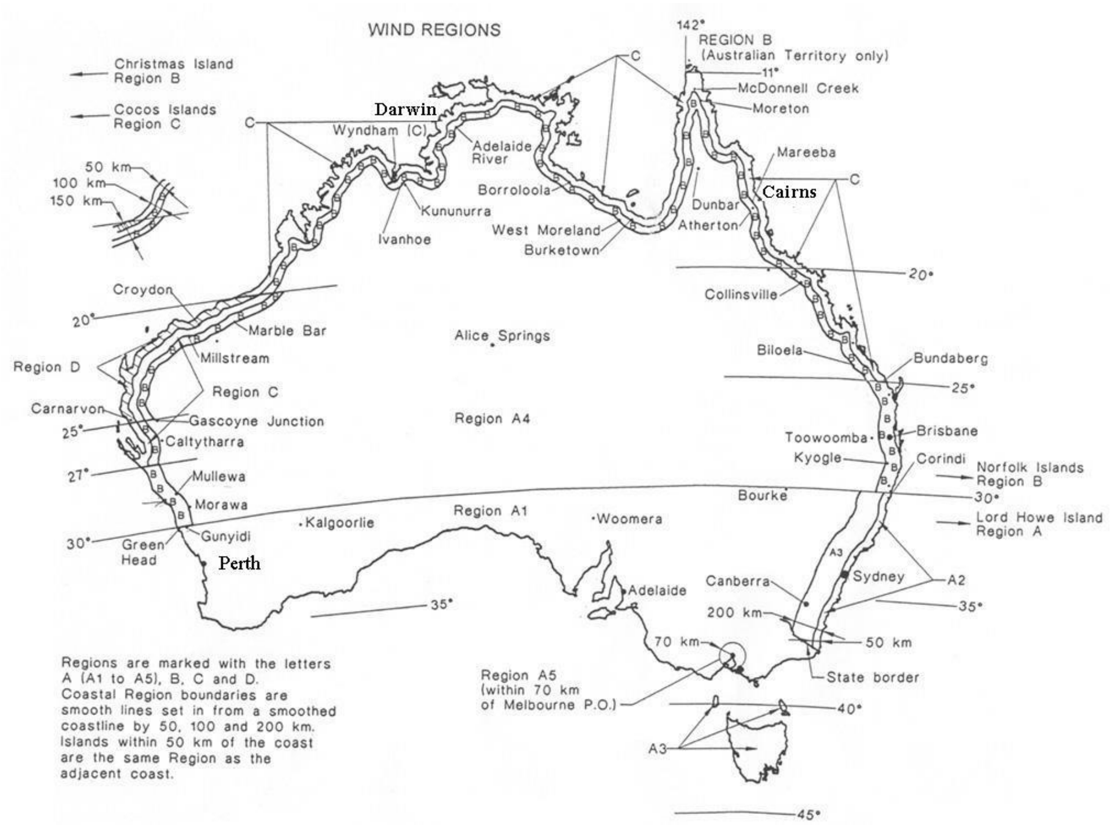

Figure 5.2 - Wind Regions of Australia (AS1170.2, 2002)

5.3 Vertical Loading (Weight of Home)

Using manufacturer's specifications, the dead load created by an average sized relocatable home that will be transmitted through to the footings was calculated to be 112.1 kN (see Appendix B). This value along with the above regional wind pressures will be used in a similar fashion to Figure 5.3 for the analysis of the two systems.

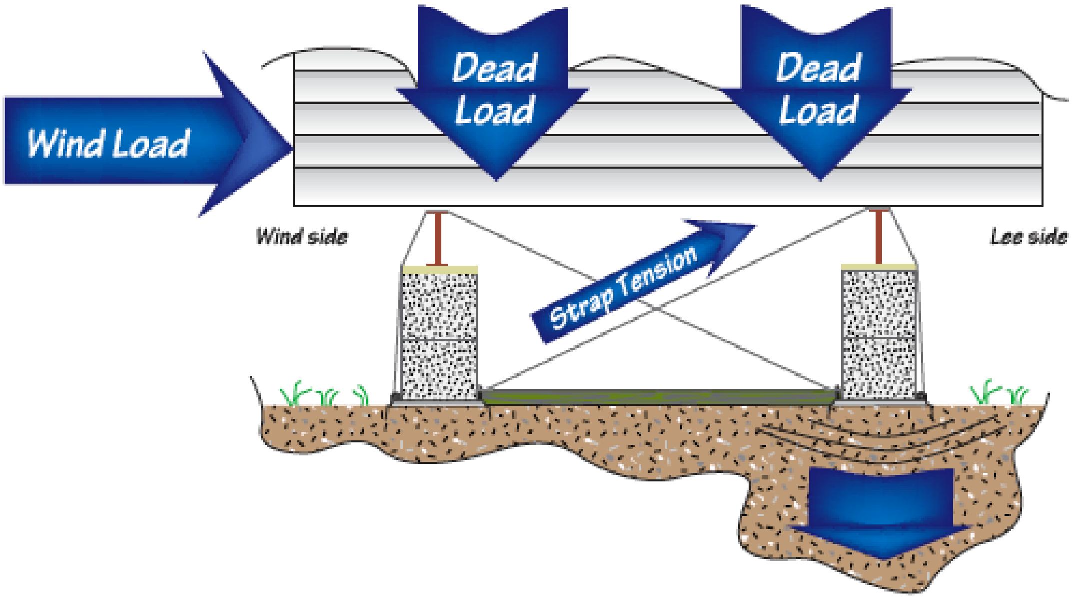

Figure 5.3 – Applied Loads (MHRA, 2002)

5.4 Termite Prone Regions

Termites cause major problems to wooden structures all over the world. In Australia, the regions more susceptible to termite damage are shown in Figure 5.4. The use of steel, concrete and pressure-treated timber should be considered in the design of footings in these regions. In almost all designs across Australia, pieces of galvanised sheet metal known as ant caps are used as a form of barrier between the flooring and support systems. They are used to force termites into the open so they are easily detected during physical inspections.

In termite prone regions, the design and construction of the support system should ensure that there is no contact between untreated wood members of the foundation and the ground.

For the purpose of this research it will be assumed that ant caps are used throughout the nation for the sake of future protection in case of a spread of termite infested areas.

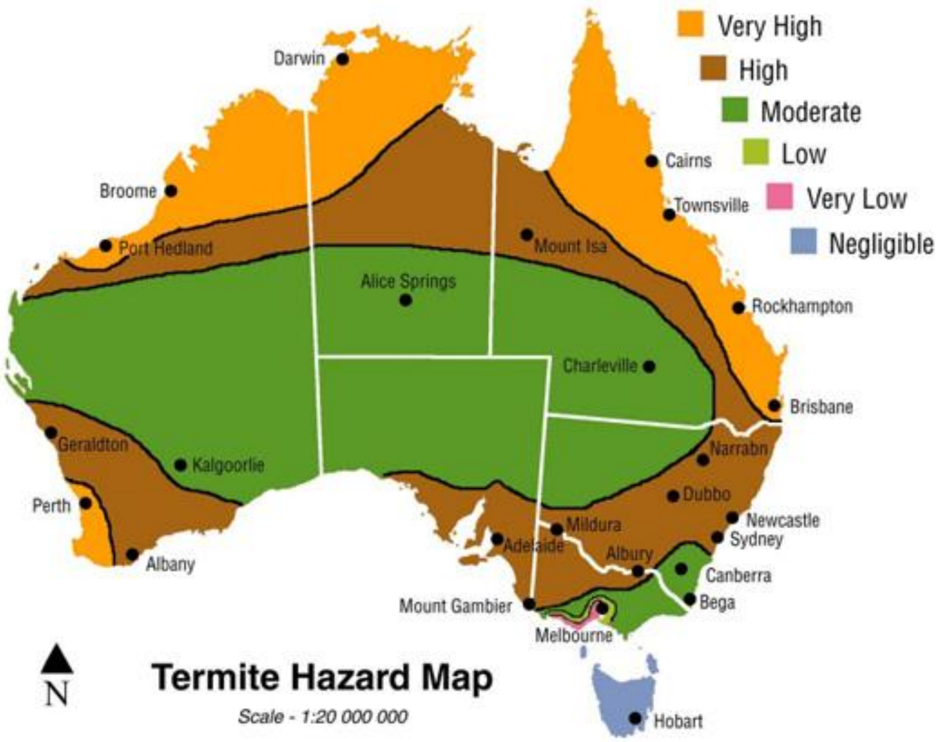

Figure 5.4 – Termite Hazard Regions (CSIRO Student Research, 2002)

CHAPTER 6 – American Case Study

The Manufactured Housing Research Alliance (2002) have covered vast areas of selection, design as well as installation procedures for a wide variety of manufactured homes in the United States of America. This thesis is written with the intent that those in the industry will be able to recognise the available footing system options for relocatable homes here in Australia, whether the reader is a developer looking for a more efficient solution, a contractor faced with unusual conditions, or a student wanting a better understanding of available solutions. It will present a compilation of foundation ideas from years of research from industry experts and offer practical and cost-effective solutions.

A common phrase in the engineering society is that there is no single best solution for anything. Several hypotheses are generally plausible and the final solution will often be a combination of the consideration of available funds, aesthetics, available installation techniques, size or locality of the home among several other factors. Hopefully however, the use of this research will allow for a narrowing of options, making final selection easier for the developer.

In the U.S. manufactured homes have been taken to a new level with non-proprietary systems built of readily available materials now covering four foundation classifications:

- Pier and ground anchor support systems (most popular method of securing manufactured homes to the ground
- Crawl-space systems
- Slab-on-grade foundation systems
- Basements

For relevance to this study, a closer look at the pier and ground anchor support system will follow. It should be noted that here in Australia, rod bracing is the preferred method.

6.1 Pier and Anchor Foundation System

This has long been the commonly accepted form of support system for manufactured homes in the United States. It is easily adapted to site conditions, does not require great precision and is quickly constructed.

Most commonly piers are installed under the main bearers and along the mating line as shown in Figure 6.1. Perimeter piers and other manufacturer-specified piers could also be found depending on individual circumstances (see Figure 6.2).

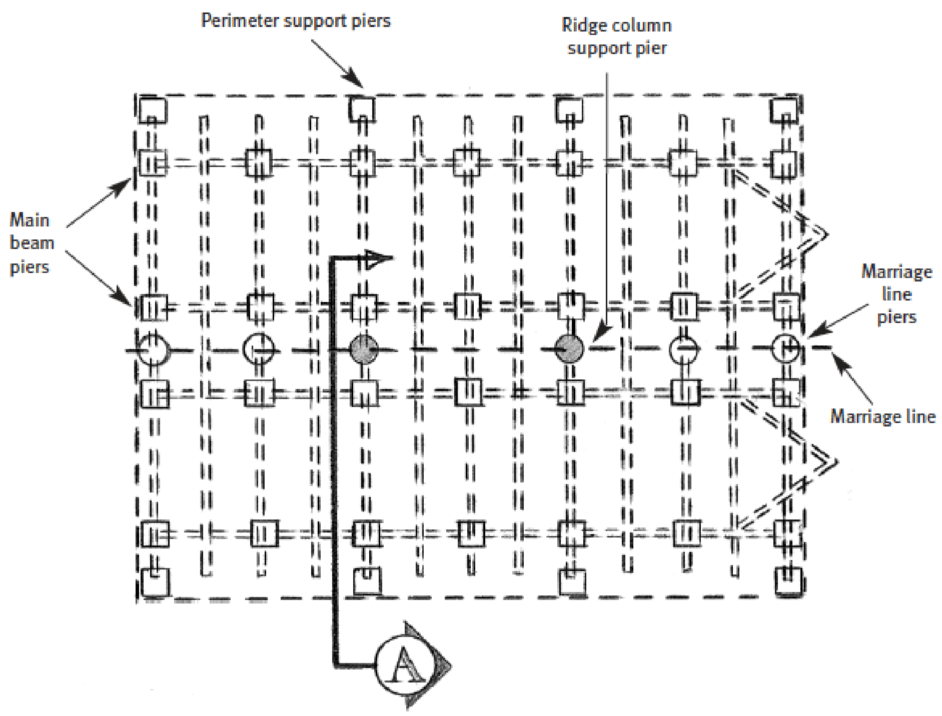

Figure 6.1 – Pier and Anchor Foundation Components (MHRA, 2002)

The pier height and building weight generally dictate the allowable pier spacing. The most common piers are steel jack stands or hollow core concrete masonry blocks with cores positioned vertically and placed one on top of each other to the required height (Figure 6.3).

