THE RACKING RESISTANCE OF TIMBER FRAME WALLS ASSESSED BY EXPERIMENTAL AND ANALYTICAL TECHNIQUES

A thesis submitted for the degree of Doctor of Philosophy

by

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July 1987

VOLUME I
SYNOPSIS

This investigation begins with an historical survey of timber frame walls. The modern platform frame system is described in detail and the relevant experimental and analytical work reviewed. The development of a racking test method which allows for variability in vertical load is detailed. This test has been used to obtain data on the racking resistance of timber frame walls and the investigation indicates how these data have been used in formulating two empirical design methods for practical situations. The use of a computer based structural analysis program to model the test data, which could ultimately lead to a more sophisticated design approach, is also considered.

Overleaf:

Frontispiece: The University of Surrey Racking Test Rig
To my Father, who sadly never lived to see his influence on my life.
Acknowledgements

My supervisors have nominally been numerous; if it has been my project that has driven them from the University here is my opportunity to apologise. Dr. Len Hollaway may thus be likened to Catherine of Aragon, for he has outlived the thesis. I am deeply indebted to him for his help, patience and counselling.

Initially my thanks should go to Professor Stafford Smith and Guildway Ltd., in particular their structural engineer, Brian Fordham, for starting me working on timber frame walls. Latterly I owe a great debt to Professor Noel Simons for giving me the opportunity to finish off the experimental work and write up the thesis. The finance for the extra tests has come from contract testing of timber framed walls. I would like to thank all the firms who have entrusted me with their work firstly for the finance but secondly in allowing me to make further use of the test results. In particular I would like to single out The Karlit Sales Co. and FIDOR for special mention.

Within the timber frame industry people have been very helpful and kind. I would like to thank especially the following: Dr. Bill Chan for inviting me on to the BSI Timber Frame Sub-Committee; Peter Grimsdale, Alan Mayo and John Stringer for, in different ways, keeping alive my interest in research in timber frame walls, finally all the people worldwide who helped make my study tour on timber and timber frame housing so informative and enjoyable, to represent them all I have singled out Dr. Chris Steida of COFI in Vancouver for his particular kindness and generosity.

In the laboratory praise above all is due to Peter Haynes who has worked with me for seven years carrying out virtually all the timber frame wall tests.
His dedication and enthusiasm has been wonderful and will always be remembered. Thanks are also due to Mike Jackman for helping with the nail tests and all the other jobs needed to round off the test work. Harry Wickens has also suffered long and hard from work on timber frame walls. I am indebted to him for processing much of the raw data from the wall tests into something useable, without his help it would not have been possible to consider the mountain of data accumulated from test programmes through the years.

In the computing section my thanks are due to Dr. Riccardo Foschi for allowing me to use his program and dispelling my doubts about its likely success and to TRADA for allowing me to duplicate their copy of the program. However, having the program would have been worthless on its own. Dr. Mick Creed of University College Cork and his wife, Norma, have really showed the meaning of true friendship in getting me through the computing barrier. The nine days spent in Cork will not be forgotten, nor the help and encouragement given before and since. In the University of Surrey many people have kindly answered my continual pestering, but Mike Gunn has had to bear the brunt of it and I am very grateful for all their help.

This thesis has had nearly as many typists as supervisors, all have been unstinting in their help. Many thanks to Jane Scutt, Margaret Davies, Shelley Butler and Paula Harris. I am also very grateful for all the drawing work carried out by Graham Barnard and Steve Cobb. A special vote of thanks is due to Anne for her help in correcting the manuscript, a long and very tedious job, and for her support and encouragement.

Finally I owe an enormous debt of gratitude to my Mother, who has been the scapegoat for the work, suffering my disappointments without being able to share the moments of success and happiness. Both she and Anne have been very tolerant and have taken many of life's pressures from my shoulders during the final months of writing.
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CORRIGENDA

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bitumen impregnated insulating board.

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 NB. the overlay has been replaced by separate graphs.

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CHAPTER 1

INTRODUCTION

1.1 TIMBER FRAME CONSTRUCTION

The modern system of timber frame, using small section members, was developed in America from the traditional construction methods taken over by 17th century British settlers. Its evolution was made possible in the 19th century by the introduction of both mechanised sawing and wire nails. The former system allowed small sections to be cut accurately and with ease and the latter eliminated the need for carpentry joints. The new form of construction developed rapidly to meet the massive need for housing created by immigration and soon spread throughout the world to areas where there were major supplies of softwood. Later development in other areas, such as Britain, owes much to its suitability for prefabrication. Today there are more timber frame dwellings built in the world each year than houses of any other form of construction (TRADA, 1985).

The rapid development of timber frame in Britain began in the 1960's when the possibility of finishing the timber shell to give a traditional appearance, was noted. Previously timber frame had been all wood with external walls finished in cedar shiplap, etc., which made it of interest to a very small section of the housing market. The advantages of timber frame were seen to be:

(i) its suitability for large developments,
(ii) factory prefabrication,
(iii) reduction in site work,
(iv) reduction in delays due to weather,
(v) greater flexibility in programming finishing work.

They commended it to the larger developers and civil engineering firms who were becoming active in the domestic housing sector.
The essential part of the system is the timber frame wall which is responsible for transferring vertical loads in the structure to the foundations and resisting any horizontal loading which, in Britain, results totally from wind effects. A standard truss roof system is usually incorporated with either a suspended timber or concrete slab groundfloor, all in common with traditional brick houses; subsequent floors are normally of timber construction. The external cladding of the building allows a variety of treatments, many of which may be combined, including:

(i) timber boarding - traditional to timber frame,
(ii) render - not common due to differential movements,
(iii) tile hanging - popular for upper floors,
(iv) brick slips - a quick method of achieving a traditional British appearance,
(v) brick skin - the most common form of construction.

In the last case, the timber frame panels replace the inner leaf of the double skin brick wall and, until recently, were considered to be the only load bearing elements.

Of the different framing methods available today, platform frame is the most common and is almost exclusively used in Britain. It consists of storey height stud walls which, where necessary, support floor and roof elements. All these parts act together to form the structural shell of the building. The unique part of the system, the wall, consists of small section timber verticals, termed studs, spaced at regular intervals to support directly floor joists and roof trusses. The studs are made into a frame using top and bottom rails of the same timber section. The external wall frames are sheathed with a wood based board to provide shear stiffness and are lined with plasterboard. A vapour check is provided behind the lining to prevent water vapour generated in the house from permeating the wall. Insulation is laid between the studs giving a U value typically better than 0.40W/m². Finally, a breather paper may be necessary on the external walls to prevent it from getting wet during construction. A typical wall is shown in Figure 1.1.
Internal walls, using a similar form of construction, include plasterboard partitions and separating walls.

The frames may be prefabricated or site assembled, but in Britain, the former method is nearly always adopted. They are erected as units, by hand or by crane, depending on size, allowing the structural shell of the building to be completed very quickly, leaving the house free for finishing by all trades simultaneously. The erection of the platform frame system is demonstrated in Figure 1.2.

1.2 THE FORCES ON A TIMBER FRAMED BUILDING

The forces on the timber framed building covered by this investigation are limited to the vertical self weights and imposed loads, and the horizontal wind loads. The vertical loads are carried through joists or trusses on to the load bearing walls and once the distribution of the loads had been determined, a standard method of a design (BS 5268 Part 2), may be applied to check the adequacy of the stud section. The wind load has a number of effects on the building. Its direct action is to cause pressure on one or more faces and suction on the others. The horizontal load is distributed into the upper and lower horizontal diaphragms by the studs acting as beams; the roof, the floors and the foundations all act as diaphragms, depending on which wall of the house is being considered. The bending resistance of the studs can be calculated and the size of stud can be checked by the interaction formula (BS 5268 Part 2). The diaphragms, in turn, are supported by the side walls, which transfer the horizontal loads to the foundations by their in-plane shear action. The shear resistance of the wall to the horizontal force is known as the racking resistance and is the subject of the current investigation. The racking resistance covers the initial stiffness of the wall and its factor of safety against failure. It is considered independently of the bending and axial design of the wall and at present is not covered by standard design procedures although, as will be described later, a checking
procedure will soon be available in Britain (BS 5268 Part 6, draft to be published). The importance of the vertical load to the racking resistance of the wall will be made clear in this report and, as shown in Figure 1.3, the racking resistance is dependent on the general construction of the building as well as its size, shape and orientation to the prevailing winds.

1.3 THE NEED FOR A DESIGN METHOD FOR TIMBER FRAME WALLS

It has been noted that vertical and face loads can be checked by standard design approaches, but that no set method is available for checking the racking resistance of the wall. A detailed method redressing this omission would present a complex problem and would need to cover:

(i) the materials making up the wall; the frame material, the sheathing and the fixings,
(ii) the configuration of the wall; its length, height and the size and location of openings,
(iii) the applied loads, both horizontal and vertical.

A strong argument can be advanced stating that there is no need for a design method for the majority of houses constructed. Traditional brick houses in Britain are checked by their visual appearance and only in the cases of narrow walls, with many large or awkwardly positioned openings, would checks be required. This is a result of long-term experience of the performance of brick buildings which records very few instances of wall failure. However, where problems have occurred, they indicate the types of detail that must be avoided. A similar approach is adopted for timber frame construction in countries where it is traditional, i.e. the U.S.A., Canada and Scandinavia, and there is no design requirement for the walls. In areas where there are special problems, for instance in New Zealand with earthquakes and Australia with cyclones, or where special construction techniques are utilised, such as minimum use of structural sheathing, wall designs are necessary, but are often of a specialist nature directly related to the initial problem.
In Britain there is a requirement to prove the structural adequacy of a timber frame wall which is not a result of high applied loads or partial structural cladding. Regulations are considered necessary because the method of construction is new to the country. To be more contentious, it may be argued that this is a barrier erected by authorities representing the monopolistic traditional brick dwellings industry. Certainly the argument is given some credence by the fact that so little note has been taken of the vast experience of timber frame housing available in the Western world and by the actions of the "Campaign For Traditional Homes" in the early 1980's. However, such arguments become emotive and it is much better to accept that all timber frame houses should be structurally checked for racking resistance, along with all other structural actions. Advantage may then be taken in promoting them as fully engineered structures.

Once this approach has been accepted, it is necessary to check the design methods in use and identify the changes that may be necessary to provide a structurally comprehensive solution. During the 1970's and the early 1980's, the standard method of calculating the shear load capacity of the wall was to take a shear stiffness per metre run for the materials used in its construction, which was multiplied by the length of full height panel. The shear stiffness values were often very high, although when used with this very simple design approach, they achieved safe design figures, as there has been no evidence of wall failure in timber frame housing. The values were based on very early American test figures and cannot now be justified. The need for a Code of Practice on timber frame walls presenting a more rational approach to shear wall design was, therefore, both urgent and strongly felt. The requirements set down for the Code of Practice were:

(i) to provide national standards for the construction of walls in terms of quality, workmanship, etc.,

(ii) to advise on grey areas in design, such as the contribution of normally non-structural materials, like plasterboard,
(iii) to define minimum stiffnesses for the standard wall materials,

(iv) to detail a test method that could be used to derive the performance values in (iii) above, and could also be used to check the behaviour of typical walls,

(v) to outline a simple design method that could be used to prove the adequacy of walls based on the material characteristics found in either (iii) or (iv) above.

The draft for comment of this Code of Practice (BS 5268 Part 6) is about to be published. Due to the very close interaction between the code inclusions, on design and the test method, and this investigation, it is necessary to explain the background to this research project and the liaison with the British Standards Subcommittee preparing the draft Code of Practice.

1.4 THE BACKGROUND TO THE INVESTIGATION

The period of study commenced in 1976, when Professor Stafford-Smith set up a research fellowship at Surrey in conjunction with Guildway Ltd., manufacturers of timber frame houses. The aims of the project were to set up a test frame and provide design values for Guildway manufactured panels from which a more general design approach could be evolved. Stafford-Smith was co-opted onto the drafting committee of the Code of Practice, which was then beginning to consider in detail the testing and design of wall panels. Shortly afterwards, Stafford-Smith left the University and this saw the start of the investigation in its present form. Links with the code committee were strengthened when, due to Government cut-backs at the Building Research Establishment's Princes Risborough Laboratories, the test frame became almost solely responsible for racking tests carried out to the preferred test method. The investigation contributed much to the development of the test method and provided results which led to early wall
design proposals (Robertson and Griffiths, 1981). However, differences within the committee led to a quiescent period in its work, during which time the test programme for the study continued. Financial links with Guildway had terminated in 1978, but contract testing of sheathings on standard panels not only funded research work, but added to the information covering the materials parameters. Separate research programmes investigated other aspects of wall design, such as length, openings, method of base fixings, etc.

The code drafting committee was reconstituted in 1983 as a result of the bad publicity given to timber frame houses (see Chapter 2), and began to rely more heavily on the work of this current investigation. Early findings were directly used in three areas:

(i) to select the method of test to be adopted by the code,
(ii) to detail the test method,
(iii) to formulate the design method.

Parts of the contents of the draft code (BS 5268 Part 6, to be published) are very much a consequence of this study and, as such, have been included in Appendix A. However, they represent only an intermediate stage in the general aims and the potential of this study for the following reasons:

(i) the scope of the design was limited by the requirements of the code,
(ii) the limited timescale allowed for production of the code, which resulted in item (iii),
(iii) the restriction to the use of early test results only.

1.5 THE AIMS OF THE INVESTIGATION

The major aim of the project since its inception in 1976 has been to provide a comprehensive design approach for timber frame walls. It has been assumed throughout that the design method would have to be based on full-scale
testing and thus an early objective was to achieve a standard test method. In consequence, the aims of the project have been very similar to the requirements for the Code of Practice noted in 1.3. However, the scope has widened, particularly since 1984, when firsthand information on the treatment of the design and testing of timber frame walls in many other countries became available. Additionally, the opportunity to adapt a finite element diaphragm analysis to model the wall racking test was taken up to check the results of the test programme and to extend their scope. Consequently, the main aims of the current research are to investigate:

(i) the methods of construction, the design regulations and the methods of test applicable to timber frame walls in other parts of the world and to relate them to British experience and requirements,

(ii) and to interpret the standard method of test adopted for use in Britain, noting the contribution of the research programme to its contents,

(iii) and to develop an empirical design method for use by structural engineers to cover all significant variables in wall construction,

(iv) and to prove the Code of Practice design method in the light of the current analysis,

(v) and to derive a less rigorous design method for more general use whereby the adequacy of the many houses which present no special structural design problems can be rapidly checked, thus reducing the differentials in requirements between traditional brick and timber frame wall designs,

(vi) the possible use of a computer based finite element analysis for timber frame walls and check that, if the material parameters are applied correctly, an analytical model can be developed to predict wall behaviour as measured in full-scale wall tests.
Figure 1.1 Section Through A Timber Frame External Wall (after TRADA, 1985)

Figure 1.2 Platform Frame Construction (after TRADA, 1985)
The Effect of Wind Direction and House Design on Shear Wall Performance
2.1 HISTORY OF TIMBER FRAME

2.1.1 Traditional Timber Frame Construction in Britain

Timber frame houses built in the 13th century still exist in Britain and there is evidence that when they were constructed the tradition was already well developed (Mason, undated). Timber frame was a popular form of construction, for dwellings from cottages to large manor houses, throughout the Middle Ages and into the early 18th century. There were two schools of construction using either the box frame or the cruck frame; although some carpenters were skilled in both techniques. Several ideas (Mason, undated: Harris, 1978) have been put forward for the regional split which locates cruck frames in the North West and box frames in the South East with a changeover belt covering Devon and Cornwall, Wessex and the Midlands. One report suggests the former to be a highland construction and the latter more suited to lower lands, whilst another takes cruck frame to be of Celtic origin and box frame to be a Norman introduction. Certainly the later the construction, the more likely it was to be a box frame.