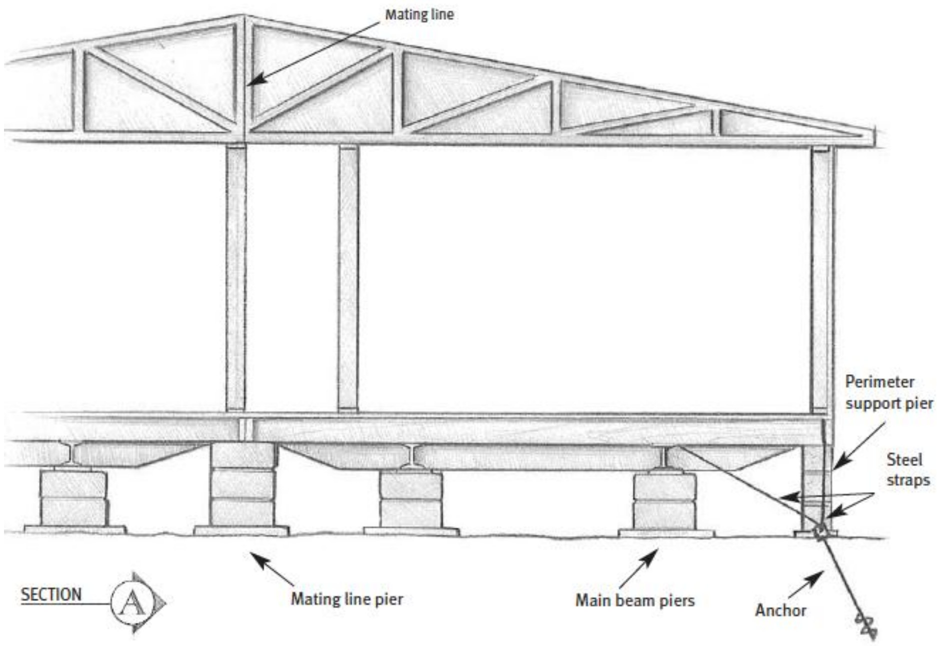

Figure 6.2 – Section A (MHRA, 2002)

Depending on the height, the blocks can be singly or doubly stacked laid in interlocking fashion as shown. It is recommended that piers higher than 36 in. (91.4cm) should be configured as doubly stacked and that piers higher than 80 in. (203.2cm) require an engineer's design. Results from the analysis in Chapter 8 should determine safe-working heights of dry stack blocks in Australia.

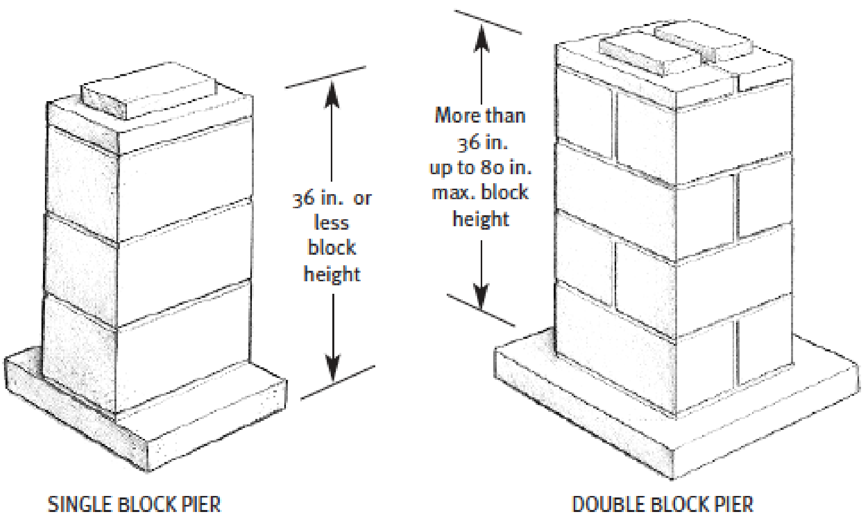

Figure 6.3 – Concrete Block Pier Configuration (MHRA, 2002)

Piers are set on square concrete pads, which spread the pier loads over a larger area, making the base more stable. The pier spacing and soil bearing capacity will determine the size of the pads. Pads should be set on compacted or undisturbed soil, with loose or organic matter cleared until solid soil is exposed. Without this clearing, uneven settlement can occur creating damaging stresses throughout the home.

Screw-in ground anchors are the most common devices for resisting wind uplift forces on manufactured homes in the United States. They are attached to the UB bearer by steel straps (see Figure 6.4), which require periodic checks to ensure they remain in tension.

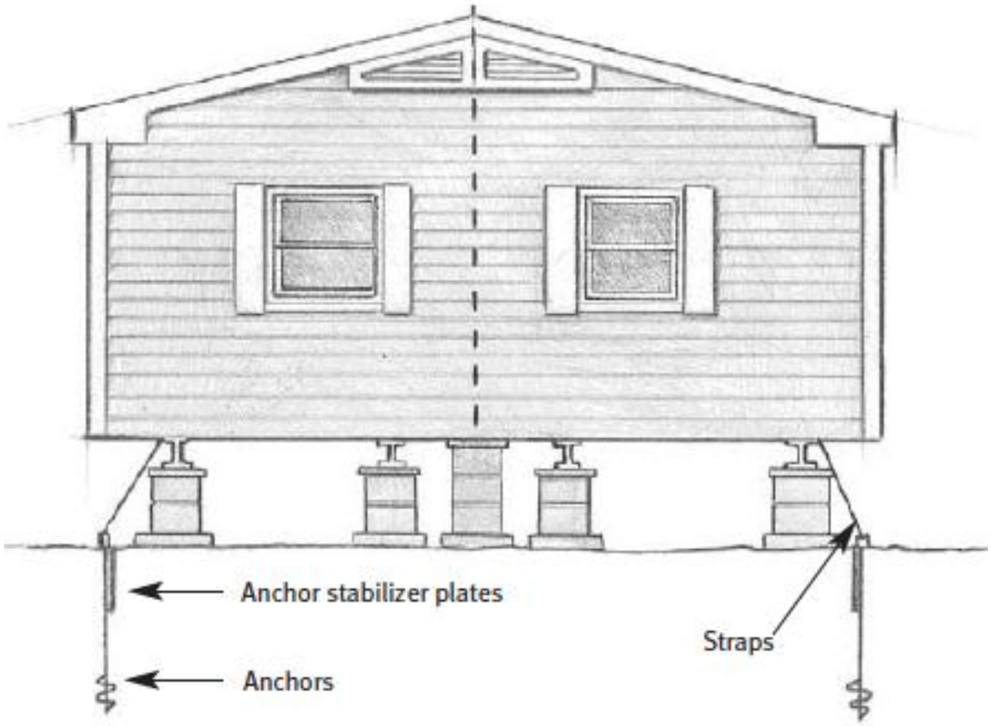

Figure 6.4 – Bearer to Anchor Connection (MHRA, 2002)

6.1.1 Cost of Construction

With other support systems involving slabs, crawlspaces or basements, the pier and anchor support system has the lowest initial cost. However, from the savings of labour and material costs, you can lose out with overall useable space. Depending on the wind regions, you may require a significant amount of anchors and straps, including on sidewalls, which may become expensive. Installation of the system is usually completed in a single working day.

6.1.2 Wind Load Resistance

Of all the support systems, the pier and anchor system is often specified as an effective way to resist wind forces. Proper tie-down and bracing of the superstructure to the bearers means the only section of the load path yet to be completed is from the bearers to the ground. This is accomplished with the use of the anchors, set in each direction so that wind in any direction will be fully resisted and transferred through to the foundation. Often cross bracing is not required if the piers themselves can carry the required bending moments.

6.1.3 Gravity Load Resistance

In order for the pier and anchor support system to adequately support gravity loads, it must be properly designed to take into account the soil bearing capacity and ensure correct pier spacing is used. Concrete blocks are very strong in compression but should still be checked against the weight of the house. Buckling failure is very uncommon, but must be checked especially when using steel piers.

6.1.4 Seismic Load Resistance

There are no provisions in the American Standards for the design of seismic resistance, meaning manufactured homes are not specifically designed to withstand seismic loads, however calculations have shown that a home capable of resisting wind forces will exceed the requirements for the highest seismic forces in the codes. Seismic occurrences are so rare in Australia that they will not be considered in this study.

6.2 Conclusions Drawn From Case Study

Based on the diagrams and information given by the MHRA, a number of conclusions can be drawn that are of relevance to footing system designs here in Australia:

- The overall pier layout is similar to what is found here, however the use of perimeter and marriage line piers are not common practice and are considered to be over-engineered perhaps for an additional factor of safety against wind loads, or perhaps frost heave, snow or seismic loads is more likely behind the reason for their presence.
- Dry stack masonry blocks appear to be used for almost all designs in the United States, except when basements or very high footings are required. Here this form of support system is also very common, however there are no standards relevant to use of dry stack blocks as house supports.
- From discussions with staff at Norfolk Relocatable Homes Pty Ltd, it is apparent that the standard block footing layout is different to that used overseas. Here they use standard 190x190x90 high blocks for the top 2 courses of the footing. For anything higher 390x190x90 high blocks are used doubly wide and alternated each course up to the required height (see Figure 7.3). For this reason, the chosen heights for the analyses in Chapter 8 were 270mm (2 courses of 190x190x90 and 1 course of 390x190x90), 540mm, 810mm and 1080mm. The bottom course is grouted to a square base footing, usually designed by geotechnical engineer.
- Screw-in ground anchors are used to resist uplift forces on the homes. In Australia, common practice is to provide subfloor bracing in the form of galvanised rods (M12 to M16 typical) to tie-down the superstructure elements to the foundation.

CHAPTER 7 – Analysis Parameters

The analysis of using dry stack blocks as a suitable footing system involves an advanced understanding of engineering statics, namely the basic equilibrium theories.

Wind in the horizontal direction is resisted by friction interfaces within the support system. If the frictional resistance is insufficient to resist the wind forces, then bracing is required. Similarly, uplift pressures created on the roof are resisted by the self-weight of the home. If the uplift forces are greater than the resisting self-weight, tie-down rods are required. The number of rods required will then be calculated and shown in the final recommendation tables.

7.1 Horizontal Wind Actions

Detailed calculations were required to find wind pressures acting on the homes in each region (see Appendix A). The wind speeds calculated in each region were as follows:

Region A – 39.15 m/s
Region B – 49.59 m/s
Region C – 58.65 m/s
Region D – 74.80 m/s

These speeds were then converted into pressures using the appropriate formula in AS1170.2. The results were as follows:

Region A – 0.515 kPa
Region B – 0.826 kPa
Region C – 1.156 kPa
Region D – 1.880 kPa

These pressures shown as the symbol p in Figure 7.1 are acting on the walls of the house and are transferred through to the support system, which must be able to resist the shear forces created through either frictional resistance or bracing rods.

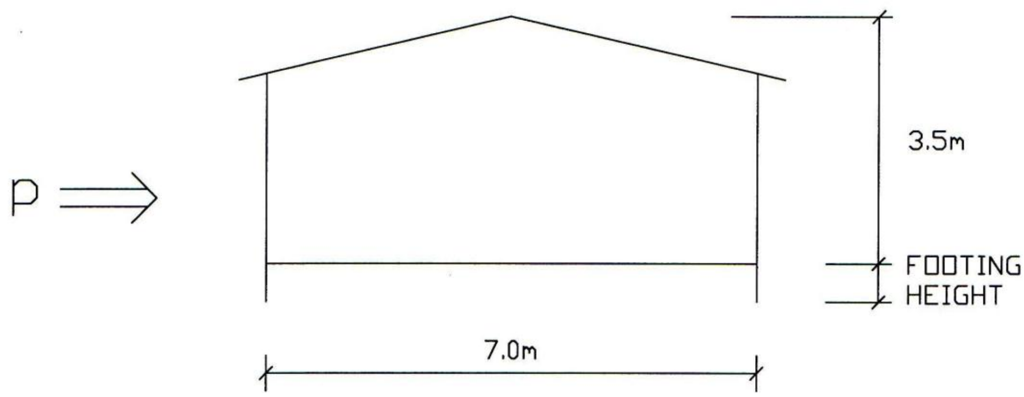

Figure 7.1 – Horizontal Wind Pressures Acting on Home

From assumptions made, the calculations showed that the wind pressures could be considered equal for both longitudinal and cross winds because the calculations were made based on worst-case scenario, and no site information was given. This however doesn't mean that the same bracing will be required in each direction. There is a larger surface area of wall hit by cross winds and so a larger force will be created in this direction. It is therefore expected to find that significantly more bracing will be required in the direction of the cross winds in comparison to that of the longitudinal winds.

AS1684.2 states that the total racking force applied to a home is equal to the elevation area multiplied by the wind pressure. This creates a force in kilonewtons and must be calculated for both longitudinal and cross wind cases, as there will be a differing elevation area.

From the reiterated calculations, the following tables show the cross and longitudinal racking forces determined for each wind region for four sets of heights.

	270mm	**540mm**	**810mm**	**1080mm**
A	27.38	29.34	31.30	33.26
B	43.92	47.07	50.21	53.36
C	62.06	66.51	70.95	75.40
D	100.05	107.21	114.38	121.54

Table 7.1 – Cross Racking Forces (kN)

	270mm	540mm	810mm	1080mm
A	14.32	15.41	16.51	17.61
B	22.96	24.73	26.49	28.25
C	32.45	34.94	37.43	39.92
D	52.31	56.32	60.34	64.35

Table 7.2 – Longitudinal Racking Forces (kN)

7.2 Vertical Wind Actions

Based on the theories of aerodynamics, wind passing over an object such as a roof creates a suction or uplift on the structure. The AS1170.2 wind code specifies that for a 13^{o} roof pitch (standard pitch used in Norfolk Homes designs), a varying pressure acts over the roof surface with a separate external pressure coefficient for the upwind and downwind slopes of the roof. For the standard home used for this research, external pressure coefficients of -0.78 and -0.5 were found for the upwind and downwind slopes respectively (Figure 7.2).

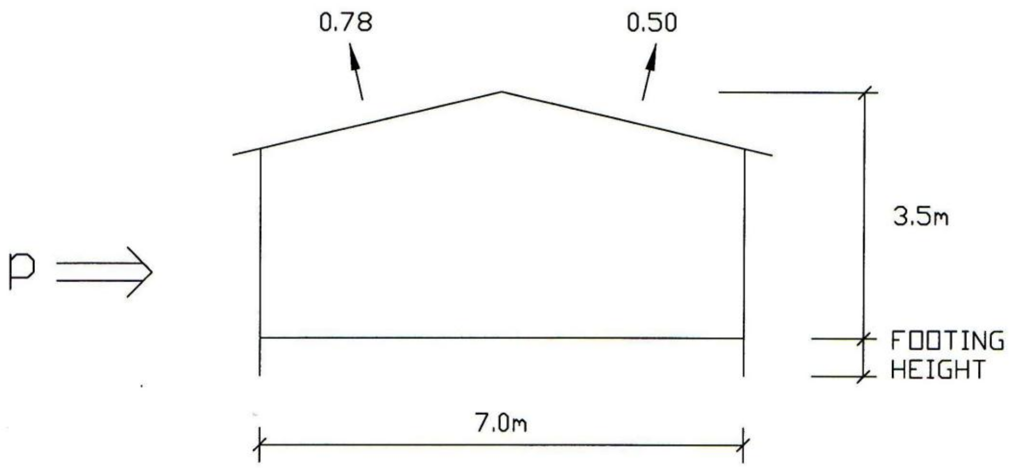

Figure 7.2 – Vertical Wind Pressures Acting on Home

During windstorms, these pressures act against the self-weight of the home to try and lift it off its stumps, consequently making it easier to move around laterally. From Appendix A it was found that the uplift forces created in each of the four wind regions were as follows:

Region A – 29.52 kN

Region B – 47.36 kN

Region C – 66.93 kN

Region D – 107.88 kN

Now that the uplift force has been calculated, the net uplift force can be determined simply by applying a 112.1kN downward force created by the self-weight of the home. The calculated net uplift in each wind region were as follows:

Region A – 112.1 - 29.52 = 82.58 kN

Region B – 112.1 - 47.36 = 64.74 kN

Region C – 112.1 - 66.93 = 45.17 kN

Region D – 112.1 - 107.88 = 4.22 kN

It is of interest that during a 1:500yr windstorm in Region D, the force created by the wind passing over the roof and causing uplift is almost enough to negate the downward force created by the self-weight of the home. The weight of the home is effectively reduced from 11210kg to just 422kg.

A typical stump layout is similar to that of Figure 6.1. For the standard home used in this research four rows of six stumps are used at even spacing and so it was assumed that the net vertical loading would be divided evenly between the 24 stumps. Therefore, the net vertical force applied to each stump during a 1:500yr windstorm was calculated to be a downward force of:

Region A – 82.58/24 = 3.44 kN/stump
Region B – 64.74/24 = 2.70 kN/stump
Region C – 45.17/24 = 1.88 kN/stump
Region D – 4.22/24 = 0.18 kN/stump

From the results above it was found that an inverse relationship existed between the wind pressure and net uplift. A smaller wind pressure will in turn provide a greater downward vertical force on each stump. Provided that the dry stack blocks have a larger bearing capacity than the force applied, the downward vertical force will in turn help resist the home from sliding off its stumps. It should therefore be found in the analysis that lesser tie-downs are required in the lower wind categories.

7.3 Horizontal Wind Resistance

Wind resistance in the horizontal plane exists in the form of friction interfaces within the support system. The interfaces are related to coefficients of friction determined by experimental data, whereby two surfaces are placed together and the force required to slide one surface over the other is measured and a coefficient of friction is calculated. This is best explained with the following equation:

$$P = \mu N$$

Eqn 7.1

where P = frictional force exerted by one surface on the other (kN)
μ = coefficient of friction
N = normal force exerted by one surface on the other (kN)

The interfaces are shown below in Figure 7.3.

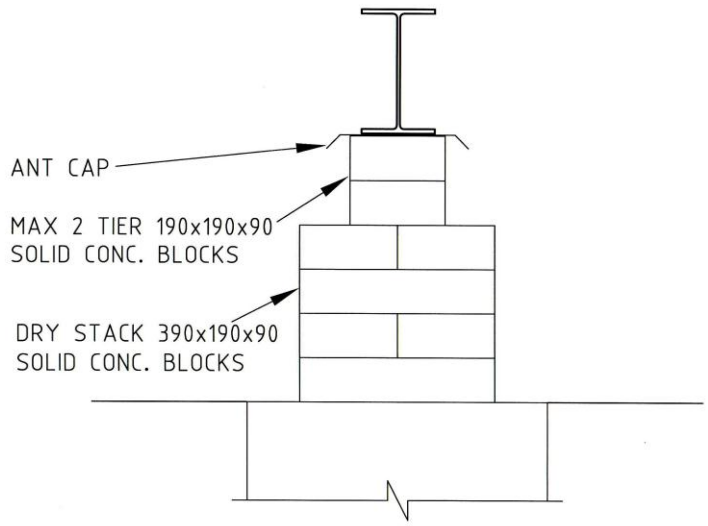

Figure 7.3 – Frictional Interfaces

Many interfaces have been tested and the coefficients of friction relating to this research have been provided as follows:

Interface	Coefficient of friction (μ)
Bearer > Antcap	0.3
Antcap > Dry stack block	0.45
Dry stack block > Dry stack block	0.7
Dry stack block > Footing	N/A (cast in concrete)

Table 7.3 – Frictional Coefficients (Coefficient of Friction, 2011)

The conclusions made from the above values are as follows:

- The lower the coefficient of friction, the easier it is for the two surfaces to slide over each other. As shown above, it is going to require more than twice the force to slide bricks over bricks than it is to slide the bearer over the antcap.
- If a horizontal force is applied to the home (i.e. from wind), the first place that the home will begin to slide is at the connection of bearer and antcap because this has the lowest coefficient of friction.
- For the analysis of tie-downs and bracing, only the bearer to antcap interface will be required. The sliding failure will occur at this location, therefore other interfaces can be ignored.

Results from Section 7.1.2 show the downward force applied on each stump during a 1:500yr windstorm. These values form the normal force, N in equation 7.1, that will be applied on each support. Table 7.3 then provided me the coefficient of friction between the bearer and antcap, the point at which sliding failure will occur on the home. The required total frictional force to cause this sliding failure can now be determined from formula 7.1. The results are as follows:

Region A: P = 0.3 * 3.44 kN/stump * 24 stumps = 24.77 kN
Region B: P = 0.3 * 2.70 kN/stump * 24 stumps = 19.42 kN
Region C: P = 0.3 * 1.88 kN/stump * 24 stumps = 13.55 kN
Region D: P = 0.3 * 0.18 kN/stump * 24 stumps = 1.26 kN

The above values signify the forces required to slide a home (without any tie-down or bracing support) in any direction between the bearer and antcap, in the event of a 1:500yr windstorm. Amazingly, in Region D only 1.26 kN of force, which is equivalent to roughly 126 kg is required to slide the home around on its supports. It is obvious that substantial tie-downs and bracing will be required.

Tables 7.1 and 7.2 show the racking (sliding) forces exerted on a home in each wind region, from cross and longitudinal winds respectively. One discovery made from comparing the racking forces in Tables 7.1 and 7.2 with the above calculated forces required to slide the home, is that relocatable homes in Region A will slide in the direction of cross winds, but not in the direction of longitudinal winds. In the case of longitudinal winds, the racking force does not exceed the required sliding force, and therefore no longitudinal bracing is required.

Homes in Regions B, C and D were all subject to forces capable of causing sliding failure in both cross and longitudinal directions, and therefore bracing will be required in both directions.

7.4 Dry Stack Self-Weight

In order to correctly analyse the dry stack blocks, in addition to the weight of the house acting vertically downwards on the footing, the self-weight of the dry stack blocks must also be taken into account. This weight will be very minor for short stacks, however can become quite heavy as they stack up.

The weights of the dry stack blocks are simply the volume multiplied by the density of concrete (24 kN/m3). Self-weights to be used in the analysis in Chapter 8 are listed below.

2 courses 190x190x90 blocks = 0.19*0.19*0.18*24 = 0.156 kN
1 course 390x390x90 blocks = 0.39*0.39*0.09*24 = 0.329 kN

7.5 Tie-down Tensile Strength

If tie-down rods are required, we need to know exactly how much lateral movement is required to attain the ultimate tensile strength of the rod.

A standard M12 galvanised threaded tie-down rod has 27kN tensile strength (ASI, 2009). This strength is achieved when the rod is yielded. Therefore we can determine the stress required to cause this yielding. From this stress we can find the strain required, and consequently the lateral movement required. This is outlined in the proceeding calculations:

σ = P/A = 27000 N / 110 mm^2 = 245 MPa
ε = σ/E = 245 MPa / 200 GPa = 0.001227
The rod elongation is therefore:

270mm stack: 0.001227 * 270 mm = 0.3314 mm
540mm stack: 0.001227 * 540 mm = 0.6627 mm
810mm stack: 0.001227 * 810 mm = 0.9941 mm
1080mm stack: 0.001227 8 1080 mm = 1.3255 mm

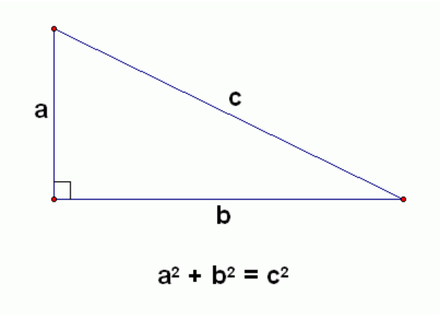

Figure 7.4 – Pythagorean Theorem (TutorVista.com, 2010)

Based on the Pythagorean theorem illustrated above, and solved to find a, the lateral movements required to cause the bar to yield are therefore:

270mm stack: 13.382 mm

540mm stack: 26.763 mm

810mm stack: 40.145 mm

1080mm stack: 53.526 mm

These values show that little movement is required for the rod to yield, so it will be assumed that the full 27kN strength can be attained by the tie-downs.

7.6 Cross Bracing Tensile Strength

With regards to cross wind, a cross bracing bar laid horizontally will achieve the maximum 27kN tensile strength. If the bar is laid vertically it can be assumed to provide 0kN tensile strength towards resisting the cross wind forces. For varying heights of the dry stack blocks, this will mean that cross bracing rods will form various angles to the horizontal plane, and will therefore be capable of providing only a percentage of the full tensile strength.

The stumps shown in Figure C.2 are located at 3.25m centres. Varying the height of the stumps, the angles formed with the horizontal plane and therefore corresponding maximum tensile strengths are:

270mm stack: $\tan^{-1}$ (0.27m / 3.25m) = 4.75^o

$27 \cos 4.75^o$ = 26.907 kN

540mm stack: $\tan^{-1}$ (0.54m / 3.25m) = 9.43^o

$27 \cos 9.43^o$ = 26.635 kN

810mm stack: $\tan^{-1}$ (0.81m / 3.25m) = 13.99^o

$27 \cos 13.99^o$ = 26.199 kN

1080mm stack: $\tan^{-1}$ (1.08m / 3.25m) = 18.38^o

$27 \cos 18.38^o$ = 25.622 kN

7.7 Longitudinal Bracing Tensile Strength

Standard practise in Australia is to run the longitudinal bracing rod at 45^o from the footing to the bearer as shown in Figure C.3. Using similar calculations to the cross bracing tensile strength is Section 7.6, this achieves a maximum tensile strength of 19.092kN.

A typical layout taken from calculations in Appendix H is shown below. As can be seen, six stumps have tie-downs, two bays of cross bracing are used and four corners have longitudinal rods.