Many of the larger buildings would have been architect designed, but the standard timber frame construction was vernacular, i.e. built by local craftsmen using local material. Thus the occurrence of timber frame is patchy, depending on a supply of timber or lack of stone, with the infill materials even more varied in their form.

The traditional timber frame house was built in bays, typically around 5.0m wide following the classical theory that the provision of heavy structural piers at proper intervals reduces the size of the intermediate structure for floors, walls and roofs and leads to an economic structure. An example of bays applied to the box frame is shown in Figure 2.1. The main differences between box and cruck
frames is in the design of the cross frame. In the former case, it was constructed with vertical posts, horizontal ties and roof trusses, braced and supported as necessary, depending on the required openings and span of floors; the main posts also supported the wall plates which joined the cross frames at eaves level and supported the rafters and infill wall frames. The cruck frame was formed from long curved timbers framed together in pairs, joined by tie beams or collars and rising from ground level to support roof purlins. Intermediate styles for wider buildings and larger spans included many specialist frames, such as aisled, hammer beam and base cruck. The main differences between box and cruck frames are shown in Figure 2.2 and are described by Mason (undated), Harris (1978), Mercer (1975) and Brunskill (1978).

The secondary framing reduced the openings in the gable wall and between cross frames into small panels which could be infilled to complete the walls (Figure 2.3). The method of infilling was extremely varied and dependent on the local materials and the importance of the building. The authors previously noted cover the infill walls, and in addition Clifton Taylor (1972) details the building materials.

Most common, in the early days, was one of the many forms of mud daub on a wattle trellis base. These finishes were gradually replaced by plastered laths, plastered stones, brickwork and even tile hanging. In more important buildings decorative framing was often incorporated.

The design of these traditional buildings differed from those used today in many respects, notably the size of the timbers and the structural actions of the walls. The frames, probably, were totally load bearing, taking out the horizontal wind load through the action of the frame braces or even in using the bulk of the members and the strength provided by the carpentry joints. However, the infill panels would also have acted as very stiff shear units and
it would be interesting to measure the overall racking performance of one of these walls in comparison with the modern form of construction.

2.1.2 Modern Methods of Timber Frame Construction

The design of timber frame walls has changed considerably since the 18th century when they were a common method of construction in Britain. As outlined in Chapter 1, the present method is based on the principle of using small section timbers at close regular spacings and has been developed from use in North America.

TRADA (undated), noted four methods of timber framing. They are described below:

(i) Stud frame construction, where a series of small section vertical members, known as studs, are spaced at fixed centres from 400 to 600mm apart. They are framed with horizontal head and sole plates to form structural wall panels, which can be modified to accept window and door openings. Floors and roofs span onto the load bearing walls and sheathing boards, nailed to the frame, provide the stiff diaphragm action needed to resist horizontal forces.

(ii) Post and beam construction, a development of the box frame houses, where a structural framework of widely spaced beams are supported on individual columns. Floor and roof elements span between these beams and non-load bearing infill panels form wall or window units between the columns.

(iii) Portal frame construction; the modern equivalent of the cruck frame, where rigid portal frames in solid, laminated or plywood construction support joisted floor or roof elements with non-load bearing stud frame construction infill wall panels.

(iv) Membrane construction, where the structural elements of the roofs, floor and walls take the
form of stressed skin panels of composite construction using outer skins of plywood on lightweight timber framing.

The stud frame construction is the basis for the majority of timber frame housing in North America, Scandinavia, Australia, New Zealand and Great Britain. It is the only form of construction covered by this investigation, although some of the test results may prove to be applicable to membrane structures and could certainly be used in calculating the shear resistance of the non-load bearing infill panels.

Stud frame construction follows different forms which, in general, may be classified by their use in either prefabricated or on site construction, although it should be noted that most prefabricated methods will also be suited to site construction. The historical method for on site domestic construction is balloon frame as detailed in Figure 2.4. In this system, stud members are continuous over two storeys and floor joists are fixed to the sides of studs. The method requires relatively long members of small section for studs and is not suited to prefabrication in Great Britain. The standard method adopted by the prefabricated building industry is that of platform frame, as shown in Figure 2.5. Here, wall panels are of single storey height and are erected upon platforms formed by the ground and first floor construction. Both wall and floor units are suited to factory prefabrication, being small enough in size to facilitate transportation and to allow very rapid site erection. Two other methods noted in the TRADA design guide, mainly suited to on site construction, are, firstly the modified frame method, where the wall frames are of single storey height with the joists fixed to the sides of the studs as in balloon framing, and secondly the independent frame method, where the wall structure is separate from the first floor element, although the floor support may be fixed to the face of the wall. Neither method is common in Great Britain and, together with the balloon frame method, have been eliminated from the scope of this thesis.
2.1.3 The Development of Platform Frame Construction in Britain

Platform frame construction has probably been used in Britain throughout this century, but to a very limited extent and as a progression from barn construction. In the early 1960's, there were a number of specialist manufacturers building timber frame houses clad with shiplap boarding and thus of interest to only a very small fraction of house buyers. Most of these houses were "stick built", i.e. constructed on site, but the potential of platform frame for factory fabrication was soon appreciated. Later in the sixties, one of the prefabricators went to America and saw brick and blockwork being used in conjunction with timber frame. He returned with the idea of building timber frame houses of traditional appearance, replacing the inner brick leaf wall with factory built timber frame units. The idea caught on, first for private dwellings where manufacturers were able to offer increased specification, and later in the public sector, where the advantages of prefabrication could be fully utilised. Timber frame then became popular with developers and the civil engineering contractors, who were entering the mass housing market. Their influx of capital allowed factory manufacturing processes to develop rapidly. Computer aided detailing of panels linked to a computer controlled sawmill, semi automatic computer controlled frame tables and automatic gun nailing of sheathings all became common, allowing the industry to meet the steadily increasing demand. It can clearly be seen now that this was the start of the problems for the timber frame industry as quantity became more important than quality, particularly on site, where many houses were erected by sub-contract gangs with little experience, and less understanding, of timber frame and no corporate identity with either the manufacturer or the product.

At the peak of its popularity, in the middle of 1983, timber frame construction accounted for 25% of the domestic market in Great Britain, which included 50% of the houses
built in Scotland. In Northern Ireland, the share was 35%. The brick and blockwork industry retaliated to these incursions with a campaign through press and television damning timber frame construction. The World in Action programme in 1983 severely criticised the industry. It showed evidence of poor standards of workmanship on site, hinted at constructional problems and cast doubt on the long term durability and safety in fire of timber frame construction. The comments on workmanship were quite justified and could have been used to advantage, by the industry, to get rid of bad practice. The hints and doubts were largely iniquitous, their evidence was based on very specialised cases, which, in many instances, could be explained as the sort of teething problems likely to occur with the introduction of any new system. A number of cases cited concerned construction details which had already been eliminated and one case of rot in timber referred to a flat roof problem that was independent of the form of wall construction. No effort was made in the programme to allay the worries of the public by demonstrating that the same timber frame construction system had been used safely, economically and without major problems in many other areas of the world, with similar environmental conditions to those in Britain.

Very little response was made by the timber frame industry publicly to what it considered to be a malicious attack by a brick and block industry worried by the inroads made by timber frame in the domestic housing market, and thus not worthy of reply. Since 1983, there have been further criticisms in the press. These have sometimes been due to badly presented data, including one case where information on faults in newly constructed timber frame homes was leaked in advance of a further report which also included a similar percentage of problems in traditional homes. So dramatic was the effect of the bad publicity for timber frame construction, without any strong positive response being made, that the market share for the system by 1985 had fallen to 7% in Great Britain, which included 30% in
Scotland. In Northern Ireland, the construction had dropped below 50% of the 1983 output.

To improve its share of the market in the future, the timber frame industry will have to stand on quality, hence the introduction of the NBA/TRADA site checklist, and will need to educate the public concerning the emotive subjects of rot and fire in timber. This will not be easy with incidents like the Bradford Football Stand and Hampton Court fires, but the success of timber frame abroad must be used and must be shown to a wider audience than building society surveyors and local authority engineers. Their acceptance of timber frame is essential, but it is the house buying public who will decide its market level.

Many books and journals of building and architecture which cover the structural use of timber have given a sketchy introduction to timber frame construction in Britain. TRADA (1985) is probably the best and most up-to-date. In Britain, TRADA has been the single dominant body backing timber frame construction and has acted as a united voice for the many fragmented parts of the timber frame industry. Its Timber Frame Housing Design Guide (TRADA, undated), was, in the 1970's, the only source of information relating directly to British requirements outlining: building regulations, methods of construction, production and erection, specification and typical details. One section covered, non-controversially, loading on the wall structure and standard design methods for vertical load; but other parts, concerning design and testing, were questionable, being based on American practice and will be discussed later in this Chapter.

TRADA and the NBA (TRADA, 1980a) updated and simplified this manual in terms of the building details, the specification, on site information and the applicable regulations. The book was a great improvement and best illustrates construction techniques used today in Britain. Unfortunately, the main text refers only to plywood and
standard U.K. timber sizes. Canadian Lumber Standards (CLS) sized timber is nowadays more common for framing and both medium board and bitumen impregnated insulating board are widely used for sheathing.

A sister volume (TRADA, 1980b) covered structural recommendations for the main building elements: walls, floors, and roofs, but in terms of racking resistance, showed no difference from the original guide (TRADA, undated).

Later, booklets and papers (BBA, 1983 and BS 5268 Part 6, to be published) are more concerned with British regulatory practice and will be covered in Section 2.3.

2.2 TIMBER FRAME CONSTRUCTION DETAILS

2.2.1 General

This section on construction covers only details unique to timber frame and, in particular, the platform frame system. The scope is therefore limited to the foundation details, concentrating on the joint with the wall panel, and the wall details which cover panel construction and also fixings between panels and fixings to external linings, etc. Differences in floor and roof systems to accommodate timber frame are so minimal that they do not warrant special attention in this investigation.

The construction details are given first for the standard methods used in Britain. The normal practice in other parts of the world are then included for comparison. No attempt has been made to include specialist details incorporated by individual builders and manufacturers. Thus the sizes and spacings quoted often represent minimum standards which may be improved upon in practice.

2.2.2 Foundations And Base Fixings

Timber frame wall construction can be used with either solid concrete or suspended timber ground floors. The choice of floor system will affect the design of the
foundation and the method of fixing the base of the timber frame wall (TRADA, 1980a).

In the former system, a timber base plate usually carries the wall frame. The plate is laid on a damp-proof course (dpc) and a thin mortar bed and is fixed with shot fired or masonry nails, at 300mm centres, which penetrate at least 25mm into the slab. An alternative indirect fixing is shown in Figure 2.6, where a steel channel section is shot fired to the foundation and carries the base plate which is fixed by horizontal nailing. An extension of the external flange of the channel allows the bottom rail of the panel to be similarly attached. This improves on the sole use of vertical nails to join the bottom rail to the base plate, since uplift forces will be resisted by nails in shear as well as nails in withdrawal. This method of fixing is of particular use where panels are lined on both sides prior to erection. Size and spacing of the channels is not standardised but, typically, sections would be 300mm long allowing four nails per flange per timber section, and would be positioned near the ends of walls and at intervals not exceeding 2.0m.

If the edge of the slab is brick faced, a different method, as shown in Figure 2.7, is adopted where galvanised steel straps embedded in the concrete at 1200mm centres are bent over and nailed to the plates.

In suspended timber floors, as shown in Figure 2.8, the timber base plate is fastened to the brick wall on top of the dpc using 25mm x 16 gauge galvanised mild steel straps at 1200mm centres taken down at least four courses of brickwork and twice nailed with 50mm nails to the plate. The floor assembly is then fixed to the bottom rail using either a double joist system in the line of floor span or a header joist and full depth blocking, as shown. Both the floor decking and the wall units rest on the joists with the wall brought up to the height of the floor using a separate packer section. As with the solid ground floor system, all joints
between parallel timber sections are made with 75mm long 3.75mm diameter nails at 300mm centres.

The Canadians (CMHC, 1979) use very similar methods for fixing panels to the foundations. Usually a suspended floor system is used, combined with a reinforced concrete foundation. The base plate (or cill plate) is commonly fixed to the foundation using anchor bolts, as shown in Figure 2.9. The bolts should be 12.7mm diameter at not more than 2400mm centres, with at least two bolts per cill unit. They should be embedded at least 100mm in the foundation wall, deformed to prevent withdrawal, and a large flat washer should be used between the nut and the cill. The nailing specification for subsequent timbers is 82mm long, 3.75mm diameter nails at 400mm centres. One other difference is that the floor boards are seen to be carried underneath the timber frame. This detail indicates that the floor structure is completed before the wall unit is erected and may mean that it will be exposed to damp during construction. The system, common throughout the world, is becoming more popular in Britain, but is usually incorporated only if the flooring material is moisture resistant.

A similar foundation system is recommended by the U.S. Department of Agriculture - Forest Service (Anderson, 1975). They give an additional recommendation for high wind areas, as shown in Figure 2.10, where a steel strap is effectively fixed to the foundations by the cill plate and is attached to the stud by three 63mm long, 3.0mm diameter nails. An Australian system for hurricane wind loads carries the method of holding down even further, taking 12mm tie-down rods through the wall structure to a fixing on top of the panel head binder. If a second floor is required, a further tie-down rod acts between the top rail of the first floor panel and the head binder of the second floor panel. The roof is directly fixed to the head binder by anchor plates, ensuring that its normally lightweight construction is positively restrained to the foundations and is unlikely to uplift in high winds.
In New Zealand (SANZ, 1984), the wall plates are either fixed to concrete foundation walls, or sub floor jack studs, which are not used in Great Britain. In the former case the plate should be held down with M10 bolts penetrating 75mm into the concrete, or by R10 dowels bent at least 90°, set not less than 75mm into the concrete and projecting sufficiently to allow for not less than a 75mm length of dowel to be clinched over the timber. The fixings should be at 1.4m maximum centres and not more than 300mm from the corners of foundation walls.