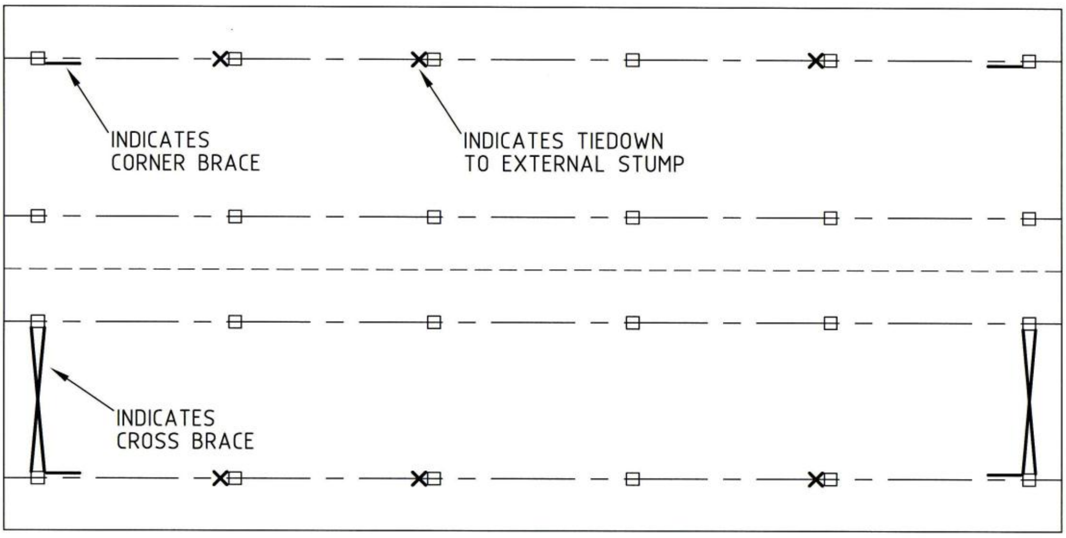

Figure 7.5 – Wind Region B, 540mm Stack Layout

Chapter 8 will use the parameters determined here in Chapter 7 to analyse the dry stack block footing option for a typical home in each of the four wind regions, and for four sets of heights. Recommendation tables will show the results of the 16 analyses and will provide information regarding required tie-down, longitudinal and cross bracing for each circumstance.

CHAPTER 8 – Dry Stack Block Analysis

Appendices C through R provide detailed calculations required to create the following recommendation tables. 3-D representations of the tables are also provided in Figures 8.1-4. For information regarding a specific design please refer to the corresponding Appendix.

Footing Height	Tie-downs	Cross Bracing	Longitudinal Bracing
270mm	-	2 bays M12 with t'buckle	-
540mm	-	2 bays M12 with t'buckle	-
810mm	-	2 bays M12 with t'buckle	-
1080mm	8 stumps N12 galv rod	2 bays M12 with t'buckle	-

Table 8.1 – Wind Region A Recommendations

Footing Height	Tie-downs	Cross Bracing	Longitudinal Bracing
270mm	4 stumps N12 galv rod	2 bays M12 with t'buckle	4 corners M12@45^o
540mm	6 stumps N12 galv rod	2 bays M12 with t'buckle	4 corners M12@45^o
810mm	8 stumps N12 galv rod	2 bays M12 with t'buckle	4 corners M12@45^o
1080mm	12 stumps N12 galv rod	3 bays M12 with t'buckle	4 corners M12@45^o

Table 8.2 – Wind Region B Recommendations

Footing Height	Tie-downs	Cross Bracing	Longitudinal Bracing
270mm	6 stumps N12 galv rod	3 bays M12 with t'buckle	4 corners M12@45^o
540mm	8 stumps N12 galv rod	3 bays M12 with t'buckle	4 corners M12@45^o
810mm	12 stumps N12 galv rod	3 bays M12 with t'buckle	4 corners M12@45^o
1080mm	16 stumps N12 galv rod	3 bays M12 with t'buckle	4 corners M12@45^o

Table 8.3 – Wind Region C Recommendations

Footing Height	Tie-downs	Cross Bracing	Longitudinal Bracing
270mm	8 stumps N12 galv rod	4 bays M12 with t'buckle	4 corners M12@45^o
540mm	12 stumps N12 galv rod	4 bays M12 with t'buckle	4 corners M12@45^o
810mm	18 stumps N12 galv rod	5 bays M12 with t'buckle	4 corners M12@45^o
1080mm	24 stumps N12 galv rod	5 bays M12 with t'buckle	4 corners M12@45^o

Table 8.4 – Wind Region D Recommendations

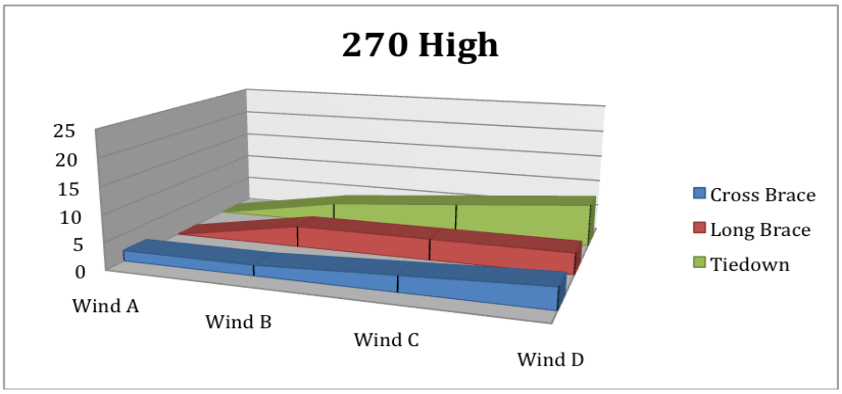

Figure 8.1 – 270mm Stack 3.D Graph

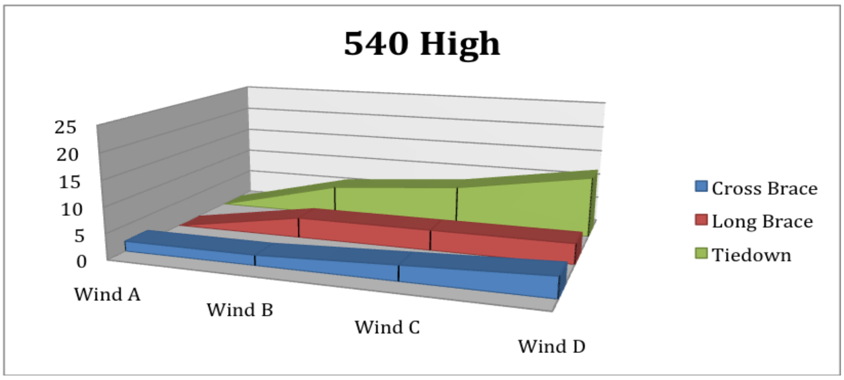

Figure 8.2 – 540mm Stack 3.D Graph

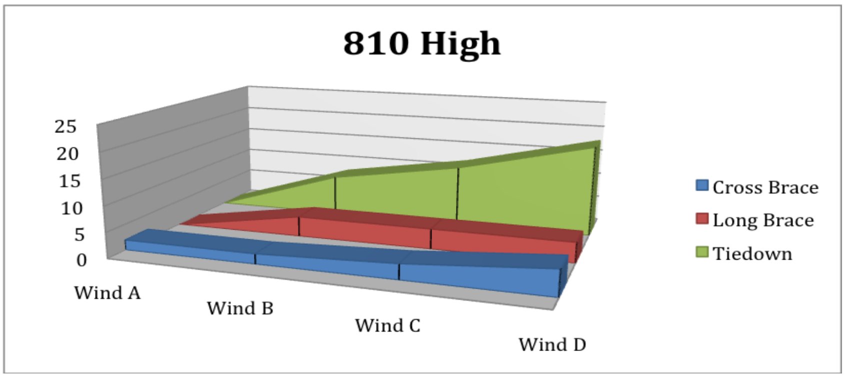

Figure 8.3 – 810mm Stack 3.D Graph

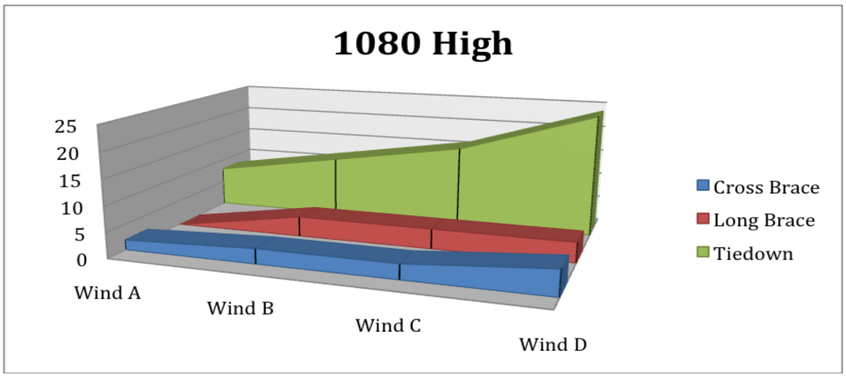

Figure 8.4 – 1080mm Stack 3.D Graph

8.1 Analysis Discussion

The results found and the recommendations made are all based on a standard sized home, worst case wind directional multiplier, worst case shielding – the list goes on. The tables provide only a guide to a manufacturer as to what they can expect to require if relocating a home in these conditions.

There are a number of ways that the requirements can be more exact to a specific situation. They include:

- If the location of the home is known, the directional multiplier can be correctly used, instead of taking the worst case scenario of $M_d = 1.0$.
- If houses or other imposing structures are known to exist in the surrounding areas, the shielding multiplier can be correctly analysed.
- If the house is in a built up environment, the terrain/height multiplier, $M_{z,cat}$ can be reduced if the site can be classed as category 3 rather than the 2.5 used for this analysis.
- Manufacturers often have a scale large enough to weigh their homes in the factory, which takes the guesswork out of the dead load calculations.
- It is possible to reiterate the calculations for intermediate footing heights should this be required.
- For the analysis it was assumed that the sidewalls of the homes are taken through to the ground similar to what is shown in Figure 7.1. This assumption was made because it is standard practise for most relocatable home manufacturers. For higher dry stack footings under stronger wind pressures, it became apparent that the house becomes very light in the strong windstorms because of the uplift pressure created on the roof. Due to the fact that the sidewalls were considered as taken through to the ground, this both increased the elevation area for the wind and meant that downward suction under the house could exist. Use of this downward suction could have helped resist sliding motions on the stumps and therefore it is suggested for higher stumps and stronger wind forces to stop the sidewalls at the location of the timber floor frame. The analysis would have to be reiterated to determine the bracing requirements for this circumstance.

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APPENDIX A – Wind Calculations

The Building Code of Australia (2009) assigns four importance levels for the building of structures. These levels coincide with the consequences of the risk to human life through building failure. The levels are described in Table A.1.

Importance Level Examples		BCA 2009		
BUILDING DESCRIPTION	BCA CLASS	FAILURE CONSEQUENCES		IMPORTANCE LEVEL
		HUMAN HAZARD	PUBLIC IMPACT	
Isolated farm building	10a	Low	Low	1
Residential shed or garage	10a	Mod	Low	2
Small school shade structure	9b	Mod	Mod	2
Produce sales building	6	Mod	Mod	2
Shearing shed	8	Sub	Mod	2
Large commercial storage warehouse	7	Mod	Sub	3
Large (250+) school assembly shelter	9b	Sub	Sub	3
Shed housing hospital emergency generator	10a	Sub	Ext	4
Emergency vehicle shed	10a	Sub	Ext	4

Importance Level 1 is only applicable to isolated farm sheds

Table A.1 – Building Code Importance Levels (BCA, 2009)

From this code, a relocatable home clearly lies in importance level 2. From this, the structure is to be designed to withstand a 1:500 year windstorm event as shown in Table A.2.

IMPORTANCE LEVEL	ANNUAL PROBABILITY OF EXCEEDANCE FOR WIND SPEED	
	NON-CYCLONIC	CYCLONIC
1	1:100	1:200
2	1:500	1:500
3	1:1000	1:1000
4	1:2000	1:2000

Table A.2 – Annual Exceedance Probability for Wind Speed (BCA, 2009)

The Australian Standard AS/NZS 1170:2 Structural design actions, Part 2: Wind actions (2002) is required to calculate wind pressures acting on structures. The code was used to determine pressures acting on a relocatable home situated in each of the four wind regions shown in Figure 5.3.

First the site wind speed is derived from the following equation given in Section 2.2 of the code:

$$V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t)$$

Eqn A.1

where $V_{sit,\beta}$ = site wind speed defined for the 8 cardinal directions (β) at the reference height (z) above ground

V_R = regional 3s gust wind speed, in metres per second, for annual probability of Exceedance of 1/R

M_d = wind directional multipliers for the 8 cardinal directions (β)

$M_{z,cat}$ = terrain/height multiplier

M_s = shielding multiplier

M_t = topographic multiplier

Regional Wind Speeds (V_R)

REGIONAL WIND SPEED m/s				
Speed Probability	Region A	Region B	Region C	Region D
V_{100}	41	48	59	73
V_{200}	43	52	64	79
V_{500}	45	57	69	88
V_{1000}	46	60	74	94
V_{2000}	48	63	77	99

Source: AS/NZS 1170.2 Table 3.1 and Clause 3.4.

Table A.3 – Regional Wind Speeds (AS1170.2, 2002)

Based on Table A.3, regional wind speeds for non-cyclonic regions A and B were taken as 45m/s and 57m/s respectively, while 69m/s and 88m/s were taken for cyclonic regions C and D.

Wind Direction Multiplier (M_d)

As the orientation of the building is unknown, Section 2.2 states that Md is to be taken as 1.0 for all directions.

Terrain/Height Multiplier ($M_{z,cat}$)

Terrain, over which the approach wind flows towards a structure, shall be assessed on the basis of the following category descriptions: (Section 4.2.1)

(a) *Category 1* – Exposed open terrain with few or no obstructions and water surfaces at serviceability wind speeds.

(b) *Category 2* – Water surfaces, open terrain, grassland with few, well-scattered obstructions having heights generally from 1.5m to 10m.

(c) *Category 3* – Terrain with numerous closely spaced obstructions 3m to 5m high such as areas of suburban housing.

(d) *Category 4* – Terrain with numerous large, high (10m to 30m high) and closely spaced obstructions such as large city centres and well-developed industrial complexes.

TERRAIN/HEIGHT MULTIPLIERS FOR GUST WIND SPEEDS IN FULLY DEVELOPED TERRAINS—SERVICEABILITY LIMIT STATE DESIGN—ALL REGIONS AND ULTIMATE LIMIT STATE—REGIONS A1 TO A7, W AND B

Height (z) m	Terrain/height multiplier ($M_{z,cat}$)			
	Terrain category 1	Terrain category 2	Terrain category 3	Terrain category 4
≤3	0.99	0.91	0.83	0.75
5	1.05	0.91	0.83	0.75
10	1.12	1.00	0.83	0.75
15	1.16	1.05	0.89	0.75
20	1.19	1.08	0.94	0.75
30	1.22	1.12	1.00	0.80
40	1.24	1.16	1.04	0.85
50	1.25	1.18	1.07	0.90
75	1.27	1.22	1.12	0.98
100	1.29	1.24	1.16	1.03
150	1.31	1.27	1.21	1.11
200	1.32	1.29	1.24	1.16
250	1.34	1.31	1.27	1.20
300	1.35	1.32	1.29	1.23
400	1.37	1.35	1.32	1.28
500	1.38	1.37	1.35	1.31

Table A.4(A) – $M_{z,cat}$ for Regions A and B (AS1170.2, 2002)

TERRAIN/HEIGHT MULTIPLIERS FOR GUST WIND SPEEDS IN FULLY DEVELOPED TERRAINS—ULTIMATE LIMIT STATE DESIGN—REGIONS C AND D ONLY

Height (z) m	Multiplier ($M_{z,cat}$)	
	Terrain categories 1 and 2	Terrain categories 3 and 4
≤3	0.90	0.80
5	0.95	0.80
10	1.00	0.89
15	1.07	0.95
20	1.13	1.05
30	1.20	1.15
40	1.25	1.25
50	1.29	1.29
75	1.35	1.35
≥100	1.40	1.40

Table A.4(B) – $M_{z,cat}$ for Regions C and D (AS1170.2, 2002)

A relocatable home park can expect category 3 winds, however it would be too safe to assume that category 2 winds wouldn't occur. A category 2.5 terrain was chosen, and, limiting this research to single storey units ($z \leq 3$m), the following results were obtained from Tables A.4(A) and A.4(B):

Regions A & B: $M_{z,cat} = (0.91+0.83)/2 = 0.87$
Regions C & D: $M_{z,cat} = (0.90+0.80)/2 = 0.85$

<u>Shielding Multiplier (M_s)</u>

It was assumed that no shielding occurs and therefore $M_s = 1.0$, which is a conservative approach.

<u>Topographic Multiplier (M_t)</u>

Again, it was assumed that relocatable home parks are likely to be designed on fairly even topography and therefore $M_t = 1.0$.

Site Wind Speeds ($V_{sit,\beta}$)

The product of the above figures as per equation A.1 gave the following site wind speeds for each region:

$V_{RegionA}$ = 45*1.0*(0.87*1.0*1.0) = 39.15 m/s
= 39.15*60*60/1000 = 140.94 kph

$V_{RegionB}$ = 57*1.0*(0.87*1.0*1.0) = 49.59 m/s
= 49.59*60*60/1000 = 178.52 kph

$V_{RegionC}$ = 69*1.0*(0.85*1.0*1.0) = 58.65 m/s
= 58.65*60*60/1000 = 211.14 kph

$V_{RegionD}$ = 88*1.0*(0.85*1.0*1.0) = 74.80 m/s
= 74.80*60*60/1000 = 269.28 kph

As the delivery trucks are expected to travel at no more than 110 kph, and would be subject to possible minor head winds, no regions can be omitted from the study. To clarify, if it was the case that Region A site wind speed was 60 kph, then there would be no use designing for such a small wind speed if it is going to be subject to stronger winds during the transportation phase.

Now, the design wind pressures (p) was determined for structures as follows:

$$p = (0.5\rho_{air})[V_{des,\theta}]^2 C_{fig} C_{dyn}$$

Eqn A.2

where p = design wind pressure acting normal to a surface
ρ_{air} = density of air, which shall be taken as 1.2 kg/m^3
$V_{des,\theta}$ = building orthogonal design wind speeds
C_{fig} = aerodynamic shape factor
C_{dyn} = dynamic response factor (=1.0 where structure isn't sensitive to wind)

ρ_{air}, $V_{des,\theta}$ and C_{dyn} are known, so all that's left was C_{fig} which for external pressures is determined by the following equation:

$$C_{fig} = C_{p,e} K_a K_c K_l K_p$$

Eqn A.3

where $C_{p,e}$ = external pressure coefficient

K_a = area reduction factor

K_c = combination factor

K_l = local pressure factor

K_p = permeable cladding reduction factor

As mentioned previously, internal pressures have a cancelling out effect on the design of the foundation and so will be ignored.

External Pressure Coefficient (Cp,e)

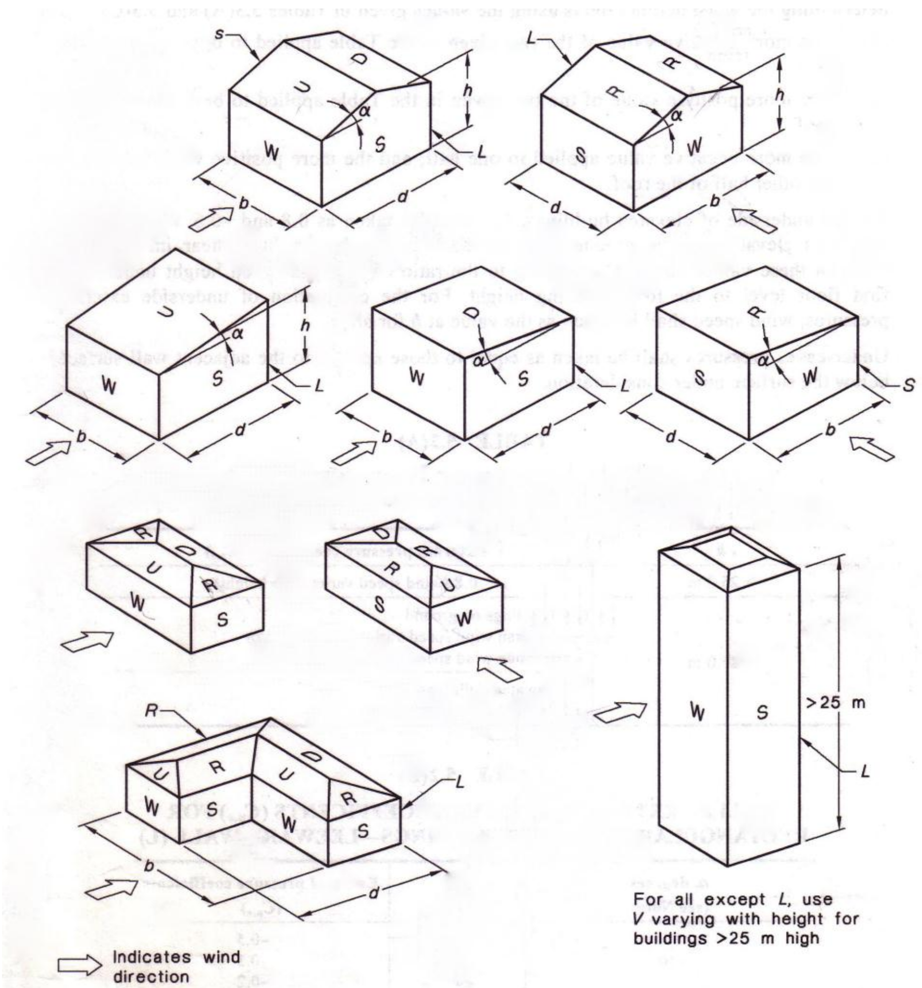


Figure A.1 – Parameters for Rectangular Enclosed Buildings (AS1170.2, 2002)

The windward wall pressure is required for the horizontal loading calculations. The windward wall is denoted W in Figure A.1. From Table A.5 the external pressure coefficient on the windward wall is 0.7.

WALLS—EXTERNAL PRESSURE COEFFICENTS ($C_{p,e}$) FOR RECTANGULAR ENCLOSED BUILDINGS—WINDWARD WALL (W)

h	External pressure coefficients ($C_{p,e}$)
> 25.0 m	0.8 (wind speed varies with height)
≤ 25.0 m	For buildings on ground— 0.8, when wind speed varies with height; or 0.7, when wind speed is taken for $z = h$
	For elevated buildings— 0.8 (wind speed taken at *h*)

Table A.5 – $C_{p,e}$ for Rectangular Enclosed Buildings – Windward Wall (AS1170.2, 2002)

<u>Combination Factor (K_c)</u>

As shown in Table A.6, design case (b) is assumed to occur, therefore K_c = 0.8.

ACTION COMBINATION FACTORS FOR WIND PRESSURE CONTRIBUTING FROM TWO OR MORE BUILDING SURFACES TO EFFECTS ON MAJOR STRUCTURAL ELEMENTS

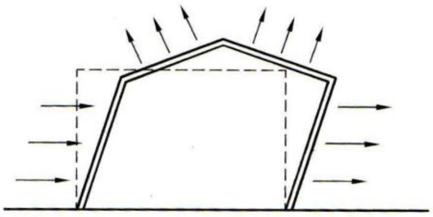
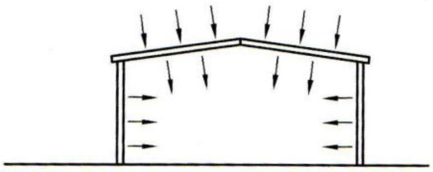
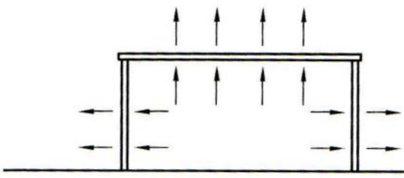
Design case	Combination factor (K_c)	Example diagrams
(a) Where wind action from any single surface contributes 75 percent or more to an action effect	1.0	—
(b) Pressures from windward and leeward walls in combination with positive or negative roof pressures	0.8	
(c) Positive pressures on roofs in combination with negative internal pressures (from a wall opening)	0.8	
(d) Negative pressures on roofs or walls in combination with positive internal pressures	0.95	
(e) All other cases	1.0	—

Table A.6 – Combination Factor (K_c) (AS1170.2, 2002)

Area Reduction Factor (K_a)

For other than roofs and sidewalls, K_a = 1.0

Local Pressure Factor (K_l)

For other than cladding elements, K_l = 1.0

Permeable Cladding Reduction Factor (K_p)

Assumed that K_p = 1.0 (conservative approach)

Aerodynamic Shape Factor (C_{fig})

The product of the above values as per equation A.3 gave:

C_{fig} = 0.7*1.0*0.8*1.0*1.0 = 0.56

Design Wind Pressure (p)

The final design wind pressures for each of the four regions, derived from equation A.2, will be used as the forces acting on the homes in the horizontal plane and were calculated as follows:

$p_{RegionA} = (0.5*1.2)*[39.15]^2*0.56*1.0 = 0.515$ kPa
$p_{RegionB} = (0.5*1.2)*[49.59]^2*0.56*1.0 = 0.826$ kPa
$p_{RegionC} = (0.5*1.2)*[58.65]^2*0.56*1.0 = 1.156$ kPa
$p_{RegionD} = (0.5*1.2)*[74.80]^2*0.56*1.0 = 1.880$ kPa

Roof Uplift

Using Figure 7.2, the uplift created during a 1:500yr windstorm in each region is calculated using the following equation:

$$\text{Uplift} = p \times C_{p,e} \times b \times d \times \cos\theta$$

Eqn A.1

where p = average roof uplift pressure (kPa)
$C_{p,e}$ = external pressure coefficient
b = total width of roof (m)
d = total depth of roof (m)
θ = roof pitch (^{o})