Many of the details, common overseas, are incorporated into British practice and are worthy of consideration in the design of test panels.

2.2.3 Walls and Panel Fixings

A standard method of construction applies to all wall panels in Great Britain whether they are used for:

(i) external walls,
(ii) separating walls formed of double frames,
(iii) load bearing internal partitions,
(iv) internal partitions resisting only horizontal loads, or
(v) lightweight non-load bearing partitions.

A common section of timber is used for the wall frame and for all additional members, e.g. head binders, noggins, etc. Typical section sizes are 97 x 44mm and CLS 89 x 38mm. Any species of wood recommended in BS 5268 Part 2, can be used, providing that it can be proven adequate in resisting bending, due to wind applied to the face of the panel, and axial compression, due to vertical loading, and it is sufficiently durable for its condition of use. In practice, the most common timbers are North American Spruce/Pine/Fir (SPF) and European Redwood/Whitewood. General structural (GS) grade wood is acceptable for the majority of uses, where the applied stresses are low.
Frames without openings are made up with top and bottom rails and are recommended (TRADA, 1980a) to have studs at 600mm centres which should line up with joists or trussed rafters. In practice, 400mm centres are also used but they have not been covered by this investigation and may be ignored in the knowledge that such walls should behave at least as well as one with studs at 600mm centres if the total vertical load is the same.

TRADA (1980a) states that frames should neither exceed 3.6m, for ease of handling, nor be less than 1.8m long in outside walls. However, more modern techniques, using cranes to install panels, means that wall lengths are limited only by transportation and storage restrictions; thus 7.2m lengths are common. In Scandinavia, a 1.2m long module system is often used for external walls, increasing the size only to allow for special openings such as patio doors. The panels are fully sheathed, lined, insulated and glazed and thus the module length represents a restriction in weight when the panel has to be carried by two men. Whatever form of construction is used, panel length will be in multiples of 600mm and stud centres will be adjusted accordingly. The standard panel height is 2.4m, although 2.7m is becoming increasingly common for non domestic use and 2.1m high panels are used in Scandinavia for holiday chalets. All further work refers to 2.4m panels unless specifically stated otherwise.

Openings can be of any width, to a normal maximum of 2.1m, and of any height, limited by the top of the opening being 2.1m above floor level. They should not normally be positioned within 300mm of either the ends of walls or other openings, although design rules can easily be applied to cover infringements of these guidelines. Small openings can be accommodated between studs and are framed out using standard section timber. Larger openings require lintols which are carried on cripple studs which should each be fixed to a full height stud. Lintols for standard height openings may be either full height between the top of the cripple stud (2.1m) and the underside of the bottom rail, or may
be to a designed height and framed out with short lengths of timber at the standard stud positions. Below the opening the panel is framed with a horizontal rail and short studs at standard centres. Cripple studs to support the horizontal rail are unusual as end nailing is normally sufficient to transfer window loads. Figure 2.11 shows the differences between short and long openings and advises on positioning of windows for economy in frame timber.

Sheathing and bracing requirements for panels will depend on their use. For external panels, the latest guidelines (BS 5268 Part 6, to be published) include the following board types:

(i) Plywood: any grade or species covered in BS 5268 Part 2, having a minimum thickness of 5.5mm,

(ii) Fibre building boards of the following types:
   (a) Sarking and sheathing grade bitumen impregnated insulation board in accordance with BS 1142 Part 3, and not less than 12.3mm thick,
   (b) Type HM medium board in accordance with BS 1142 Part 2, and not less than 5.9mm thick,
   (c) Type TE tempered hardboard in accordance with BS 1142 Part 2 and not less than 5.9mm thick.

(iii) Wood Chipboard: Type III in accordance with BS 5669, and not less than 11.6mm thick,

(iv) Gypsum Plasterboard in accordance with BS 1230, and not less than 12.5mm thick, although here the board is allowed to make only a 'contribution' and its use as a structural sheathing is limited by a number of other clauses.

Restrictions are placed on the use of these boards; plasterboard should be used for internal linings only and chipboard should not be considered if the timber frame wall is to be the external wall of the building.

TRADA (1980a) recommends that all boards should be to the full height of the panel and should be used in 1.2m
widths except where cutting around openings or at one end of a panel if the length is not a multiple of 1.2m. In practice, methods of sheathing around window openings vary (Figures 6.39 and 6.41); as the variations affect panel performance. They will be considered in detail in Chapter 6.

Separating walls form a special construction. They are lined with a minimum 27mm of plasterboard consisting first of 18mm horizontally laid planks followed by the normal 12.5mm lining board. In terraces, it is possible that a wind load on the building face would have to be resisted by separating walls and partitions only. For this reason, it is standard practice to provide secondary bracing on the external face of the panel in the cavity between the walls of the separate dwellings. This takes the form of either 100 x 25mm timber diagonals (Figure 2.12), or single sheets of the structural sheathing. As acceptance grows, for fewer limitations on the structural use of plasterboard, secondary bracing may not be required.

Fixings are required to be either steel fasteners coated by hot dip galvanising, sherardizing or other suitable treatment against corrosion, or manufactured from stainless steel (BS 5268 Part 6, to be published). TRADA (1980a) recommends two 100mm long by 4.0mm diameter nails in all butt joints in frame members or, alternatively, three 75mm long, 3.75mm diameter nails skew driven. Parallel timbers should be nailed with 75mm long nails at 300mm centres. Sheathings normally require 50mm long, 3.0mm diameter nails at 150mm centres in dense sheathings. These nail sizes and spacings are guidelines only; where they affect performance, further details are given in BS 5268 Part 6 (to be published). Their effect is discussed in detail later in the thesis. Nail lengths depend on penetration in the frame timber: for wood based sheathings, the rules quoted in BS 5268 Part 2 should be applied and for plasterboard, British Gypsum guidelines should be consulted.

Manufacturers have varying methods of assembling wall panels. In general, external corners are built up with two
extra frame timbers and internal partitions are joined to an additional stud fitted into the main wall (Figure 2.13). Vertical joints between panels are made with 75mm long nails at 300mm centres and in most cases are reinforced by the use of a standard section head binder, nailed to the top rail and continuous for 600mm over the joint. A suspended floor system can be carried by the wall at first floor level with subsequent walls fixed to the floor units, as shown in Figure 2.8. A complete wall assembly will be similar to that shown in Figure 2.15.

The most common system for the manufacture of timber frame walls in Great Britain is for the panels to be framed and externally sheathed in the factory. They are palletised and delivered to site and erected either by crane, on large sites, or by manhandling, on smaller contracts. Temporary bracing is used where necessary until the shell of the building has been completed. Once the structure is weatherproof, the insulation can be inserted between studs and the vapour barrier and internal lining, normally gypsum plasterboard, attached. The most widely used external finish is the single skin brick wall which is linked to the timber frame by special brick ties, at 600mm centres along studs and at 400mm maximum centres vertically. A 50mm cavity is maintained between the two walls which should be ventilated and broken up by fire stops, as necessary. Other external finishes, such as vertical tile hanging and vertical or horizontal boarding, can either replace or be used in conjunction with the brickwork. Exterior finishes to the timber frame wall are rarely considered as contributory to structural performance, except the brick skin, which will be discussed in Chapters 5 and 6.

Standard Canadian and American practice for platform frame construction is very similar to that used in Britain, described above (Anderson, 1975 and CMHC, 1979). It is more common for the walls to be manufactured on site when they are framed in a horizontal position on the sub floor and are sheathed prior to erection to avoid the need for scaffolding. A typical wall frame system is shown in
Stud sizes in Canada are often greater than in Britain due to the cheapness of low quality timber and the need for greater insulation. However, the minimum size of 89 x 38mm is standard in both Canada and the U.S.A. Sheathing materials are commonly plywood or flakeboard, with oriented strand board now entering the market. In the U.S.A. some bitumen insulating board is also used. In both countries there are fewer regulations, which results in the complete wall being considered structural so that, in effect, the plasterboard lining and any external facings will be contributory (Wolfe, 1982). Fixings and standard details are similar to U.K. practice. In Canada it is common to fix plasterboard sheets horizontally rather than vertically, to reduce the amount of nailing; blocking is not required under this joint. There is also a trend to use the external sheathing boards in the same way with or without battens and this has led to a series of tests being included in the investigation, which is reported in Chapter 6.

In Australia and New Zealand, a greater percentage of construction is carried out on site and although the basic method represents platform frame, there are differences and the term is not used in standard design guides (SANZ, 1984). In both countries, the sheathing material is placed only where it is needed based on design calculations. Large areas of external wall are left unsheathed, clad only in building paper. The reasons for this reduced requirement for structural sheathing is that the regulations covering plasterboard are less demanding. In Australia, plasterboard is considered structural and is capable of high load resistance, particularly if the wall is clad on both sides as will be the case in internal partitions. In New Zealand, plasterboard can be used either in conjunction with a timber sheet material, or with diagonal braces, made either of timber or thin gauge steel. It cannot be used alone.

In both countries, panel fastenings are different. In New Zealand, the structurally sheathed panels have to be
fixed to the floor joists or boundary joists, either by straps, at each end of the whole sheet braced element and each capable of carrying 6kN in both tension and compression, or by overlapping the board and nailing it with six very closely spaced nails at each end. In Australia, the bottom rail of the panel is fastened direct to the joists with either steel straps, carried under the joists, or special proprietary plate fastenors or M10 bolts at 1.2m centres.

The method of only partially sheathing the external walls is becoming popular in the U.S.A. Further advantage is gained by cladding the unsheathed frame with a high performance insulation board such as Styrofoam.

Scandinavian systems are very similar in detail to those described for Britain. One variation in base fixing is that the panels may be laid onto a rubber sealer strip nailed to the cill plate. The walls are more likely to be 145mm deep, but if the cill plate is kept 89mm wide, the extra width may be used as a cable conduit hidden by the skirting board. The panels are often lined with medium-board (MDF), allowing factory fixing and thus protection of the vapour barrier. An external bitumen impregnated insulating board is also factory fixed dispensing with the need for breather paper and allowing the panel to be completed before delivery to site. During erection, panels are vertically located with tongued and grooved joints and structural continuity is achieved by skew-nailing the panels together from the outside.

The differences noted for timber frame wall construction overseas are due to either differences in building regulations or differences in tradition, normally relating to available materials. The structural alternatives, noted above may be summarised in the use of:

(i) partial sheathing of the external face,

(ii) horizontally placed boards,

(iii) flakeboard and oriented strand board sheathings,
(iv) metal strip braces,
(v) wood based sheet materials as internal linings,
(vi) moisture resistant plasterboard as a sheathing board.

Some of these forms of construction already exist in Britain and it is likely that the others will soon be tried. It will be of value, therefore, to consider their effect on wall design.

2.3 REGULATIONS COVERING THE CONSTRUCTION OF TIMBER FRAME HOUSES

2.3.1 General

Regulations for buildings may be expected to cover:

(i) their structural adequacy,
(ii) the quality of materials and construction,
(iii) environmental effects.

The firmness with which controls are applied will depend on previous experience of the form of construction. When platform frame construction began to expand in Britain, local experience was still very limited and of a short-term nature. It is therefore understandable that more stringent forms of control were required than in other parts of the building industry, although it is not clear why so little advantage was taken of overseas knowledge. A new form of construction presents a further problem in that some experience of behaviour is necessary to define the requirements included in the regulations. The limited interest shown abroad for comprehensive rules or guidelines did not help in the easy formulation of British requirements. It has taken approximately fifteen years to establish a form of national control, based on standard guidelines for timber frame construction in the U.K. This section looks first at the development of U.K. regulations and then, in terms of structural consideration only, makes comparison with practice overseas.
2.3.2 Construction Controls in Britain

Structural requirements for timber frame are comprehensive, but concentrate on stability. Floors and roofs can be covered by standard methods which are equally applicable to traditional construction. Vertical loads and horizontal face loads on the walls are also well covered by standard codes (BS 5268 Part 2), leaving only the racking resistance to be considered independently. Consequently, one of the major requirements of a checking authority is proof that the applied wind load can be carried by the shear walls of the building. This has necessitated a design method for the shear resistance of timber frame walls and a method of test which can be used to provide initial data and can also check the adequacy of the design calculations.

Initially, the design approach was simple and conservative, considering only the external timber frame walls of the building but, as knowledge of behaviour has grown, further parts of the overall structure have been investigated in terms of their likely contribution to the racking resistance of the structure. These include partition walls, the external brick skin and the shell effect of the building where the whole is considered greater than the sum of the parts. These changes would significantly add to the complexity of the design method, but will be slow to be introduced due to inertia caused by tradition (e.g. in the use of plasterboard) and the cost of the research necessary to prove the changes.

Intermediate guidelines have therefore been necessary. These have included:

(i) A very simple design method based on total length of full height panels and American test figures (TRADA, 1980a and b).

(ii) A similar design method to (i) above, but incorporating racking resistances based on the British method of test.
(iii) A more comprehensive design method (Robertson and Griffiths, 1981), using board design figures taken from manufacturers' literature and based on the British method of test.

The latest regulatory document for timber frame walls which covers most aspects of their structural behaviour, but concentrates on racking resistance, is the draft Code of Practice, BS 5268 Part 6 (to be published).

Structural considerations for which calculations are not generally required, e.g. fixings between panels and between panels and floors, roofs, foundations, etc., are covered by acceptable standard details as quoted by TRADA (unpublished, and 1980a) and in manufacturers' brochures. In the latter case, the Council of Forest Industries of British Columbia (COFI) give details of framing, relating to North American practice, and board manufacturers base details of sheathing fixings on the test conditions applicable to their test performance figures.

A further structural aspect is integrity, which in this case may be considered as the way in which the elements of the structure act together. Originally very few guidelines were laid down, although it is in this area that significant changes in construction have been made. Problems have been found from practical experience and have then been researched allowing standards for good practice to be written. A typical example is the effect of shrinkage in the timber frame which causes differential movement between the frame and the brickwork. Research by the Building Research Establishment has resulted in two papers, BRE DAS 75 and BRE DAS 76, which suggest that gaps or soft joints should be provided on the basis of a shrinkage of at least 6mm per storey in the timber frame. A recommendation that timber delivered to site be kept as dry as possible at all stages of construction is also included.