Uplift_A = 29.52 kN
Uplift_B = 47.36 kN
Uplift_C = 66.93 kN
Uplift_D = 107.88 kN

APPENDIX B – Vertical Load Calculations

Another load that is applicable to a relocatable home is the vertical load caused by its self-weight. The loads were taken from AS1170.1 and specifications provided by Norfolk Relocatable Homes Pty Ltd (Figure B.1) to approximate the weight of a standard sized home. Calculations are as follows:

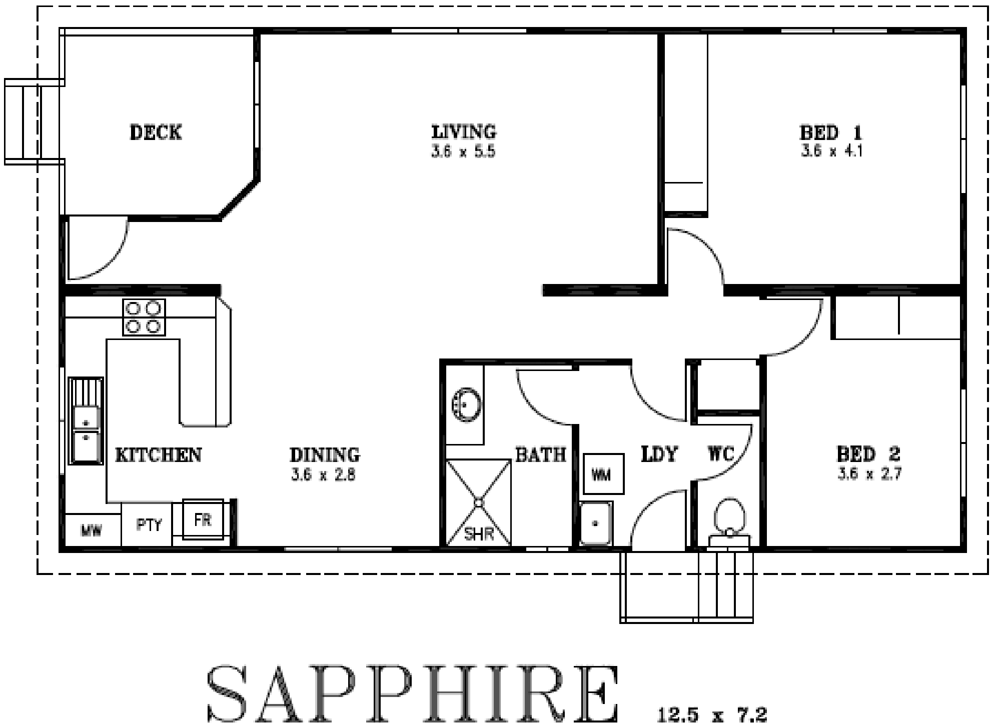


Figure B.1 – Typical Floor Plan (Norfolk Homes, 2010)

<u>House Weight</u>

Roof

Corrugated sheeting:	0.12 kPa
Fibro ceiling:	0.09 kPa
Timber framing:	0.06 kPa (assuming 600 spacing)
Sub Total	0.30 kPa used x 12.5 x 7 = 26.25 kN

Floor

145x45 joists @450cts:	0.10 kPa
19mm flooring:	0.14 kPa
4-12.5m Bearers:	0.31kN/m x 4 x 12.5 / (12.5 x 7) = 0.177kPa
Tiling/carpet etc:	0.10 kPa allowed
Sub Total	0.517 kPa x 12.5 x 7 = 45.23 kN

Walls

Studs:	0.04 kPa
Services:	0.01 kPa
2-10mm ply sheeting:	0.20 kPa
Sub Total	0.25 kPax2.4m high x67.7m walls = 40.62 kN

Total = 26.25 + 45.23 + 40.62 kN

= 112.1 kN or 11210 kg

APPENDIX C – Region A: 270mm Stack Analysis

Figure C.1 represents a typical dry stack block requiring a tie-down bolt. Calculations in the following Appendices will determine whether or not these tie-downs are required for each of the individual footing heights and wind regions.

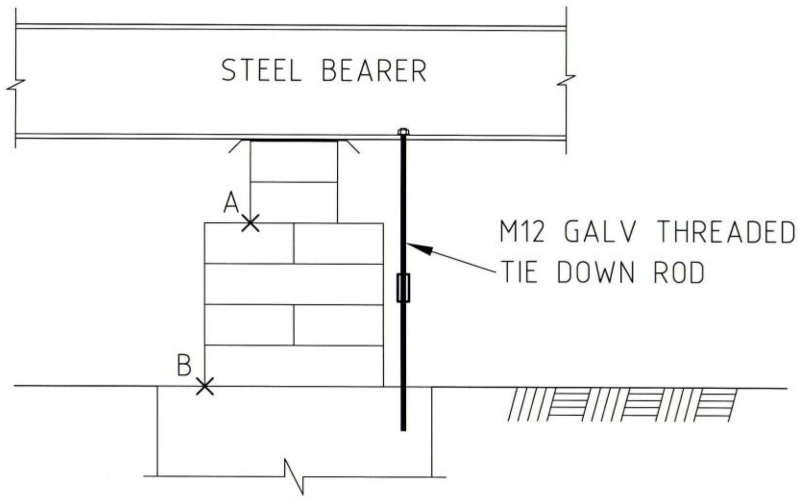


Figure C.1 – Typical Tie-down

A general rule can be applied throughout the analyses, whereby if the moment of overturning (M_{OT}) is greater than the moment of resistance (M_R) then bracing is required. Likewise, if $M_{OT} < M_R$ then no bracing is required.

Check for Tie-downs:

Using engineering statics and the theory of equilibrium, check for overturning about point A (2 courses 190x190x90):

M_{OT} = [27.38 kN (Table 7.1) / 24] * 0.18m = 0.205 kNm

M_R = [3.44 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.342 kNm

$M_{OT} < M_R \therefore$ OK (won't overturn about point A)

Now check about point B (2 courses 190x190x90 + 1 course 390x390x90):

M_{OT} = [27.38 kN / 24] * 0.27m = 0.308 kNm

M_R = [3.44 kN + 0.156 kN + (1*0.329 kN)] * 0.39/2 = 0.765 kNm

$M_{OT} < M_R \therefore$ OK (won't overturn about point B)

Therefore, no tie-down required.

If the force created by the cross wind is greater than the resisting forces (frictional resistance at the bearer to antcap interface), then cross bracing is required to provide a load path for the force to travel into the foundation. A typical cross bracing layout is shown below in Figure C.2.

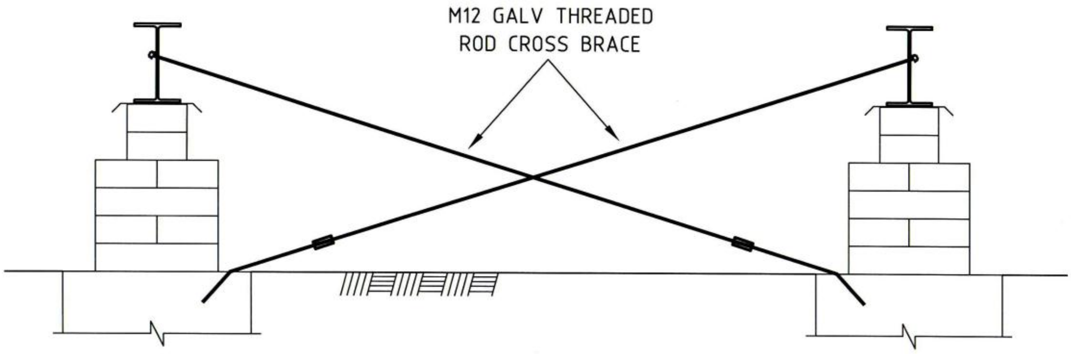


Figure C.2 – Typical Cross Bracing

<u>Check for cross bracing:</u>

M_R = [24.77 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.201 kNm

M_{OT} = [27.38 kN / 24] * 0.27m = 0.308 kNm

$M_{OT} > M_R$ $\therefore$ Cross bracing required

The number of bays required is:

27.38 kN / 26.907 kN (Section 7.6) = 1.02 $\therefore$ Use 2 bays

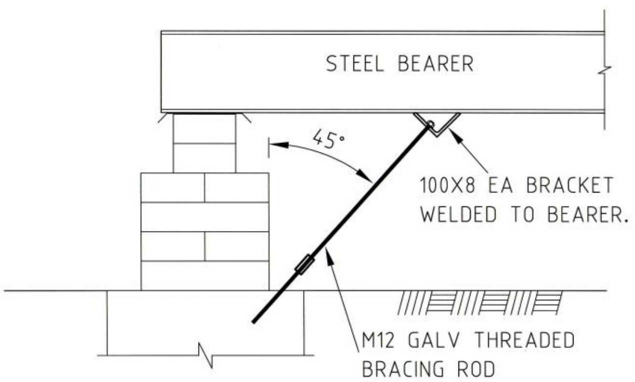


Figure C.3 – Typical Longitudinal Bracing

<u>Check for longitudinal bracing:</u>

As mentioned in Section 7.3, the longitudinal racking force was less than the resisting forces available and therefore no longitudinal bracing is required.

APPENDIX D – Region A: 540mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [29.34 kN (Table 7.1) / 24] * 0.18m = 0.220 kNm

M_R = [3.44 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.342 kNm

$M_{OT} < M_R$ $\therefore$ OK (won't overturn about point A)

Now check about point B (2 courses 190x190x90 + 4 courses 390x390x90):

M_{OT} = [29.34 kN / 24] * 0.54m = 0.660 kNm

M_R = [3.44 kN + 0.156 kN + (4*0.329 kN)] * 0.39/2 = 0.957 kNm

$M_{OT} < M_R$ $\therefore$ OK (won't overturn about point B)

Therefore, no tie-down required.

Check for cross bracing:

M_R = [24.77 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.201 kNm

M_{OT} = [29.34 kN / 24] * 0.54m = 0.660 kNm

$M_{OT} > M_R$ $\therefore$ Cross bracing required

The number of bays required is:

29.34 kN / 26.635 kN (Section 7.6) = 1.10 $\therefore$ Use 2 bays

Check for longitudinal bracing:

As mentioned in Section 7.3, the longitudinal racking force was less than the resisting forces available and therefore no longitudinal bracing is required.

APPENDIX E – Region A: 810mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [31.30 kN (Table 7.1) / 24] * 0.18m = 0.235 kNm

M_R = [3.44 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.342 kNm

$M_{OT} < M_R$ ∴ OK (won't overturn about point A)

Now check about point B (2 courses 190x190x90 + 7 courses 390x390x90):

M_{OT} = [31.30 kN / 24] * 0.81m = 1.056 kNm

M_R = [3.44 kN + 0.156 kN + (7*0.329 kN)] * 0.39/2 = 1.15 kNm

$M_{OT} < M_R$ ∴ OK (won't overturn about point B)

Therefore, no tie-down required.

Check for cross bracing:

M_R = [24.77 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.201 kNm

M_{OT} = [31.30 kN / 24] * 0.81m = 1.056 kNm

$M_{OT} > M_R$ ∴ Cross bracing required

The number of bays required is:

31.30 kN / 26.199 kN (Section 7.6) = 1.20 ∴ Use 2 bays

Check for longitudinal bracing:

As mentioned in Section 7.3, the longitudinal racking force was less than the resisting forces available and therefore no longitudinal bracing is required.

APPENDIX F – Region A: 1080mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [33.26 kN (Table 7.1) / 24] * 0.18m = 0.249 kNm

M_R = [3.44 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.342 kNm

$M_{OT} < M_R$ $\therefore$ OK (won't overturn about point A)

Now check about point B (2 courses 190x190x90 + 10 courses 390x390x90):

M_{OT} = [33.26 kN / 24] * 1.08m = 1.497 kNm

M_R = [3.44 kN + 0.156 kN + (10*0.329 kN)] * 0.39/2 = 1.342 kNm

$M_{OT} > M_R$ $\therefore$ Tie-down required

With tie-down:

Check for overturning about point B:

M_R = [(24.77 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.466 kNm

M_{OT} = [33.26 kN (Table 7.1) / x] * 1.08m < 5.466 kNm

x = 6.57 $\therefore$ 7 stumps required (use 8 around perimeter)

Check for cross bracing:

M_R = [24.77 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.201 kNm

M_{OT} = [33.26 kN / 24] * 1.08m = 1.497 kNm

$M_{OT} > M_R$ $\therefore$ Cross bracing required

The number of bays required is:

33.26 kN / 25.622 kN (Section 7.6) = 1.30 $\therefore$ Use 2 bays

Check for longitudinal bracing:

As mentioned in Section 7.3, the longitudinal racking force was less than the resisting forces available and therefore no longitudinal bracing is required.

APPENDIX G – Region B: 270mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [43.92 kN (Table 7.1) / 24] * 0.18m = 0.329 kNm

M_R = [2.70 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.271 kNm

$M_{OT} > M_R$ ∴ Tie-down required

Now check about point B (2 courses 190x190x90 + 1 course 390x390x90):

M_{OT} = [43.92 kN / 24] * 0.27m = 0.494 kNm

M_R = [2.70 kN + 0.156 kN + (1*0.329 kN)] * 0.39/2 = 0.621 kNm

$M_{OT} < M_R$ ∴ OK, however tie-downs are required to stop overturning at point A.