Structural integrity problems are closely associated with site control and in this area little change has been
made for timber frame. The standards applied to traditional housing were considered adequate in view of the fact that timber was commonly used in roofs and floors, and that much of the construction was similar. However, the importance of the main differences, such as prefabrication, were overlooked and the problems that have arisen have resulted in very localised regulations being applied which vary throughout the country from being non-existent to being ridiculously stringent. A national set of standards is required, highlighting the problem areas and providing a set of checks for both the builder and the inspector to follow. Work in this field is at present being carried out by the Timber Frame Technical Committee, set up by the House Builders Federation and TRADA. Independently, they are preparing site checklists and buyers’ guides to timber frame houses.

The only regulatory body to publish advice on the requirements for timber frame walls has been the British Board of Agrément in their MOAT No. 26 (BBA, 1983). This covers mainly environmental aspects reviewing quality requirements, means of control and method of determining wall characteristics. The key areas relate to:

(i) insulation,
(ii) fire,
(iii) physiological properties,
(iv) vermin and bacteria,
(v) durability,
(vi) structural performance.

The final section covers racking resistance, together with vertical load, bending strength and impact strength. However, throughout the report the guidelines are very limited and, in the case of racking resistance, only state that the panel type should have been subjected to either the ASTM E72-80 test or the PRL/University of Surrey test.
2.3.3 Wall Design and Test Regulations Abroad

American codes and standards may be classified under two categories, specification and performance. A specification code lists material type, quality, size and spacing to perform a certain function; whereas a performance code states how a building element must perform under a specific loading. Sherwood (1982) claims the Americans have many performance standards, listing:

(i) the Department of Housing and Urban Development Minimum Property Standards (HUD-MPS),

(ii) the Basic Building Code,

(iii) the Southern Standard Building Code,

(iv) the Uniform Building Code,

(v) American National Standards Institute (ANSI).

For more standard problems, such as design of floors or vertical loading in walls, these standards quote the load that should be applied and performance levels and requirements in terms of deflection limits and factors of safety, etc. Wall design for racking resistance is not covered so comprehensively. ANSI, for instance, quote wind pressures for use throughout the U.S.A. and then simply require lateral deflection to be in accordance with accepted engineering practice. The only valuable contribution is from HUD-MPS, which states the minimum acceptable performance standards for a panel, made up of the wall materials, when subjected to the ASTM E72-80 test.

Wall design in America, is really related to specification. Anderson (1975) covers standard practice, describing balloon and platform framing and outlining the major details of construction. Minimum standards for all fixings are quoted and, in a section on wall sheathing, typically acceptable forms of construction are noted, covering boarding, plywood, insulation board, gypsum plasterboard and bracing. No design values are quoted and no indication is given of any variation in requirements for use in different parts of the country, although this type of
approach is included in consideration of moisture content in timber.

Sherwood (1980) comments that "Light frame construction has developed over many years, largely by a trial and error method. Techniques that worked were adopted and often became the basis for comparison for judging new techniques". Certainly this is the case with timber frame walls using the ASTM E72-80 test on standard wall panels. Minimum performance levels were set by HUD-MPS, but there is no indication of performance levels achieved by wall configurations. New materials and forms of construction are accepted after rigorous testing at reputable laboratories, such as the Forest Products Laboratory at Madison, or the American Plywood Association's Laboratory at Tacoma. Acceptable standards for construction, to match normal requirements, are then stated in the test reports (e.g. Adams, 1983; Tissell, 1983; Price and Gromala, 1980 and Wolfe, 1982) which allows the system to be brought into common use.

A similar solution is adopted in Canada. CMHC (1979) details the building requirements, which are very similar to those of the U.S.A., and test data from America or from the Council of Forest Industries of British Columbia (COFI) is accepted.

In both countries, a great deal of information on construction techniques and special uses of materials can be gained from manufacturers' literature. Particularly relevant to the design and layout of walls are brochures published by the APA and COFI. As there are a large number of these booklets and they are constantly being updated, they have not been referenced. One advantage of these brochures is that they are available through the British offices of the APA and COFI and the details contained can normally be considered acceptable to U.K. regulations.

In addition to covering structural performance, Sherwood (1982) details performance requirements in codes.
and standards for environmental effects in light frame
construction, including noise control, moisture control,
energy, solar concepts, heat transfer and air leakage.

In New Zealand, both design information and
constructional details are covered by the Code of Practice
For Light Timber Frame Buildings NZS 3604 (SANZ, 1984). A
complete specification is given for a system very similar to
platform frame. Wall design considers horizontal wind loads
and earthquake loads. Using the tables and equations in the
Code, it is possible to calculate the number of "bracing
units" required by the walls in their principal directions.
A further table then gives the bracing units per metre run
attributable to standard wall forms from which the length
of braced wall required can be calculated. The system is
therefore based on a racking resistance per metre run applied
over the total length of full height wall panel, with special
clauses included to cover very short lengths of such wall.
BRANZ (1982) details a special instance where the sheet
lining material can be combined with steel strip braces and
holding down straps to provide a relatively high resistance
in a short, 800mm long, panel.

The Code of Practice refers to a test method (Cooney
and Collins, 1979, revised 1982) which enables the number
of bracing units provided by a particular form of wall
construction to be measured.

No evidence has been found of Australian regulations
for timber frame houses, however brochures from both the
plywood industry (PAA, 1982) and the plasterboard industry
(CSR, 1982) indicate the importance put on the numerical
assessment of racking resistance of walls, particularly in
areas of very high wind loading. The brochures outline
calculations for applied wind loads on a building and then
show different forms of panel construction that may be
considered acceptable quoting racking resistances in terms
of kN/m run of full height panel. They conclude by showing,
for a typical house, the areas where structural bracing will
be necessary; little differentiation is made between internal
and external walls.
The racking resistances are referenced to tests at two different centres and include the work of Frodin and Ross (1975a, b and c). It is clear that at that time there was no specific racking test method used in Australia and thus the panel tests were based on the general test method outlined in their timber code which is similar to that used in Britain and detailed in BS 5268 Part 2.

Plywood values ranged from 2.4 to 4.0 kN/m, depending on the nail spacing and holding down method adopted. Plasterboard values covered both singly lined and lined both sides forms of construction and lay between 2.5 and 4.5 kN/m.

No evidence of standard requirements for either wall construction or wall behaviour was found in Scandinavia. This may have been due to language barriers, but is more likely to be correct, based on the traditional nature of construction and a very high record of acceptable performance. A representative of a Scandinavian house manufacturer in Britain stated that his houses required no special certification in Sweden, but in Britain this had to be provided by a consulting engineer and would in future be related to BS 5268 Part 6 (BSI, to be published) in terms of wall performance.

2.4 TIMBER FRAME PANEL TEST METHODS

2.4.1 General

The structural regulations applied to timber frame walls necessitate the use of either an acceptable design method together with proven material performance values, or a standard test method from which basic performance data can be established. The test method may also be required to cover research work from which design methods may be developed in order to reduce the performance data required for the introduction of variations in use of materials.

In view of the vast documented information on axial loading and bending of timber members, the only structural area requiring development through test work was the
determination of the wall racking resistance. When timber framed house construction in Britain started to develop at such a rate that special regulations were considered necessary only one test method was commonly used, the original ASTM E-72 test. This test provided data to back up the American regulations which have been shown to be rather loose in terms of structural performance requirements. It was considered that the much tighter controls proposed by the British Authorities would require a more rigorous test method, more closely examining the behaviour in practice of the wall.

This section details the development of the British test method noting the basic principles incorporated in the tests. Full details have not been given, however, as the test method has been adopted in all the test programmes reported in this thesis. The test procedure and the method of evaluation of the results are covered in Chapter 5, which introduces the programme of wall racking tests.

The American ASTM E-72 test and its later development is noted for comparison with the British method and finally other specialist racking tests favoured by different countries are outlined. In the majority of cases, these tests have been developed to meet the unique requirements of the country, either in terms of applied loading or method of construction, and can be related to either the British or American test philosophy.

2.4.2 The Development of The British Racking Test

The earliest proposal for an independent British racking test to replace the ASTM holding down strap with a system of vertical loads was prepared by Lantos (1967). It is strange to note that TRADA, who were responsible for the report, ignored the vertical load system of testing for a further fifteen years, preferring the ASTM E-72 test from which their design values had been derived. Lantos included cyclic loading and a variable safety factor based on the number of similar panels tested in his proposals. He recommended a deflection limit of 0.003 times the panel height.
The Princes Risborough Laboratories of the Building Research Establishment (PRL) were the pioneers of the British test. Work did not start until late 1967 as earlier that year they published a report, under their old name, (Forest Products Research Laboratory, FPRL, 1967) detailing tests carried out in a horizontal position on panel lengths up to eight feet long using a method similar to the ASTM E-72 test. However, the high uplifts reported inferred that the test was not truly to ASTM standards. By 1969 the laboratory had changed its test method (FPRL, 1969). Panels nailed and screwed to the base of the test rig were tested at zero vertical load, and later under a 157kg stud load, to a deflection limit of 0.003 times the panel height: finally they were loaded to failure. No interpretation of the results for design use was noted.

The first work that was truly representative of the present test method covered two research projects (FPRL, 1971a and FPRL, 1971b). The first programme covered eight feet square plywood and plasterboard panels examining:

(i) vertical load only to failure,

(ii) racking load only to failure,

(iii) racking with vertical load, conducting single cycle stiffness tests to 8.0mm deflection at 0, 500, 1000, 1500, and 2500 lb/stud before testing to failure at 500 lb/stud.

Variations in panel length were also examined and an idea to relate the results to polar rotation of the nails was proposed, but no analysis was attempted. The second programme covered fibre building boards, introduced variations in nail centres and examined the affect of openings in the eight feet square panels. Further work (PRL, 1972) covered variation in board thickness, stud centres and the effect on racking resistance of an end return wall.

Wattie (1973) used a similar test method to develop a wall design method and Thomas (1975) introduced rapid cyclic loading when testing an external wall system comprising
timber frame, plasterboard lining and an external brick slip finish on a weldmesh base. One panel was subjected to 14,400 load cycles to a deflection of 2.7mm in a total time of 10 hours. No deterioration was noted then or in a further similar test. The work covered in this thesis was started in 1975 and development work on the test method was taken over from Princes Risborough Laboratories such that the test method was officially documented (BBA: 1983) as the Princes Risborough Laboratories/University of Surrey (PRL/UofS) test. There is little published evidence of the changes made during that period as the development work was carried out in conjunction with the British Standards Insitution and was incorporated into successive working draft documents which were not made available for public comment. However, the main changes can be traced through test reports prepared at Surrey.

Reporting to Fidor, Griffiths (1976) explained that the test deflection limit had been reduced to 0.002 of the panel height and that a cyclic component had been introduced further altering the method of reduction of the results. The rationale behind the changes was noted and later elaborated (Griffiths, 1978a and Griffiths, 1978b).

Since 1978, the changes to the test method have been minor and have all been incorporated in a Building Research Establishment Research Paper (Mayo, 1984). This paper also includes an attempt at assessing the performance of plasterboard in timber frame walls. More recent work (Griffiths, 1984) has concentrated on the reduction of test results and the application of panel performance to wall design based on a method first published in 1981 (Robertson and Griffiths, 1981).

Finally the test method and the evaluation of test results has been incorporated in the draft Code of Practice for timber framed walls (BSI, to be published). The draft shows no changes to the procedure, although it includes more advice to researchers on how to set up racking tests, noting
areas of potential difficulty. The reduction of the results has been altered in two important details. Firstly the factor of safety required for plasterboard and other materials not specified by the Code has been increased from 1.6 to 2.4 and secondly the basic racking resistance of a combination of materials has been changed to refer to the zero vertical load performance to simplify its use.

A summary of the British test, in its final form is given, to enable comparison with other test methods. A panel is tested in stiffness, and strength with the performance related to the applied vertical load. A stiffness test comprises four cycles of load to a deflection of 0.002 of the panel height taken from a datum fixed in an initial stabilisation load cycle. Stiffness tests can be carried out at up to three different vertical load conditions on one panel. In the failure test, after the required vertical load has been applied, the panel is continuously racked until a maximum value is attained. Design values for a given combination of materials are normally based on tests carried out on more than one identical panel. Stiffness loads are averaged for identical tests, whereas the lowest failure load for identical tests is carried forward. These values are multiplied by partial safety factors to cover the number of similar tests and finally a design value can be calculated if, for a given vertical load both the stiffness and strength performance have been measured. The design load is the lower value relating to a deflection of 0.003 of the panel height or the factored failure load. Thus both types of test performance load have to be modified to find the design load: the stiffness value includes an enhancement factor to allow for the increased deflection and the failure value includes an overall factor of safety. By careful planning of the test programme, the performance of a combination of materials through the practical range of vertical loads can easily be determined. Tests to provide data for material variations are normally carried out on 2.4m panels. The results may be applied to wall designs using modification factors derived from the more complex programme of tests which are the subject of this thesis.
In conclusion, it should be noted that the PRL/UofS test is now the only method acceptable in Britain for providing design data for timber frame construction.

2.4.3 The Development of American Test Methods

The American standard test method for wall panel racking resistance is the ASTM E-72 test. The method was fully adopted by ASTM in 1954, having been developed by the National Bureau of Standards and the U.S. Forest Products Laboratory during the 1930's. It was originally intended as a comparison to pass or fail wall types when judged against a standard configuration. It has undergone only slight alterations through the years, but its use has changed dramatically. In their state of the art paper, Yancey and Cattaneo (1974) comment that section 14 deals with "racking load - complete assemblies" and section 15 and 16 cover "racking load - evaluation of sheathing materials on a standard wood frame". However, ASTM E-72-80 (ASTM 1980) does no more than detail the results required for inclusion in the test report; a method of evaluation is not included.

There has been much criticism of the ASTM E-72 test (Isenberg, 1963) (Yancey and Cattaneo, 1974), and in 1976 Yancey (1976) outlined a new test method including a vertically applied uniformly distributed top load, very similar to the U.K. test at that time. Yancey reviewed alternative test methods tried in America and then presented results of tests conducted to his new method. No attempt was made to show how these results would be used in panel or wall design.

The ASTM E-564-76 (1976) test method is in many ways very similar to Yancey's proposal, differing mainly in the method of fixing the panel; Yancey favoured two M12 bolts, while the Standard, noting the importance of the type and spacing of anchorages, recommends that they duplicate the actual building system as closely as possible. The E-564 test method allows for consideration of vertical loading, although no advice is given on the magnitude of loads or how they should be applied. Calculation methods for the
ultimate shear strength and the racking stiffness are given but are not adapted for design purposes. It is interesting to note that ASTM E-72-80 recommends that where the objective of the test is to measure the performance of a complete wall then the E-564 method should be used.