With tie-down:

Check for overturning about point A:

M_R = [(19.42 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.642 kNm

M_{OT} = [43.92 kN (Table 7.1) / x] * 0.18m < 2.642 kNm

x = 2.99 ∴ 3 stumps required (use 2 each side)

Check for cross bracing:

M_R = [19.42 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.158 kNm

M_{OT} = [43.92 kN / 24] * 0.27m = 0.494 kNm

$M_{OT} > M_R$ ∴ Cross bracing required

The number of bays required is:

43.92 kN / 26.907 kN (Section 7.6) = 1.632 ∴ Use 2 bays

Check for longitudinal bracing:

The longitudinal racking force is 22.96 kN (Table 7.2). It was determined in Section 7.7 that one longitudinal rod will achieve 19.092 kN tensile strength. Therefore 2 sets of longitudinal bracing is required in each direction in case of wind in either longitudinal direction, giving 38.184 kN capacity > 22.96 kN ∴ OK.

APPENDIX H – Region B: 540mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [47.07 kN (Table 7.1) / 24] * 0.18m = 0.353 kNm

M_R = [2.70 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.271 kNm

$M_{OT} > M_R \therefore$ Tie-down required

Now check about point B (2 courses 190x190x90 + 4 courses 390x390x90):

M_{OT} = [47.07 kN / 24] * 0.54m = 1.059 kNm

M_R = [2.70 kN + 0.156 kN + (4*0.329 kN)] * 0.39/2 = 0.813 kNm

$M_{OT} > M_R \therefore$ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(19.42 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.642 kNm

M_{OT} = [47.07 kN (Table 7.1) / x] * 0.18m < 2.642 kNm

x = 3.21 $\therefore$ 4 stumps required

Check for overturning about point B:

M_R = [(19.42 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.423 kNm

M_{OT} = [47.07 kN (Table 7.1) / x] * 0.54m < 5.423 kNm

x = 4.69 $\therefore$ 5 stumps required (use 3 each side)

The maximum value of x must be used, therefore adopt 6 stumps with tie-downs.

Check for cross bracing:

M_R = [19.42 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.158 kNm

M_{OT} = [47.07 kN / 24] * 0.54m = 1.059 kNm

$M_{OT} > M_R \therefore$ Cross bracing required

The number of bays required is:

47.07 kN / 26.635 kN (Section 7.6) = 1.767 $\therefore$ Use 2 bays

Check for longitudinal bracing:

The longitudinal racking force is 24.73 kN (Table 7.2). It was determined in Section 7.7 that one longitudinal rod will achieve 19.092 kN tensile strength. Therefore 2 sets of longitudinal bracing is required in each direction in case of wind in either longitudinal direction, giving 38.184 kN capacity > 24.73 kN $\therefore$ OK.

APPENDIX I – Region B: 810mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [50.21 kN (Table 7.1) / 24] * 0.18m = 0.377 kNm

M_R = [2.70 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.271 kNm

$M_{OT} > M_R$ $\therefore$ Tie-down required

Now check about point B (2 courses 190x190x90 + 7 courses 390x390x90):

M_{OT} = [50.21 kN / 24] * 0.81m = 1.695 kNm

M_R = [2.70 kN + 0.156 kN + (7*0.329 kN)] * 0.39/2 = 1.005 kNm

$M_{OT} > M_R$ $\therefore$ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(19.42 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.642 kNm

M_{OT} = [50.21 kN (Table 7.1) / x] * 0.18m < 2.642 kNm

x = 3.42 $\therefore$ 4 stumps required

Check for overturning about point B:

M_R = [(19.42 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.423 kNm

M_{OT} = [50.21 kN (Table 7.1) / x] * 0.81m < 5.423 kNm

x = 7.50 $\therefore$ 8 stumps required

The maximum value of x must be used, therefore adopt 8 stumps with tie-downs.

Check for cross bracing:

M_R = [19.42 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.158 kNm

M_{OT} = [50.21 kN / 24] * 0.81m = 1.695 kNm

$M_{OT} > M_R$ $\therefore$ Cross bracing required

The number of bays required is:

50.21 kN / 26.199 kN (Section 7.6) = 1.916 $\therefore$ Use 2 bays

Check for longitudinal bracing:

The longitudinal racking force is 26.49 kN (Table 7.2). It was determined in Section 7.7 that one longitudinal rod will achieve 19.092 kN tensile strength. Therefore 2 sets of longitudinal bracing is required in each direction in case of wind in either longitudinal direction, giving 38.184 kN capacity > 26.49 kN $\therefore$ OK.

APPENDIX J – Region B: 1080mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [53.36 kN (Table 7.1) / 24] * 0.18m = 0.400 kNm

M_R = [2.70 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.271 kNm

$M_{OT} > M_R$ ∴ Tie-down required

Now check about point B (2 courses 190x190x90 + 10 courses 390x390x90):

M_{OT} = [53.36 kN / 24] * 1.08m = 2.401 kNm

M_R = [2.70 kN + 0.156 kN + (10*0.329 kN)] * 0.39/2 = 1.197 kNm

$M_{OT} > M_R$ ∴ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(19.42 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.642 kNm

M_{OT} = [53.36 kN (Table 7.1) / x] * 0.18m < 2.642 kNm

x = 3.64 ∴ 4 stumps required

Check for overturning about point B:

M_R = [(19.42 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.423 kNm

M_{OT} = [53.36 kN (Table 7.1) / x] * 1.08m < 5.423 kNm

x = 10.62 ∴ 11 stumps required (use 6 each side)

The maximum value of x must be used therefore adopt 12 stumps with tie-downs

Check for cross bracing:

M_R = [19.42 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.158 kNm

M_{OT} = [53.36 kN / 24] * 1.08m = 2.401 kNm

$M_{OT} > M_R$ ∴ Cross bracing required

The number of bays required is:

53.36 kN / 25.622 kN (Section 7.6) = 2.083 ∴ Use 3 bays

Check for longitudinal bracing:

The longitudinal racking force is 28.25 kN (Table 7.2). It was determined in Section 7.7 that one longitudinal rod will achieve 19.092 kN tensile strength. Therefore 2 sets of longitudinal bracing is required in each direction in case of wind in either longitudinal direction, giving 38.184 kN capacity > 28.25 kN ∴ OK.

APPENDIX K – Region C: 270mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [62.06 kN (Table 7.1) / 24] * 0.18m = 0.465 kNm

M_R = [1.88 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.193 kNm

$M_{OT} > M_R \therefore$ Tie-down required

Now check about point B (2 courses 190x190x90 + 1 course 390x390x90):

M_{OT} = [62.06 kN / 24] * 0.27m = 0.698 kNm

M_R = [1.88 kN + 0.156 kN + (1*0.329 kN)] * 0.39/2 = 0.461 kNm

$M_{OT} > M_R \therefore$ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(13.55 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.619 kNm

M_{OT} = [62.06 kN (Table 7.1) / x] * 0.18m < 2.619 kNm

x = 4.27 $\therefore$ 5 stumps required (use 3 each side)

Check for overturning about point B:

M_R = [(13.55 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.375 kNm

M_{OT} = [62.06 kN (Table 7.1) / x] * 0.27m < 5.375 kNm

x = 3.12 $\therefore$ 4 stumps required

The maximum value of x must be used therefore adopt 6 stumps with tie-downs

Check for cross bracing:

M_R = [13.55 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.110 kNm

M_{OT} = [62.06 kN / 24] * 0.27m = 0.698 kNm

$M_{OT} > M_R \therefore$ Cross bracing required

The number of bays required is:

62.06 kN / 26.907 kN (Section 7.6) = 2.306 $\therefore$ Use 3 bays

Check for longitudinal bracing:

The longitudinal racking force is 32.45 kN (Table 7.2). It was determined in Section 7.7 that one longitudinal rod will achieve 19.092 kN tensile strength. Therefore 2 sets of longitudinal bracing is required in each direction in case of wind in either longitudinal direction, giving 38.184 kN capacity > 32.45 kN $\therefore$ OK.

APPENDIX L – Region C: 540mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [66.51 kN (Table 7.1) / 24] * 0.18m = 0.499 kNm

M_R = [1.88 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.193 kNm

$M_{OT} > M_R$ ∴ Tie-down required

Now check about point B (2 courses 190x190x90 + 4 courses 390x390x90):

M_{OT} = [66.51 kN / 24] * 0.54m = 1.496 kNm

M_R = [1.88 kN + 0.156 kN + (4*0.329 kN)] * 0.39/2 = 0.653 kNm

$M_{OT} > M_R$ ∴ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(13.55 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.619 kNm

M_{OT} = [66.51 kN (Table 7.1) / x] * 0.18m < 2.619 kNm

x = 4.57 ∴ 5 stumps required

Check for overturning about point B:

M_R = [(13.55 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.375 kNm

M_{OT} = [66.51 kN (Table 7.1) / x] * 0.54m < 5.375 kNm

x = 6.68 ∴ 7 stumps required (use 4 each side)

The maximum value of x must be used therefore adopt 8 stumps with tie-downs

Check for cross bracing:

M_R = [13.55 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.110 kNm

M_{OT} = [66.51 kN / 24] * 0.54m = 1.496 kNm

$M_{OT} > M_R$ ∴ Cross bracing required

The number of bays required is:

66.51 kN / 26.635 kN (Section 7.6) = 2.497 ∴ Use 3 bays

Check for longitudinal bracing:

The longitudinal racking force is 34.94 kN (Table 7.2). It was determined in Section 7.7 that one longitudinal rod will achieve 19.092 kN tensile strength. Therefore 2 sets of longitudinal bracing is required in each direction in case of wind in either longitudinal direction, giving 38.184 kN capacity > 34.94 kN ∴ OK.

APPENDIX M – Region C: 810mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [70.95 kN (Table 7.1) / 24] * 0.18m = 0.532 kNm

M_R = [1.88 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.193 kNm

$M_{OT} > M_R$ ∴ Tie-down required

Now check about point B (2 courses 190x190x90 + 7 courses 390x390x90):

M_{OT} = [70.95 kN / 24] * 0.81m = 2.395 kNm

M_R = [1.88 kN + 0.156 kN + (7*0.329 kN)] * 0.39/2 = 0.846 kNm

$M_{OT} > M_R$ ∴ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(13.55 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.619 kNm

M_{OT} = [70.95 kN (Table 7.1) / x] * 0.18m < 2.619 kNm

x = 4.88 ∴ 5 stumps required

Check for overturning about point B:

M_R = [(13.55 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.375 kNm

M_{OT} = [70.95 kN (Table 7.1) / x] * 0.81m < 5.375 kNm

x = 10.69 ∴ 11 stumps required (use 6 each side)

The maximum value of x must be used therefore adopt 12 stumps with tie-downs

Check for cross bracing:

M_R = [13.55 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.110 kNm

M_{OT} = [70.95 kN / 24] * 0.81m = 2.395 kNm

$M_{OT} > M_R$ ∴ Cross bracing required

The number of bays required is:

70.95 kN / 26.199 kN (Section 7.6) = 2.708 ∴ Use 3 bays

Check for longitudinal bracing:

The longitudinal racking force is 37.43 kN (Table 7.2). It was determined in Section 7.7 that one longitudinal rod will achieve 19.092 kN tensile strength. Therefore 2 sets of longitudinal bracing is required in each direction in case of wind in either longitudinal direction, giving 38.184 kN capacity > 37.43 kN ∴ OK.

APPENDIX N – Region C: 1080mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [75.40 kN (Table 7.1) / 24] * 0.18m = 0.566 kNm

M_R = [1.88 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.193 kNm

$M_{OT} > M_R$ ∴ Tie-down required

Now check about point B (2 courses 190x190x90 + 10 courses 390x390x90):

M_{OT} = [75.40 kN / 24] * 1.08m = 3.393 kNm

M_R = [1.88 kN + 0.156 kN + (10*0.329 kN)] * 0.39/2 = 1.038 kNm

$M_{OT} > M_R$ ∴ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(13.55 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.619 kNm

M_{OT} = [75.40 kN (Table 7.1) / x] * 0.18m < 2.619 kNm

x = 5.18 ∴ 6 stumps required

Check for overturning about point B:

M_R = [(13.55 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.375 kNm

M_{OT} = [75.40 kN (Table 7.1) / x] * 1.08m < 5.375 kNm

x = 15.15 ∴ 16 stumps required

The maximum value of x must be used therefore adopt 16 stumps with tie-downs

Check for cross bracing:

M_R = [13.55 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.110 kNm

M_{OT} = [75.40 kN / 24] * 1.08m = 3.393 kNm

$M_{OT} > M_R$ ∴ Cross bracing required

The number of bays required is:

75.40 kN / 25.622 kN (Section 7.6) = 2.943 ∴ Use 3 bays

Check for longitudinal bracing:

The longitudinal racking force is 39.92 kN (Table 7.2), which is greater than the capacity of 2 sets of longitudinal bracing in each direction (38.184 kN). Before increasing the bracing to 3 sets, it is worth checking if the frictional resistance in the dry stack can handle the extra required bracing.