As a comparison to the British test method, the ASTM racking tests are briefly detailed. The original ASTM E-72 test in its 1980 version uses an eight feet square panel, which is fixed into the test rig and restrained in such a way that the leading edge of the panel cannot lift under application of racking load. No vertical load is applied to the panel, although the tensile force induced into the holding down rods providing the restraint will be considerable and is representative of a concentrated vertical load applied to the leading edge of the panel. The specimen is loaded in three stages to 3.5, 7.0 and 10.5 kN at a uniform loading rate not exceeding 1.75 kN/minute. The loads are removed at the end of each stage and the panel sets noted. Finally, the panel is loaded to failure or until the total deflection of the panel exceeds 100mm. No method of evaluation of the results is given which could be used in a wall design procedure. Details for the presentation of the results are recorded, however.

The later E-564 test does allow top loading. The wall, which should not be less than eight feet in either length or height, should be anchored to the base of the rig so as to duplicate as nearly as possible the system intended for use in actual building construction and provisions may be made for the application of simulated gravity or other loadings simultaneous with the racking load. At least two similar tests should be conducted with the results agreeing within 10%. The method assumes both the design and the failure load to be known as it suggests: the rate of loading to be such that at least 10 minutes is taken to reach design load and, that within the single load to failure test, at loads approximating one third and two thirds of the estimated ultimate load, the racking load is removed and the recovery of the wall noted after 5 minutes. Evaluation of performance
is based on the mean values obtained from identical tests. The ultimate shear strength is calculated as the maximum load divided by the panel length and the shear stiffness, determined at a third of the maximum load, is calculated as the racking load divided by the total racking deflection all multiplied by the panel aspect ratio i.e. the height of the panel divided by its length. No indication is given of any requirement relating the design load to either the ultimate shear strength or the shear stiffness.

There is no evidence of the E-564 test having been used in Britain, however, TRADA (Kay and Aiyanyo, 1967 and others, unpublished) have made wide use of the E-72 test.

2.4.4 Racking Test In Other Countries

In Canada, American practice has in general been followed but in 1979, Parasin made an examination of the PRL/UofS test method and carried out an extended programme of tests on douglas fir and CSP sheathed panels. For the first time, tests were conducted with the panels laid horizontal. This caused no particular problem, but Parasin (1979) reported some difficulty in following the test procedure. His advice was well received in Britain and some of his suggestions incorporated in the test method. The test results (Parasin, 1980) form the basis of COFI's design literature in the U.K.

In Japan, the ASTM E-72 test has been used consistently by Sugiyama, both on full scale panels (1978 and 1985) and on one third scale models (Yasamura, 1983 and 1984). Kamiya (1981a) commenced his work with four different types of test on 1820 x 2730mm panels. Two tests involved the ASTM E-72 method, with different base fixings to the panel, while the remaining test used a method similar to that used in Britain; one test was conducted at zero vertical load and the second under a vertical load of 200kg/m. The final test, with the vertical load, was adopted for the main programme covering different wall lengths and openings. Iizuka (1975)
conducted his test under zero vertical load on a rig specially developed to investigate more traditional Japanese framing methods.

It is clear that for the lightweight houses used predominantly in Japan, a zero or low vertical top load racking test is most suitable. It is noticeable, however, that the panels include strong foundation fastenings often carried into the studs which could motivate a vertical resistance; Kamiya used holding down straps heavily nailed to base and stud and Iizuka used four large 'dogs' per stud.

In New Zealand, the design standard for light timber frame buildings (SANZ, 1984) refer to a standard test method although the test itself is not covered by the standard. The development of the test and the Code ran in parallel as the Code details wall design based on the test results. Collins (1979 and 1980) describes the background to the Code, first published in 1978, but much earlier (1974) he covered the development of the test method. He rejected the ASTM E-72 test, stating that "the tie rods, unless they are incorporated in the building, invalidate the results of the test for engineering design purposes". The wall bracing test was first officially published in 1979 (Cooney and Collins, 1979) and was revised in 1982 after Cooney and Collins (1981) had reviewed their experiences of using the test. The developments in the test between 1974 and 1978 concerned the requirements for earthquake resistance; the double amplitude cyclic test method of progressively increasing amplitudes was chosen in order to approximate the in service conditions representative of earthquake loading. The panels should be installed in the rig in a manner similar to site practice and may be vertically loaded although it is inferred that wind uplift may make vertical loading unnecessary in many design cases. The test procedure is complex, but basically covers a stiffness deflection limit of $\frac{h}{300}$ where $h$ is the panel height, and an ultimate load, although it is not always necessary to test to failure. These results are then manipulated to find the bracing value.
of the panel or the design load in kilonewtons which is 1/20th the bracing value. A test programme can be followed (BRANZ, 1982), but it should be noted that performance cannot be readily related to U.K. figures due to the differences in panel construction and base fixing (SANZ, 1984).

The structural performance of houses in earthquakes is detailed by Cooney (1979) and Cooney and Fowkes (undated) explaining the need for the very specialist New Zealand test method.

Australian testing is based very much on the combination of racking forces and wind uplift. Tests often consider the two independently and then in combination (Frodin and Ross, 1975a, b and c). No specific racking test has been noted; the test procedure follows the standard Australian timber Code which is to load the panel to an assumed design load, then to hold it for five minutes before releasing it and finally to load the panel to failure. Panels tested (Frodin and Ross, 1975 and McDowall, 1980) were 2.7m long by 2.4m high and were fastened at the base by strapping the bottom plate to the transverse joists as would happen in Australian practice. No information was given on the reduction of results.

Reardon (1980) outlined proposals for a wall racking test suggesting a 1.8m long panel, with base fixings, cyclone rods, bracings and sheathings as appropriate to the design. He suggested cycling the panel to the design load in tension and compression before loading to failure. He also gave advice on multi cycle loading tests, proving tests and exploratory testing. Factors quoted by Reardon are very similar to those for load testing in BS 5268 Part 2, (BSI, 1984).
2.5 SUMMARY

To summarize the work on regulations and test methods covering the racking resistance of timber framed walls, Britain and New Zealand are the only countries at present active in assessing design methods and formulating a specific test procedure. However, the New Zealand work is in the main devoted to seismic loading and therefore sets different standards for the walls. This also allows for their consideration of part sheathed walls. Australia has also recognised the need for wall design due to the cyclonic conditions experienced in the South Seas area. They consider only plain walls and allow a very high plasterboard contribution, basing their test evaluation on their standard timber specimen test method. America, Canada and Japan have little need for a specialised design method as they are able to relate most of their designs to traditional standards. They all use the ASTM test method which does no more than check the adequacy of a new form of construction by comparison with the traditional form. The ASTM E-72 test is not suitable for use in establishing a design approach. Its limitations are now being recognised and changes have been made to give a more practical basis to the test. Much analytical work has been carried out in these countries but its value must be questioned if it cannot be related to accurate and practical test performances. The Canadians have begun testing using the British method as a result of their large export market. Scandinavia, the other major exporter to Britain, has not found it necessary to carry out either testing or regulatory work on their product. Racking forces will be lower in the main in Scandinavia and traditional structures have never presented worries over damage caused by wind loading.

Britain is alone in providing a fully detailed design method for full length walls including factors for openings and vertical load. This is all the more surprising considering the fully sheathed walls, the brick skin protection and the comparatively low loading regime. Worries over the adequacy of the timber frame relate directly to the recent introduction of platform frame design to the mass housing market. The British test method attempts to
model panel behaviour in a wall and to convert the test data into design values for use in a comprehensive structural checking system.
Figure 2.1 Timber Frame Buildings: Bays and Frames

Figure 2.2 Comparison of Cruck and Box Frame Construction

Figure 2.3 Interior of A Cruck Frame Barn
Figure 2.4 Balloon Frame Method of Timber Framing (after TRADA)
Figure 2.5 Platform Frame Method of Timber Framing (after TRADA)
Figure 2.6 Base Fixing Using Steel Channels Mounted on a Solid Concrete Floor
Figure 2.7 Base Details In Solid Ground Floor Construction
(Both after TRADA, 1980a)

Figure 2.8 Base Details In Suspended Ground Floor Construction
Floor joists are supported on ledge formed in foundation wall. Joists are toenailed to header and sill plate. Masonry veneer supported on top of foundation wall. Wall framing supported on top of the subfloor.

Figure 2.9 Canadian Suspended Floor Foundation Details (after CMHC, 1979)

Figure 2.10 American Wall Anchoring System (after Anderson, 1975)
Figure 2.11 Framing Around Openings (after TRADA, 1980a)

Figure 2.12 Bracing of Separating Wall Panels (after TRADA, 1980a)
Figure 2.14 The Canadian Platform Frame System (after CMHC, 1979)

Figure 2.13 Assembling The Wall Panels (after TRADA, 1980a)
Figure 2.15 Wall Details For A Two Storey House
(after TRADA, 1980a)
A major part of this thesis covers the development of the British method of testing timber frame walls. The papers covering the development work have already been detailed in Chapter 2. However, the investigations that preceded the decision to prepare this independent test method are reported in the first part of this review. The section covers work carried out in the United States since the early 1930's. Much of the test work was collated by Anderson (1965) to provide a comprehensive design guide for American house builders. In Britain, the early American work has been reviewed by Potter (1968). His work may be considered instrumental in the decision to develop a British test method and his conclusions influenced both the principles of the tests and the early work on wall testing at Surrey, which forms a part of this thesis.

It is the later work, mainly published since 1970, that is most relevant to the current investigation. Such research has been split into two sections. The first covers analytical work, but includes some small test programmes carried out principally to justify the theoretical approaches. The second section details more significant test work, where the data has been used directly, as a means of checking adequate wall performance.

In general, the major contributions to published information on timber frame wall behaviour have come from the United States. However, both sections include work from other countries known to be actively involved in the development of the structural appreciation of timber frame, viz. Canada, Australia, New Zealand, Japan, Scandinavia and Britain. A lack of published work in some of these countries is not necessarily indicative of little work being done, as
often test programmes have been used in the preparation of Standards and have not been independently reported. This is typical of much British work.

3.2 EARLY WORK

In a state of the art review to the Institute of Wood Science in London, Potter (1968) covered some factors in the behaviour of timber frame walls:

(i) type of load,
(ii) transfer of load, and
(iii) design criteria.

He examined, separately, experimental and theoretical analysis reviewing published work, mainly from the United States of America in the period since 1945. His conclusion on the experimental analysis was that, "the American Standard test has been seen to have severe limitation for the purpose of producing data leading to a general design process: some modification, such as that proposed by Lantos (1967) may be extremely suitable for Agrément testing, but it is doubtful whether even this will give data which is applicable to all kinds of construction". His findings on theoretical analyses were that they were all simplifications and assumed the unlikely case of a single concentrated top load. He took the work of Icekson (1966) as holding the most promise. His recommendations for further work are worthy of note as, in part, they form the basis of this thesis:

(i) to refine a standard racking test,
(ii) to correlate data from (i) with small scale tests,
(iii) production of design charts and tables,
(iv) determination of height-length and interpanel behaviour,
(v) consideration of the structural action of traditionally non-structural components, such as plasterboard and insulation boards.
The most important work cited by Potter, was Anderson (1965) as it summarised original test programmes, using the ASTM test, done by: Trayer (1947), Luxford (1953), Luxford and Bonner (1958), which all had relevance to British timber frame practice. Anderson quoted relative rigidities and strengths for:

(i) different types of sheathing, including diagonal bracing, plywood, insulating board, tempered hardboard and plasterboard,

(ii) different fixings, including close nailing and gluing, and

(iii) different types of panel, those with and without openings.

The eight feet square panel was taken as a test standard, but many results came from other sizes of wall and the results were adjusted in a manner which suggested that racking load was proportional to length. The single most important fact concerning this work was that the results were incorporated by TRADA into manuals (TRADA, undated and TRADA, 1980b) and then used to extend their own test results on plywood.

Neisel (1956 and 1958) and Welsch (1963) conducted tests on fiberboard (insulating board) panels and showed a relationship between ultimate racking load and lateral nail resistance.

A report not cited by Potter, Isenberg (1963), surveyed information on racking resistance of walls and presented a critical review of test methods. A new test method, proposed therein, for diagonal loading of panels never really found favour with any test authority and can safely be ignored from further discussion. However, the work highlighted the growing dissatisfaction in America with the ASTM E-72 test, which resulted in the independent approach to testing adopted in Britain.

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3.3 ANALYTICAL RESEARCH

3.3.1 Linear Nail Slip Analyses

Investigations since 1975 have concentrated upon finding an analytical method for timber frame walls, relating to the ASTM E-72 test method. Here the application of racking load distorts the frame into a parallelogram (Figure 3.1a) and is resisted by the cladding which, due to its rigidity, remains rectangular, causing it to move horizontally and rotate (Figure 3.1b). The relative displacement of the cladding with respect to the frame gives rise to the nail movements, shown in Figure 3.1c, and thus to the forces in the nails which resist the racking load on the frame.

The mechanism can be analysed and the racking load expressed in terms of the maximum force in the fasteners and the geometry of the system if the following assumptions are made:

(i) the frames and cladding remain rigid,
(ii) frame members are connected by pins,
(iii) the nail force is proportional to the displacement between the cladding and the frame,
(iv) the overturning of the panel is resisted independent of the cladding,
(v) panel failure is a result of the failure of the cladding fixings,
(vi) external loads are applied to the frame alone.

Kuenzi (unpublished) first put forward an analysis in the 1960's for a symmetrical nailing pattern, which was based on a co-ordinate system with its origin at the centre of the panel (Figure 3.2a). Taking the panel height ($\widehat{H}$) and the width ($\widehat{D}$) as measured between nail lines, and the co-ordinates of a nail as $(x_i, y_i)$, the racking load ($RL$) could be expressed as:
\[
\text{RL}_{\text{max}} = \frac{F_{i \text{ max}}}{h} \sqrt{\frac{\sum_{i=1}^{N} x_i^2 \sum_{i=1}^{N} y_i^2}{\text{max} R_{\text{max}}}}
\]

where \( F_{i \text{ max}} \) is the maximum force on the fasteners, found by small scale tests; \( N \) the total number of fasteners; and \((x_C, y_C)\) the dimension of the most heavily loaded fastener.