The remaining force is 39.92 – 38.184 = 1.74 kN

The four corner stumps already have longitudinal bracing, so the self-weight of the home and remaining 20 stumps need to be able to handle the extra racking force.

1.74 kN / 20 stumps = 0.087 kN / stump required

M_{OT} = 0.087 kN * 1.08m = 0.094 kNm

M_R = [1.88 kN + 0.156 kN + (10*0.329 kN)] * 0.39/2 = 1.038 kNm

$M_{OT} < M_R$ $\therefore$ OK, stumps can handle extra racking shear.

APPENDIX O – Region D: 270mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [100.05 kN (Table 7.1) / 24] * 0.18m = 0.750 kNm

M_R = [0.18 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.032 kNm

$M_{OT} > M_R$ ∴ Tie-down required

Now check about point B (2 courses 190x190x90 + 1 course 390x390x90):

M_{OT} = [100.05 kN / 24] * 0.27m = 1.126 kNm

M_R = [0.18 kN + 0.156 kN + (1*0.329 kN)] * 0.39/2 = 0.129 kNm

$M_{OT} > M_R$ ∴ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(1.26 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.570 kNm

M_{OT} = [100.05 kN (Table 7.1) / x] * 0.18m < 2.570 kNm

x = 7.00 ∴ 7 stumps required (use 4 each side)

Check for overturning about point B:

M_R = [(1.26 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.275 kNm

M_{OT} = [100.05 kN (Table 7.1) / x] * 0.27m < 5.275 kNm

x = 5.12 ∴ 6 stumps required

The maximum value of x must be used therefore adopt 8 stumps with tie-downs

Check for cross bracing:

M_R = [1.26 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.010 kNm

M_{OT} = [100.05 kN / 24] * 0.27m = 1.126 kNm

$M_{OT} > M_R$ ∴ Cross bracing required

The number of bays required is:

100.05 kN / 26.907 kN (Section 7.6) = 3.718 ∴ Use 4 bays

Check for longitudinal bracing:

The longitudinal racking force is 52.31 kN (Table 7.2), which is greater than the capacity of 2 sets of longitudinal bracing in each direction (38.184 kN). Before increasing the bracing to 3 sets, it is worth checking if the frictional resistance in the dry stack can handle the extra required bracing.

The remaining force is 52.31 – 38.184 = 14.13 kN

The four corner stumps already have longitudinal bracing, so the self-weight of the home and remaining 20 stumps need to be able to handle the extra racking force.

14.13 kN / 20 stumps = 0.706 kN / stump required

M_{OT} = 0.706 kN * 0.27m = 0.191 kNm

M_R = [0.18 kN + 0.156 kN + (1*0.329 kN)] * 0.39/2 = 0.129 kNm

$M_{OT} > M_R$ $\therefore$ Stumps alone can't handle extra racking shear.

The remaining moment is 0.191 – 0.129 = 0.062 kNm. Tie-downs haven't been taken into consideration for longitudinal racking, therefore check to see if they can handle the extra moment:

M_R = 27 kN * 0.39/2 = 5.275 kNm > 0.062 kNm $\therefore$ OK

APPENDIX P – Region D: 540mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [107.21 kN (Table 7.1) / 24] * 0.18m = 0.804 kNm

M_R = [0.18 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.032 kNm

$M_{OT} > M_R \therefore$ Tie-down required

Now check about point B (2 courses 190x190x90 + 4 courses 390x390x90):

M_{OT} = [107.21 kN / 24] * 0.54m = 2.412 kNm

M_R = [0.18 kN + 0.156 kN + (4*0.329 kN)] * 0.39/2 = 0.321 kNm

$M_{OT} > M_R \therefore$ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(1.26 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.570 kNm

M_{OT} = [107.21 kN (Table 7.1) / x] * 0.18m < 2.570 kNm

x = 7.51 $\therefore$ 8 stumps required

Check for overturning about point B:

M_R = [(1.26 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.275 kNm

M_{OT} = [107.21 kN (Table 7.1) / x] * 0.54m < 5.275 kNm

x = 10.98 $\therefore$ 11 stumps required (use 6 each side)

The maximum value of x must be used therefore adopt 12 stumps with tie-downs

Check for cross bracing:

M_R = [1.26 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.010 kNm

M_{OT} = [107.21 kN / 24] * 0.54m = 2.412 kNm

$M_{OT} > M_R \therefore$ Cross bracing required

The number of bays required is:

107.21 kN / 26.635 kN (Section 7.6) = 4.00 $\therefore$ Use 4 bays

Check for longitudinal bracing:

The longitudinal racking force is 56.32 kN (Table 7.2), which is greater than the capacity of 2 sets of longitudinal bracing in each direction (38.184 kN). Before increasing the bracing to 3 sets, it is worth checking if the frictional resistance in the dry stack can handle the extra required bracing.

The remaining force is 56.32 – 38.184 = 18.14 kN

The four corner stumps already have longitudinal bracing, so the self-weight of the home and remaining 20 stumps need to be able to handle the extra racking force.

18.14 kN / 20 stumps = 0.907 kN / stump required

M_{OT} = 0.907 kN * 0.54m = 0.490 kNm

M_R = [0.18 kN + 0.156 kN + (4*0.329 kN)] * 0.39/2 = 0.321 kNm

$M_{OT} > M_R$ $\therefore$ Stumps alone can't handle extra racking shear.

The remaining moment is 0.490 – 0.321 = 0.169 kNm. Tie-downs haven't been taken into consideration for longitudinal racking, therefore check to see if they can handle the extra moment:

M_R = 27 kN * 0.39/2 = 5.275 kNm > 0.169 kNm $\therefore$ OK

APPENDIX Q – Region D: 810mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [114.38 kN (Table 7.1) / 24] * 0.18m = 0.858 kNm

M_R = [0.18 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.032 kNm

$M_{OT} > M_R$ ∴ Tie-down required

Now check about point B (2 courses 190x190x90 + 7 courses 390x390x90):

M_{OT} = [114.38 kN / 24] * 0.81m = 3.860 kNm

M_R = [0.18 kN + 0.156 kN + (7*0.329 kN)] * 0.39/2 = 0.513 kNm

$M_{OT} > M_R$ ∴ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(1.26 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.570 kNm

M_{OT} = [114.38 kN (Table 7.1) / x] * 0.18m < 2.570 kNm

x = 8.01 ∴ 9 stumps required

Check for overturning about point B:

M_R = [(1.26 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.275 kNm

M_{OT} = [114.38 kN (Table 7.1) / x] * 0.81m < 5.275 kNm

x = 17.56 ∴ 18 stumps required

The maximum value of x must be used therefore adopt 18 stumps with tie-downs

Check for cross bracing:

M_R = [1.26 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.010 kNm

M_{OT} = [114.38 kN / 24] * 0.81m = 3.860 kNm

$M_{OT} > M_R$ ∴ Cross bracing required

The number of bays required is:

114.38 kN / 26.199 kN (Section 7.6) = 4.37 ∴ Use 5 bays

Check for longitudinal bracing:

The longitudinal racking force is 60.34 kN (Table 7.2), which is greater than the capacity of 2 sets of longitudinal bracing in each direction (38.184 kN). Before increasing the bracing to 3 sets, it is worth checking if the frictional resistance in the dry stack can handle the extra required bracing.

The remaining force is 60.34 – 38.184 = 22.16 kN

The four corner stumps already have longitudinal bracing, so the self-weight of the home and remaining 20 stumps need to be able to handle the extra racking force.

22.16 kN / 20 stumps = 1.108 kN / stump required

M_{OT} = 1.108 kN * 0.81m = 0.897 kNm

M_R = [0.18 kN + 0.156 kN + (7*0.329 kN)] * 0.39/2 = 0.513 kNm

$M_{OT} > M_R$ $\therefore$ Stumps alone can't handle extra racking shear.

The remaining moment is 0.897 – 0.513 = 0.384 kNm. Tie-downs haven't been taken into consideration for longitudinal racking, therefore check to see if they can handle the extra moment:

M_R = 27 kN * 0.39/2 = 5.275 kNm > 0.384 kNm $\therefore$ OK

APPENDIX R – Region D: 1080mm Stack Analysis

Check for Tie-downs:

Check for overturning about point A (2 courses 190x190x90):

M_{OT} = [121.54 kN (Table 7.1) / 24] * 0.18m = 0.912 kNm

M_R = [0.18 kN (Section 7.2) + 0.156 kN (Section 7.4)] * 0.19/2 = 0.032 kNm

$M_{OT} > M_R$ ∴ Tie-down required

Now check about point B (2 courses 190x190x90 + 10 courses 390x390x90):

M_{OT} = [121.54 kN / 24] * 1.08m = 5.469 kNm

M_R = [0.18 kN + 0.156 kN + (10*0.329 kN)] * 0.39/2 = 0.705 kNm

$M_{OT} > M_R$ ∴ Tie-down required

With tie-down:

Check for overturning about point A:

M_R = [(1.26 kN (Section 7.3) /24 stumps) + 27 kN] * 0.19/2 = 2.570 kNm

M_{OT} = [121.54 kN (Table 7.1) / x] * 0.18m < 2.570 kNm

x = 8.51 ∴ 9 stumps required

Check for overturning about point B:

M_R = [(1.26 kN (Section 7.3) /24 stumps) + 27 kN] * 0.39/2 = 5.275 kNm

M_{OT} = [121.54 kN (Table 7.1) / x] * 1.08m < 5.275 kNm

x = 24.88 ∴ All 24 stumps required

The maximum value of x must be used therefore adopt 24 stumps with tie-downs. More than 24 stumps are required, so first of all determine the capacity of 24 stumps: [121.54 kN / 24] * 1.08m = 5.469 kNm

Therefore, the remaining moment to be taken up by stumps is 5.469 – 5.275 = 0.194 kNm.

M_{OT} = 0.194 kNm / 24 stumps = 0.008 kNm / stump

M_R = [0.18 kN + 0.156 kN + (10*0.329 kN)] * 0.39/2 = 0.705 kNm

$M_{OT} < M_R$ ∴ OK, stumps can handle extra moment.

Check for cross bracing:

M_R = [1.26 kN (Section 7.3) / 24 stumps] * 0.39/2 = 0.010 kNm

M_{OT} = [121.54 kN / 24] * 1.08m = 5.469 kNm

$M_{OT} > M_R$ ∴ Cross bracing required

The number of bays required is:

121.54 kN / 25.622 kN (Section 7.6) = 4.74 $\therefore$ Use 5 bays

<u>Check for longitudinal bracing:</u>

The longitudinal racking force is 64.35 kN (Table 7.2), which is greater than the capacity of 2 sets of longitudinal bracing in each direction (38.184 kN). Before increasing the bracing to 3 sets, it is worth checking if the frictional resistance in the dry stack can handle the extra required bracing.

The remaining force is 64.35 – 38.184 = 26.17 kN

The four corner stumps already have longitudinal bracing, so the self-weight of the home and remaining 20 stumps need to be able to handle the extra racking force.

26.17 kN / 20 stumps = 1.309 kN / stump required

M_{OT} = 1.309 kN * 1.08m = 1.414 kNm

M_R = [0.18 kN + 0.156 kN + (10*0.329 kN)] * 0.39/2 = 0.705 kNm

$M_{OT} > M_R$ $\therefore$ Stumps alone can't handle extra racking shear.

The remaining moment is 1.414 – 0.705 = 0.709 kNm. Tie-downs haven't been taken into consideration for longitudinal racking, therefore check to see if they can handle the extra moment:

M_R = 27 kN * 0.39/2 = 5.275 kNm > 0.709 kNm $\therefore$ OK

APPENDIX S – Project Specification

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

ENG 4111/4112 Research Project

PROJECT SPECIFICATION

FOR: **SCOTT FENN**

TOPIC: INVESTIGATION INTO RELOCATABLE HOME DESIGN AND CONSTRUCTION

SUPERVISOR: Dr. Yan Zhuge

ENROLMENT: ENG 4111 – S1, 2011;
ENG 4112 – S2, 2011

PROJECT AIM: This project aims to investigate alternative methods of design and construction of relocatable homes in the QLD region.

PROGRAMME: Issue B, 26th October 2011

1. Research background information on current methods of design and construction of relocatable home in Queensland and other places in Australia.

2. Investigate the design and construction issues with the footing design of relocatable homes.

3. Critically analyse the alternative support system method of dry stack blocks with respect to load bearing, bracing and tie downs.

4. Submit an academic dissertation on the research.

5. Develop recommendation tables for four wind regions, reiterated with multiple footing heights.

As time permits:

6. Research the design and construction aspects for transportation of the relocatable home including loading and unloading, and craneage.

7. Provide a case study comparing local design and construction methods with those used overseas.

AGREED: _________________(student) _________________(supervisor)

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