Walker (1980) stated that the racking resistance from a single cladding at a deflection, \( \Delta \), could be derived as:

\[
\text{RL} = K A \frac{\sum_{i=1}^{N} x_i^2 + \sum_{i=1}^{N} y_i^2}{h^2 \sum_{i=1}^{N} x_i^2 + \sum_{i=1}^{N} y_i^2}
\]

where \( K \) is the stiffness of an individual fixing. He also showed that if stiffeners were uniformly spaced around the cladding perimeter at a distance, \( S \), the racking resistance could be approximated to:

\[
H = C_2 \frac{KA}{S}
\]

where \( C_2 = h \left( \frac{b}{h} \right)^2 \left( \frac{3 + \frac{b}{h}}{1 + 3 \frac{b}{h}} \right) \left( \frac{1 + 3 \frac{b}{h}}{1 + 3 \frac{b}{h}} \right) \)

\[
H = \frac{6\left( \frac{b}{h} \right)^2 \left( 3 + \frac{b}{h} \right) \left( 1 + 3 \frac{b}{h} \right)}{\left[ \left( \frac{b}{h} \right)^2 \left( 3 + \frac{b}{h} \right) + \left( 1 + 3 \frac{b}{h} \right) \right]}
\]

giving values of 0.067 and 2.5 for 0.6 and 1.2m long panels respectively.

Burgess (1976a, b, c) and Tuomi and McCutcheon (1978) took a different approach in which they assumed the fasteners to be positioned to form \( n \) equal intervals horizontally and \( m \) equal intervals vertically with the diagonal of the cladding panel making an angle \( \alpha \) with the vertical (Figure 3.2b).
Again, assuming only perimeter nails:

$$RL_{\text{max}} = F \max \sin \alpha \left[ n + m - \frac{2}{3} \left( \frac{n^2 - 1}{n} \right) \cos 2\alpha - \frac{2}{3} \left( \frac{m^2 - 1}{m} \right) \sin 2\alpha \right]$$  (3.5)

Tuomi and McCutcheon used studs at 400mm centres and thus had two "inner field" nail rows requiring a further expression; however, with a single row of nails for studs at 600mm centres the second expression can be ignored.

Walker (1978) simplified the expression, assuming $n$ and $m$ to be greater than five, to:

$$RL_{\text{max}} = C_{\text{max}}$$

where

$$C_{\text{max}} = \frac{F \max}{s}$$  (3.6)

$$C_1 = \frac{5(5+\bar{h})}{\sqrt{\frac{2}{b} + \frac{2}{h}}} \left[ 1 - \frac{2}{3} \frac{5}{\bar{b} + \bar{h}} \frac{h}{2} \right]$$  (3.7)

giving values for $C_1$ of 0.57 and 1.14 for 0.6 and 1.2m long panels respectively, again noting $\bar{h}$ and $\bar{b}$ to be dimensions between nail lines.

Tuomi and Gromala (1977) developed the theory further noting that the ultimate panel racking strength $RL_{\text{max}}$ should be increased by the racking strength of the frame. They also calculated the performance of a let-in wood brace and made comparisons with small scale test results using 2 feet square panels.

Burgess (1976a), noting the panel deflection due to nail slip to be:

$$\Delta = \frac{2V}{\sin \alpha}$$  (3.8)

gave the panel modulus as $\frac{RL}{\Delta}$ and used the result to calculate the racking load of combinations of panels and multi-sheet panels cladding around openings. For work involving
calculation of deflection he then introduced a term for the shear distortion of the plywood. Then (1976b) he compared his work with that of Kuenzi, noting very similar performances but commenting that the two formulae had not been shown to be convertible. He also noted the failure of both methods to predict the strength of weak fibreboard accurately, saying that this was due to the inability of the method to cover the plasticity of the nail failure in the racking test.

Itani et al (1982) took Tuomi and McCutcheon's work and developed equivalent diagonal braces for the panels (Figures 3.3, 3.4 and 3.5) calculating the brace stiffness to be:

\[ K = \frac{k}{4} \left[ n + m - \frac{2}{3} \left( \frac{n^2 - 1}{n} \right) \cos^2 \alpha + \left( \frac{m^2 - 1}{m} \right) \sin^2 \alpha \right] \]  

(3.9)

where \( K \) is the brace stiffness and \( k \) is the nail slip modulus, and assuming no assistance from nailing of inner studs. They extended the analysis to cover typical walls with openings. However, the method used only full height panels, calculating the contribution of each sheathing, but ignoring any increase in strength from extensions in 'L' and 'C' shaped boards, (Figure 3.4). This paper represents the final development of the research based on the assumption that the nail force is proportional to displacement. Soltis et al (1981) stressed that this assumption was an oversimplification and concluded "more emphasis be placed on characterizing wall-racking stiffness".

3.3.2 Non-Linear Nail Slip Analyses

An early published work using non-linear nail behaviour with the other ASTM test assumptions was by Kamiya (1981) and was expanded by Kamiya et al (1981a, 1981b and 1983). He used a series of straight line segments to approximate nail slip and, by acceptable approximations, arrived at a numerical solution that "is so simple a programmable electronic calculator is enough for the calculation". His technique
allows the modelling to be carried to failure and showed
good correlation with test results, although one problem
noted was that of stud bending.

McCutcheon (1985) presented a method similar to
Kamiya, but used a power curve of the form:

\[ p = A x^B \] (3.10)

where \( p \) is the nail load, \( x \) its deflection, \( A \) the amplitude
and \( B \) the exponent which lies between 0 (plastic response)
and 1 (linear response). The racking load was shown to be:

\[ RL = \bar{A} \Delta_N^B \] (3.11)

indicating the panel performance to be defined by the same
power curve, but related to an amplitude \( \bar{A} \) dependent on nail
amplitude, the exponent and the panel and nailing geometry.
In the equation (3.11), \( \Delta_N \) represents the deflection due to
nail slip; if shear deflection is also considered, then:

\[ \Delta_T = \left( \frac{RL}{NA} \right)^{1/B} + \frac{RL x H}{N G t B} \] (3.12)

where \( \Delta_T \) is the total deflection,
\( N \) is the number of independent vertical sheets in
a panel,
\( t \) is the thickness of the sheathing,
and \( G \) is the modulus of rigidity.

McCutcheon showed that the shear component was
proportionately high at low load, but its significance
reduced as load increased. Problems with initial slip on
individual nail tests led him to back analyse the results of
small scale, two feet square, panel tests to determine the
nail power curve which then gave good correlation with the
initial deflection of the full scale panels, but under-
estimated deflection at higher loads.
The small scale tests used by McCutcheon are the same as those reported by Patton Mallory et al (1983). The two feet square panel test was proposed as a replacement for the full scale ASTM E-72 test as a means for obtaining material parameters on the basis that it was considerably cheaper and that the full scale test did not predict panel performance within walls. The small scale tests were extended to cover aspect ratios (b:h) up to four and the effect of openings (Patton Mallory et al, 1985).

The assumptions made for wall deflection by Tuomi and McCutcheon (1978) which formulated the FPL wall racking equation were questioned by Easley et al (1982). Their impression of fastener movement and thus fastener forces is shown in Figures 3.6 and 3.7 and was based on experimental observations. They used two relationships, one linear and one non-linear to describe nail load versus slip and thus determine racking loads. Considering first the linear relationship, where \( p = kx \), then:

\[
RL = \frac{\Delta T b}{h\left(\frac{2h}{Kb} + \frac{1}{G t}\right)}
\]

where \( \beta = n_s + \sum_{i=1}^{\eta_e} \frac{x_{ei}^2}{b^2} \)

where \( n_s \) is the number of side fasteners and \( n_e \) the number of end fasteners and the panel has been simplified from that in Figure 3.7 to one with a single central stud, making \( x_{S1} \) zero. The equation (3.13) includes the shear deflection of the board to be:

\[
\Delta s = \frac{RL \times h}{G t b}
\]

which is similar to equation 3.12.

The non-linear load relationship is shown in Figure 3.8 where \( C_1, C_2, C_3, C_4 \) and \( F_{So} \) are constants adjusted to fit test data for a particular case. Thus the panel strains
were shown to be:

\[ \Delta_T = \frac{RL \times h}{G t b} - \frac{2h}{c_2 b} \left( 1 - \frac{RL \times h}{b \beta c_1} \right) \text{ for } \frac{RL}{b} \leq \frac{F_s \beta}{h} \]  

(3.15)

and \[ \Delta_T = \frac{RL \times h}{b G T} + \frac{2h}{c_3 b} \left( \frac{RL \times h}{b \beta} - C_4 \right) \text{ for } \frac{RL}{b} \geq \frac{F_s \beta}{h} \]  

(3.16)

which compares with the linear value, taken from equation (3.13):

\[ \Delta_T = \frac{RL \times h}{b} \left( \frac{1}{G t} + \frac{2h}{k b \beta} \right) \]  

(3.17)

Thus it can be seen that the non-linear relationship is best solved by load stepping. Easley et al (1982) checked their results with experimental work and a finite element programme and obtained good correlations. Castillo and Gutkowski (1984) criticise Easley on a number of points, including the deflected shape. However, it has been found in the current investigation that the deflection patterns, particularly for combinations of sheets (even in 2.4m square panels), follow very much closer the Easley model than earlier assumptions (Burgess, 1976 and Tuomi and McCutcheon, 1978).

In a more recent paper (Sugiyama and Tokuda, 1985) full scale ASTM tests, heavily instrumented, showed that Easley's assumptions were correct at low vertical loads, but that Tuomi and McCutcheon's deflection pattern was true after approximately 35% of the maximum load had been reached.

Gupta and Kuo (1984) considered the work of both Tuomi and McCutcheon (1978) and Easley et al (1982) and detailed a more general method which did not impose the restraints on the panel of the previous analyses. Referring to Figure 3.9, their proposed models made \( \alpha \) and \( \beta \) independent and non zero, whereas Easley assumed \( \beta = 0 \) and Tuomi and McCutcheon constrained \( \alpha \) and \( \beta \) such that the relative displacement of the corners were along the diagonals. They
also allowed the studs to deflect sinusoidally. Their combined deflection pattern on the three panel wall used by Easley is shown in Figure 3.10. They compared their model analysis with Easley's finite element results and test data but in doing so, they increased the modulus of rigidity used in the Easley model from $90 \text{Kips/sq.in.}$ to a more realistic value of $692 \text{Kips/sq.in.}$. Gupta claimed his model solution to be in good agreement with that of the finite element analysis, and, since it was simpler, more suitable for extensive applications as in repetitive non-linear dynamic analysis. Adjusting the model to make the stud infinitely rigid had little effect on the results and further simplified the model.

### 3.3.3 Japanese One-Third Scale Tests

Some help in determining the deflected shape of panels during the ASTM E-72 test can be obtained from Japanese work. Yasumura and Sugiyama (1983 and 1984) carried out one-third scale tests on 1.2m long by 0.91m high panels, using a miniature ASTM E-72 test rig. In 1984 they discussed the board movements in the window panels and found the relationship between both shear strength ($F$) and shear stiffness ($F$) versus opening coefficient ($r$) to be:

$$F = \frac{r}{3 - 2r}$$

(3.18)

Full details are shown in Figure 3.11 where the complexity of the opening coefficient should be noted. In their earlier work, (Yasamura and Sugiyama, 1983) they commented that to reduce the effect of openings, the sheathing units should be as large as possible and that load cycling at 60% of the maximum load had no influence on the final strength of the panel.

In a previous report, Sugiyama (1978) stated his preference for the ASTM E-72 test and in referring to the effect of wall length noted that there was a direct relationship between wall length and load for different strain levels and the maximum load (Figure 3.12). He further stated that
the relationship was true for both the ASTM test and the Japanese test with no tie rods. However, the size of boards incorporated in the panels is unknown and the maximum length for which the relationships are noted is short in terms of practical wall length.

3.3.4 Finite Element Analyses

A significant development in the use of finite element programs to analyse timber frame structures can be attributed to Castillo and Gutkowski (1984). They reported on the "Non-linear Analysis of Wood Shear Walls", and later summarised their work in a further paper (Gutkowski and Castillo, 1984). They described the preparation of the WANELS program which is an analytical model for timber frame walls, including the main structural components; sheathing panels, stud-frame members, semi-rigid frame joints, panel to frame connections and gaps between discontinuous panels. Non-linear connector load-deformation relationships are incorporated using a rapidly converging step-wise technique. A matrix flexibility model was also set up to perform the linear analysis of wall panels, but it was noted that WANELS could be adapted to solve these problems. The program was tested on the small scale panel tests reported by Patton Mallory et al (1983), firstly using their data for nail load/deformation characteristics and then using McLain's empirical relationship (1975) to achieve a much better correlation. Comparison was also made with methods of analysis used by Easley (1982) and Tuomi and McCutcheon (1978).

Unfortunately the WANELS program is considered unsuitable for use with the British test method as it is not capable of coping with the vertical load. Thus, as detailed in Chapter 7, it was not considered for use in the computer analysis.

The Finite Element program, SADT, written by Foschi (1977) and the one adopted in the current investigation, is more general than the WANELS program and was more readily available and easier to adapt to the very specialised requirements of the Surrey test method. Full details of the program are given in Chapter 7.
3.3.5 Other Analytical Methods

Two entirely different approaches to those previously mentioned have been reported. The first was investigated by Nicol-Smith (1978) and extended the work of COFI on diaphragms to include plywood sheathed walls. The analysis assumed the wall to behave as a cantilever under racking load and assumed the deflection to be composed of bending shear and nail slip such that:

$$\Delta_T = (8 \frac{RL}{b^2} \times \frac{h^3}{EA}) + (\frac{RL}{b} \frac{h}{Gt}) + (0.375 h \Delta_N) \quad (3.19)$$

where $E$ is the modulus of elasticity of the flange,

$A$ is the cross-sectional area of the flange,

and $\Delta_N$ is the nail slip.

The method is not suitable for walls with inadequate anchorages and large openings, amongst other conditions, and certainly cannot be suggested for use with the British test method.

The second was reported by Leppävuori (1982) to be a linear nail slip method which attempted to cover vertical loading. A simplified panel with no internal studs was considered and the vertical and horizontal loads (Figure 3.13a), arranged to give the forces in Figure 3.13b, where only $RL$ and $V$ affect the panel. The displacement $\Delta_T$, which was made up of a nail slip component and a shear component, was given by:

$$\Delta_T = \left(1 - \frac{hxRL}{Dm} + \frac{1}{Dx} + \frac{1}{Dy}\right) \frac{h^2RL}{a} + \frac{hxRL}{a} \quad (3.20)$$

where, referring to Figure 3.14c:

$$Dm = \int_0^b kx^2 dx = \frac{1}{3} k b^3$$

$$Dx = \int k_n^2 ds = k \left(\frac{1}{2} bh^2 + \frac{1}{6} h^3\right)$$

$$Dy = \int k_t^2 ds = k \left(\frac{1}{2} b^2h + \frac{1}{t} b^3\right)$$
The proof of this equation was given by Rautakorpi and Leppänen (1976). The method was shown to give good consistency with test results, but only seemed to work when the vertical force was very high and therefore could be concluded to be equivalent to conducting the ASTM E-72 test. The experimental work investigated double sheathing, nail spacing, gluing and both 1.2m and 2.4m panel lengths.

Recent investigations in New Zealand treat walls in a similar manner to that for horizontal diaphragms (James and Bryant, 1984 and Dean et al, 1984). Stewart et al (1984) reviewed the effect of seismic loading on timber frame walls and examined the effects of load reversal and load cycling. In every case the work was very specialised and is not appropriate to this investigation.

Thurston and Hutchison (1984) also reported on cyclic loading tests and related their results to small scale tests and a new design method. They noted that in Easley's analysis for a standard sheet with $h/b = 2$, the deformation of the framed sheet was four times the nail slip in the studs, i.e. $\Delta_N = 4\Delta_y$. With reference to Figure 3.14b, they argued that a better approximation of the distortion would be:

$$\Delta_N = 2\Delta_x + 4\Delta_y.$$

They quoted work which showed $\Delta y = 1.43 \Delta x$ thus:

$$\Delta N = 7.72 \Delta x, \quad \text{or} \quad 5.40 \Delta y.$$  \hspace{1cm} (3.21)

It may be noted that Tuomi and McCutcheon (1978) assumed $\Delta_y = 2\Delta_x$ giving:

$$\Delta_N = 10.00 \Delta_x, \quad \text{or} \quad 5.00\Delta_y.$$  

Thurston and Hutchison's own test work (Figure 3.13c) supported their theory in comparison with that of Easley, but it is not clear as to what level of overall deflection in the panel the results refer, noting the comments of Sugiyama and Tokuda (1984) mentioned earlier in this section.
3.4 TEST PROGRAMMES

3.4.1 Panel Tests Carried Out Abroad

The majority of test work carried out in the United States uses the ASTM E-72 test method allowing the results to be compared with the FPL racking equation formulated by Tuomi and McCutcheon (1978). The American Plywood Association has made a very detailed study of the use of plywood in timber frame walls. The major work undertaken by Adams (1983) covered eight feet square panels and investigated:

(i) different thicknesses of plywood,
(ii) different fixing types,
(iii) different nail spacings, and
(iv) the combined use of plywood and plasterboard.

Design values for both wet and dry conditions, in terms of racking load per foot length of panel, were quoted, but the report does not explain the calculation of the design load and it can be seen that there is neither a consistent factor of safety nor a standard deflection limit. The value quoted for a panel similar in general details to the standard plywood panel tested in Chapter 6 was 200 lb/ft (2.9 kN/m).

Tissell (1983) investigated a special use of plywood in providing corner bracing only. His tests used eight feet square panels with either a single four feet wide sheet of plywood vertically fixed or one sheet of ply followed by one of insulating board. The results could be compared with Adams' work (1983) and Tissell stated that the walls detailed below would meet the requirements of the Federal Housing Authority:

(i) a single eight feet square wall comprising two four feet wide full height sheets, one of \(\frac{1}{2}\)" plywood and the other of \(\frac{1}{2}\)" insulating board, both nailed at 4" centres externally and 8" centres to internal studs;

(ii) two separated four feet sheets of \(\frac{1}{2}\)" plywood with nails as in (i) above,
(iii) three separated four feet sheets of $\frac{5}{16}$" plywood with nails at 6" centres externally and 12" centres to internal studs.

Tissell (1981) had covered another specialist construction; that of shiplap joints between plywood sheets. His tests indicated a weakness in using a single row of nails passing through the covering board only at the shiplap joint. However, the deficiencies were not as great as might have been expected, due to the 400mm centres of the studs and the use of plasterboard for an internal lining.

Rose (1977 and undated) carried out two test programmes on plywood sheathed panels. The first set is of limited value to British investigators as it covered panels used in the American "All Weather Wood Foundation" construction, where laboratory tests were checked against in-ground performance. His second paper (undated) covered tests on twelve feet long plywood shear walls as used in mobile homes. Rose concluded that the ASTM E-72 method was unsuitable due to the very low vertical loading on these walls. His tests followed a procedure similar to that of the ASTM E-564 method. The work covered window openings and other special cases including the use of cedar siding and the use of hardboard internal lining (Figure 3.15 and Table 3.1). He found that strength was the governing factor in these tests.

Tests 2, 4 and 5 (Figure 3.15) investigated the layout of plywood around an opening and the results indicated that the shear wall stiffness was greatest when the sheathing was continuous at the window corners as in test 4. Comparison of tests 1 and 1A showed the great improvement gained from the sheathing underneath the window which emphasises the over-simplification of design methods which only consider full height sheathed walls, such as Itani et al (1982) and TRADA (1980b).

Sheathing materials, other than those commonly used in timber frame walls, have been tested. In America, Price

One of the most detailed studies of different sheathings was undertaken by Iizuka (1975) in Japan. His tests were very specialised and used mainly 0.9m long by 2.4m high panels. They required a special rig to meet the particular needs of the traditional Japanese wall panel and therefore cannot be directly compared with the present studies.

In Australia, test work on plywood has been reported by Frodin and Ross (1975a, b and c) and McDowall (1980) and their work has led to design values being published. The design value for the construction most similar to the standard panel test reported in Chapter 6 was 2.25kN/m for a zero vertical load condition. Unpublished work from James Cook University in Townsville has led to design figures used in trade brochures for plasterboard (CSR, 1982) and tempered hardboard (Hardboards Australia Ltd., 1978).

The test method in New Zealand is related to seismic loading. Although its development has been well documented, as described in Chapter 2, little data based on the test have been published. The New Zealand Standard for Light Timber Frame Buildings (SANZ, 1984) gives design values for many panel configurations and it must be assumed that this work has been carried out internally at the Forest Products Research Laboratory at Rotorua or has come from unpublished commercial test reports. Work on 6mm Hardiflex cellulose cement board was reported by BRANZ (1982) and thus extends the data contained in the Standard. The design values for plywood which, allowing for approximation in the panel design, may be compared with the zero vertical load case of the standard panel test in Chapter 6, were 3.33kN/m for plain panels less than 1.8m long, rising to 4.15kN/m for longer
lengths. However, it should be noted that the method of holding down, in which the studs were directly fastened to the base plate, could have induced a vertical load of up to 2kN per stud. An example of this effect is given in Chapter 6.

3.4.2 British Test Work

Very little test information on timber framed walls has been published in Britain apart from a paper by Robertson and Griffiths (1981). This does not mean, however, that this area has been neglected; most of the testing has been carried out for private organisations and has remained unpublished.

Griffiths (1978b) commented on various factors affecting the racking performance of walls tested by the PRL/UofS method. He investigated the effect on wall behaviour of:

(i) vertical load,

(ii) panel length, and

(iii) openings.

He proposed a small-scale tension test for the approximate determination of the performance of a sheathing/fixing/frame material combination. Finally, he carried out tests on a combined brick and timber frame wall and was able to show a substantial increase in performance using Chevron ties as the only medium for transmitting load into the brick wall. In 1978 the knowledge on openings was limited and Griffiths' design method could only cover full height panels. He derived a design equation for use with nailed panel board where:

\[ RL = (0.44U + 4.46)L + (g-1)(0.14U + 4.14) \]  \hspace{1cm} (3.22)

where \( U \) is the vertical load in kN/stud and \( L \) the wall length. The term \( g \) represents the additional restraint which in practice would be applied to the leading stud of the panel. He pointed out that its value would lie between zero, when the stud was the first in the whole wall and no account was taken of the return wall, and unity, when the stud followed
a fully sheathed wall panel. The test work on openings was insufficient to establish values of $g$ for intermediate cases, where the panel followed door or window openings. The coefficients used in equation 3.22 were relevant only to those materials tested. However other coefficients could readily be established for different materials.

This design method was not developed and soon afterwards an alternative set of proposals was outlined (Robertson and Griffiths, 1981). Their report summarised and attempted to synthesise a vast quantity of test data from different sources from which they derived a design method. This centred on a basic design load for a sheathing assuming a minimum standard of frame material and nailing, and a $5.2\text{kN/m}$ ($2.5\text{kN/stud}$) vertical load. The basic design value, in $\text{kN/m}$ length of panel was multiplied by modification factors covering:

(i) vertical load,
(ii) panel or wall length,
(iii) door or window openings,
(iv) nailing,
(v) panel height, and
(vi) duration of load.

Their work formed the basis for the design method included in BS 5268 Part 6 (BSI to be published) but at that time, their results were limited and many of the design values and modification factors have since been changed. The authors also examined the effects of:

(i) sheathing both faces of the panel,
(ii) stud spacing, size, density and moisture content,
(iii) foundation fixings,
(iv) wetting of sheathings.

They commented on the contribution of plasterboard to racking resistance, noting the problems of age effect, variation in quality, likelihood of alteration and the effect of wetting.
However, they suggested a conservative approach, whereby the plasterboard was not allowed to be the sole provider of racking resistance and its contribution was limited to a proportion of the enhancement it could be shown to provide in tests.

Investigations into the performance of sheathing materials have been carried out for board manufacturers and board promotion associations such as FIDOR and the CPA. The University of Surrey has been the principal test agency and a list of their reports is noted in Appendix 2. The majority of the work however, is included in the results of the standard panel test in Chapter 6. The most significant test report was that written for the Department of the Environment, which covered a wide range of different tests for the specific purpose of providing values for modification factors for use in BS 5268 Part 6 (BSI, to be published).

Further short test programmes, primarily considering aspects of timber frame wall behaviour, have been reported by project students at Surrey (Dillon, 1980; Randall, 1981 and Baughurst, 1986). The significance of these tests, in terms of the more general design of timber frame walls, is investigated in Chapter 6.

In recent years, racking tests have been carried out at TRADA using both the ASTM E-72 and the PRL/UoFS test methods. Currently, work is also being undertaken at the Polytechnic of the South Bank, investigating the combined behaviour of brick and timber frame walls and at Imperial College, investigating analytical techniques.

Outside Britain, but related to the PRL/UoFS test method, extensive tests have been carried out on plywood by COFI in Vancouver (Parasin, 1980). A basic racking resistance, at zero vertical load, of 2.22kN/m was obtained based on tests on six identical panels; strength was the governing criteria throughout. Very little difference between Douglas fir and Canadian softwood plywood was noted.
3.4.3 Whole House Testing

Whole house tests have been undertaken in a number of countries. In general the cost of testing is prohibitive, and the complexity of the tests, covering the interaction of all the components, makes it difficult to compare the results directly with standard panel tests. Yokel et al (1974) tested a two storey conventional house shortly after construction. The building was heavily instrumented but the deflections obtained were very small under the 15 lb/sq ft wind load.

Tuomi and McCutcheon (1974) built and tested a special house in the laboratory. They noted the racking resistance to be more than adequate, but observed weak links in the connection systems between the sole plate and the floor and later the sill plate connections.

Another laboratory volumetric test was carried out by Hirashima et al (1981b) which was linked to their analytical work and panel tests (1981a and c). By keeping the structure simple it was possible to relate the full-scale performance to that of the parts using the equation:

\[ F = \frac{p_F}{\Sigma p_u} \]  

(3.23)

where \( p_F \) is the resistance force of the whole house and \( \Sigma p_u \) sums the force of each element in the racking test. The coefficient of full-scale test, \( F \), was then plotted against deformation; the results are shown in Figure 3.16. The 'blind' wall results show that if only the full height panels are considered, then both the wall and full house performances are greatly underestimated.

Possibly the most productive centre for whole house tests is the Cyclone Testing Station at Townsville in Australia, which was set up as a result of Cyclone Tracey which so devastated Darwin in 1974. The station is engaged on testing wind tunnel models, structural elements and whole houses in order to determine the critical weaknesses in house
construction in Australia and the South Sea Islands. Their reports (Boughton, 1982 and Boughton and Reardon, 1982, 1983 and 1984) have indicated that at high stress levels, damage to racking panels was not severe and, therefore, that wall racking resistance was only a minor problem. Internal plasterboard partitions alone were capable of transmitting a very high load to the foundations. This factor is of particular importance in cyclone conditions when the external walls may be very badly punctured by flying debris. However, the consequences are of greater significance as the tests have enabled plasterboard to be accepted as a structural material in resisting racking loads in all domestic buildings. Figure 3.17 shows the whole house test rig in elevation, showing how the effect of the wind loads in terms of uplift and racking are applied to the building. The raised construction of the building on concrete or timber piers is typical of North Australian practice and it was in the bracing of these piers, which are often removed by the occupiers of the house, that the main source of weakness was found.

In Britain, whole house tests have been undertaken by the Building Research Establishment at Garston but, as yet, the results remain unpublished.
Figure 3.1 Behaviour of Elements in the ASTM E-72 Test
(after Walker 1987 and Patton Mallory et al 1983)
Figure 3.2 Shear Resistance Nomenclature (after Walker 1978)

(a) general

(b) Burgess/Tuomi methods

Figure 3.3 The Equivalent Brace Method (after Itani et al 1982)
A) TYPICAL WALL  
B) ANALYSIS MODEL

A typical wall (A) and its equivalent analytical model (B). $R =$ racking force; $K =$ stiffness of diagonal member; $K_1, K_2, K_3 =$ stiffness of support springs.

Figure 3.4  The Equivalent Brace Method  
Applied to a Standard Panel  
(after Itani et al 1982)

Figure 3.5  The Equivalent Brace Method  
Applied to Walls with Openings  
(after Itani et al 1982)
Figure 3.6 (above left)
Deformation Pattern Assumed in Easley's Non-Linear Analysis

Figure 3.7 (above right)
Easley's Sheathing Fastener Force

Figure 3.8 (below right)
Non-Linear Nail Load-Slip Relationship Used in Wall Racking Theory

(All after Easley et al 1982)
Figure 3.9  Deformation Pattern of a Single Panel Wood-Framed Wall

Figure 3.10  Deformed Shape of the 3-Panel Wall

(Both after Gupta and Kuo 1984)
Figure 3.11 Small Scale Tests on Panels With Openings (after Yasumura and Sugiyama 1984)
Figure 3.12 The Relation Between Shear Load and Wall Length Due to A.S.T.M. Method

Figure 3.13 Simplification of Applied Forces and Panel Notation (after Leppavuori 1982)
Figure 3.14 Theoretical and Tested Nail Deformations in a Wall Panel
(after Thurston and Hutchinson 1984)

<table>
<thead>
<tr>
<th>Test</th>
<th>Deflection (in.) @ Shear Load = 1600 lbs (Gage (1)-(2))</th>
<th>Ultimate Load (lbs.)</th>
<th>Tentative Design Shear Load (lbs.)</th>
<th>Load Factor (a) (b)</th>
<th>Deflection (in.) @ Tentative Design Shear Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>1.071</td>
<td>3040</td>
<td>850</td>
<td>3.58</td>
<td>0.50 (L/180)</td>
</tr>
<tr>
<td>1</td>
<td>0.315</td>
<td>4780</td>
<td>1600</td>
<td>2.99</td>
<td>0.32 (L/281)</td>
</tr>
<tr>
<td>2</td>
<td>0.278</td>
<td>5760</td>
<td>1900</td>
<td>3.03</td>
<td>0.33 (L/273)</td>
</tr>
<tr>
<td>3</td>
<td>0.148</td>
<td>6020</td>
<td>2000</td>
<td>3.01</td>
<td>0.20 (L/450)</td>
</tr>
<tr>
<td>4-1</td>
<td>0.208</td>
<td>7320</td>
<td>2100</td>
<td>3.23</td>
<td>0.27 (L/333)</td>
</tr>
<tr>
<td>4-2</td>
<td>0.211</td>
<td>6620</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-3</td>
<td>0.184</td>
<td>6380</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 (Avg.)</td>
<td>0.201</td>
<td>6773</td>
<td>2100</td>
<td>3.23</td>
<td>0.27 (L/333)</td>
</tr>
<tr>
<td>4a</td>
<td>0.178</td>
<td>8800</td>
<td>2900</td>
<td>3.03</td>
<td>0.35 (L/257)</td>
</tr>
<tr>
<td>5</td>
<td>0.195</td>
<td>7740</td>
<td>2600</td>
<td>2.98</td>
<td>0.34 (L/265)</td>
</tr>
<tr>
<td>6</td>
<td>0.296</td>
<td>4100</td>
<td>1400</td>
<td>2.93</td>
<td>0.24 (L/375)</td>
</tr>
<tr>
<td>7</td>
<td>0.255</td>
<td>5480</td>
<td>1800</td>
<td>3.04</td>
<td>0.30 (L/300)</td>
</tr>
<tr>
<td>8</td>
<td>0.349</td>
<td>5060</td>
<td>1700</td>
<td>2.98</td>
<td>0.37 (L/243)</td>
</tr>
</tbody>
</table>

1/ Results are based on tests of 8-ft x 12-ft (142 in.) plywood shear wall.

2/ Tentative design shear load is based on approximately 33% of ultimate load, or deflection of 1/2-in. (L/180), whichever is less. Recommended design shear load for these constructions have not been established, and are dependent on finalization of design method currently under development.

Table 3.1 A Summary of APA Plywood Shear Wall Tests 1/
**TEST 4**

Test 4a
additional blocking to prevent plywood buckling

**TEST 7**
Additional cedar siding

**TEST 1**

**TEST 2**
No glue

**TEST 3**
Additional hardboard cladding glued internally

**TEST 5**

**TEST 6**

Flywood sheathing nailed and glued unless otherwise stated

Figure 3.15 A P A Plywood Shear Wall Panel Layout
Figure 3.16 Japanese Whole House Tests
(after Hirashima et al 1981)

Plan of test house.

Elevation of test house.

South elevation.

Coefficient of full-scale effect

\[
\text{Coefficient} = \frac{\text{Test Result}}{\text{Predicted Result}}
\]
Racking forces were applied using the hydraulic ram (a) and a cable runover pulley (b) attached to the end wall of the house at (c).

Uplift forces were applied using the hydraulic ram (d) which pulled downwards on one end of a large "see-saw" beam (e). The other end lifted a load spreader (f) which distributed the uplift loads to the roof battens.

Figure 3.17 Whole House Test Rig at the Cyclone Testing Station
(after Boughton and Reardon 1982)
CHAPTER 4

AN INTRODUCTION TO THE DESIGN
OF TIMBER FRAME HOUSES

4.1 THE NEED FOR A DESIGN METHOD

The need in Britain for a design guide for timber frame houses based on calculations for wall racking resistance was noted in Chapter 3. It is a result of the system having only recently been introduced to Britain and that its rise in popularity has come at a time when there has been a general trend to engineered structures in building as a departure from "deemed to satisfy" construction regulations. A design requirement does not imply in any way that the system is structurally unsound. The reliability of timber frame houses throughout the world is known and accepted by the building authorities and it is anticipated that any codified design method would reflect this experience.

The availability of a single standard to cover the design and construction of timber frame houses will help restore confidence in the system and make it more competitive with traditional housing for the following reasons.

(i) A uniform approach to design can be achieved.

(ii) The important areas of design can be highlighted which, together with point (i), will assist local authorities without specialist knowledge of timber frame in checking building proposals.

(iii) Design values can be seen to be based on independent unbiased investigations rather than the claims of the material suppliers.

(iv) The use of design rules will help in specifying the quality of materials and constructional details, which will assist building inspectors in identifying unacceptable levels of construction.
In seeking a format for the design procedure, little assistance was available from countries where timber frame is the traditional construction form. Here, unless there are specific loading regimes requiring design (e.g. cyclones and earthquakes), buildings are required to conform to acceptable practice. This mainly covers sizes of materials and sizes and spacing of nails. Nowhere was there to be found either a comprehensive empirical or realistic analytical design method. Where test results existed, they related to panel tests only, and their extension to cover practical walls was through the use of rather controversial analytical models. The extrapolation of the test data was very rarely checked by full scale wall tests.

In many cases the analyses were unable to cover factors thought to greatly influence wall behaviour (such as vertical load). The high degree of sophistication included in the analyses made them less suitable for hand calculation and did not match the variability to be expected in material and construction quality. As a consequence, the draft British Code of Practice for Timber Frame Walls BS 5268 Part 6 (BSI, to be published) has been based on test experience gained in the UK, mainly in the preparation of this thesis.

4.2 PROBLEMS ASSOCIATED WITH ASSESSING THE PERFORMANCE OF BUILDINGS

The assessment of the wind load applied to a building and causing racking forces is a normal starting point in checking the adequacy of the structure against horizontal load. Design guidelines are clearly laid down in CP3 ChV Part 2 (BSI, 1972) and may readily be applied in the calculation of the forces due to a once in fifty year wind load. In standard practice it would then be necessary to trace the path of the load through the structure checking the adequacy of the elements at every step. If the wind is considered, at different times, to act on each face of the building, it
will first be necessary to check the continuity through the structure which may affect the loading on separate domestic units. In the case of single buildings without construction breaks it is acceptable to simplify the loading to the overall force on the windward face of the structure but more care will be necessary in terraced house construction when the load could be separated into pressure and suction components on the windward and leeward face walls.

Considering the typical British construction form the wind load will be applied to the outer brick skin wall which, depending on the effect of stiff corners, will shield the timber frame from some of the applied load. The remaining load is applied through the brickwork face walls to the timber frame through the brick ties which act in compression on the windward face (and in tension on the leeward face). The timber frame walls distribute the load into the horizontal diaphragms and the racking walls, i.e. those acting in the line of the wind load. The face walls would normally be considered to span between diaphragms due to the use of vertical stud members but the two way stiffness of the sheathing will cause some load to be transferred direct to the racking wall, however this will not alter the overall effect on that wall. The horizontal load, carried through the racking walls, is transferred into the lower horizontal diaphragm and may be redistributed as it passes into lower storey walls until ultimately it reaches the foundations. In order to resist the racking load the timber frames must deflect in the line of the load requiring the diaphragms and face walls to move with them. Such movements will be resisted by the stiff external brick cladding further reducing the proportion of the total horizontal load carried by the timber frame walls. This effect is similar to that of shielding by the brick wall, thus, because there is a great difference between the relative stiffness of the brick and timber frame walls, the shielding will be considerable.

In addition to help given by the face walls in shielding, which is highly dependent on stiff corners, brick flank walls also assist in resisting racking forces by taking load from the external timber frame walls through the shear
action of the flexible brick ties.

Additional resistance will be given to the building by internal fittings, such as the staircase, which, dependent on their position, will stiffen the horizontal diaphragm.

Whole house design is clearly very complex and would require knowledge of the strength of every element of the construction together with the effect of its interaction with other elements. Such information could only be obtained from comprehensive whole house and component tests which would be prohibitively expensive to cover the very wide range of design variables.

Some such tests have been carried out at the Building Research Establishment and at the South Bank Polytechnic but the work remains unpublished and the results have not been available to the author of this thesis. It is highly probable that the need for whole house tests could be reduced by using computer modelling techniques together with component data but it is clear that a design solution based on tracing the path of the load through the building is a future long term project.

4.3 ASSESSING BUILDING ADEQUACY THROUGH THE STRENGTH OF INDIVIDUAL WALLS

As a result of the difficulties associated with the total approach to buildings, the standard method for design has been to ignore all but the structurally sheathed timber frame walls acting in the direction of wind loading and to attempt to show that these walls alone are capable of resisting the applied horizontal forces. This method assumes all load to be transferred to the timber frame and then to be distributed by the face walls and horizontal diaphragms into the external racking walls and, ultimately, the foundations. This is a very conservative system and ignores the following effects:
(i) shielding by brick walls,

(ii) internal walls acting in the direction of load,

(iii) internal fittings such as staircases,

(iv) the effect of the face walls and diaphragms on the side walls - "the box effect".

However in not using the secondary elements to provide racking resistance the method allows the building to be designed independently of secondary internal walls and external claddings. This is advantageous because it allows a freedom of choice in the use of the external cladding, noting the brickwork to be an unnecessary structural element which is used solely for appearance on timber frame buildings, and it allows the plasterboard walls to be considered non-structural as befits the traditional use of plaster whilst also allowing freedom of internal layout.

Initially very simple empirical means were used to check racking resistance. The most common method was to assign a racking resistance per metre run for the combination of materials used in a wall which when multiplied by the total length of full height panel in a wall gave the wall strength. The TRADA values for racking resistance (TRADA, 1980 b) have been used simply and successfully for many years but can now be shown to greatly overestimate the strength of many walls particularly those with many openings. This has not presented a safety problem because of all the factors (noted above) omitted from consideration in the design. However test results included in this thesis indicate that the racking resistance of open plan timber frame buildings (such as village halls) without brick cladding are likely to be overestimated by the TRADA method and thus will have reduced factors of safety and could potentially be at risk.

In some buildings the required racking resistance could not be achieved by the external side walls alone
(typically in narrow fronted houses with many openings in the front and back walls when the wind blows on the side wall). The racking resistance could be achieved by one of three ways:

(i) providing additional restraint to the side walls and designing them as vertical cantilevers,

(ii) sheathing the inner face of the panels with plywood thereby increasing their racking resistance per metre run,

(iii) using an internal plasterboard wall to provide the additional racking resistance.

As test data became available for plasterboard the final alternative became popular with a number of designers but it is evident that the general safety factors are then eroded as the high basic racking resistance figures for the external walls already rely on the implicit use of the internal walls for safety.

The design method presented in this thesis, which has also been adopted for use in the draft Code of Practice BS 5268 Part 6 (BSI to be published), is also based on the assessment of individual wall resistances. However the method of determining wall resistance is very much more complex and covers all the principal variables in wall design. It provides realistic design figures for the wall which can be proven by full scale tests. The design method is suitable for use in determining the strength of any timber framed wall whether it is sheathed solely with a wood based material, or it is both sheathed and lined, as in external wall construction, or it is lined on both sides with plasterboard, as in the case of internal walls. Secondary information can be used to determine the additional resistance contributed by a brick wall linked to the external racking wall by brick ties. With all this information it is then possible to give an accurate assessment of the
practical racking resistance of the building although the box effect and the shielding by the brick cladding cannot be included.

The accurate assessment of the strength of the walls has meant that structures shown to be adequate by previous design methods cannot now be proven using the structural sheathings alone. It is therefore necessary either to include the use of plasterboard lined walls or determine the shielding effect presented by the external cladding or evaluate the box effect.

The arguments for and against the use of plasterboard in considering the racking resistance of a building are extremely contentious and emotive. Information relevant to the argument includes the following.

(i) The use of TRADA or other early design values imply the contribution of plasterboard because the walls often cannot achieve their stated performance levels. This can be proven by test.

(ii) Plasterboard is a stiff material with a moderately high racking resistance. The internal walls it is used to clad have relatively few openings and thus are strongly proportioned.

(iii) The transfer of load through a semi-rigid horizontal diaphragm means that internal walls are likely to carry a very high proportion of the applied horizontal load unless the external walls are very much stiffer, which from (ii) above is clearly not the case. Tests in Australia have proven this point when, with the wind force acting on the face of a wide fronted house with internal cross walls
in the line of the load, very little racking strength was lost when the external walls had their structural sheathing effectively removed.

(iv) Plasterboard has often been considered non structural in wall use and may be treated as such by the fixers, however, plasterboard is used structurally in ceilings particularly at roof level where it is the sole means of providing diaphragm action.

(v) Plasterboard is more susceptible to damage than other sheathings to timber frame walls both due to its position and its material properties, however it is unlikely that all the plasterboard would be damaged at the same time unless the building itself was also considered to have failed.

(vi) Timber framed houses are, at present, under close scrutiny by the public through the actions of the press. A sudden change to full acceptance of plasterboard might therefore be ill advised.

The present draft of the Code of Practice (BSI to be published) uses point (v) to determine an allowance for the use of plasterboard. It is common to require damaged structures to sustain 60% of the maximum applied load. If this could be carried by the structural sheathing the remaining 40% could then be attributed to plasterboard. The plasterboard value was effectively reduced to be 50% of the resistance provided by the structural sheathing and special clauses were included to cover the use of special separating walls. An alternative logical approach would be to determine the total resistance of the plasterboard walls and to apply a secondary safety factor to their use to cover the increased potential for damage.
Whichever method is finally adopted for the design of plasterboard lined walls it will be necessary to know accurately the racking resistance of all the walls in a building and possible the racking resistance of all the walls when the structural sheathing alone is considered. The design method proposed in this thesis is dedicated to this principle and defines accuracy to mean values that can safely be reproduced by full scale tests.

4.4 WALL DESIGN

The strength of walls has been used to assess building performance because walls are elements that can be analysed and tested without undue difficulty. However, very few researchers have had the facilities to test full length walls, and their variability in terms of length and size and position of openings has detracted from such work. Thus the common approach to wall design has been to test a panel representative of the plain wall construction from which a racking resistance per metre run can be determined and to apply this to an easily assessable wall parameter to determine the wall strength. The TRADA method mentioned in the previous section is typical; here the wall parameter is the total length of full height panel. Tests could not be conducted on full length walls to check the design method because the test rigs had been specially constructed for the small panel tests, instead elements of the wall design were investigated; using short panels including either windows or doors.

The current investigation has retained a small plain panel test considering it to be the best method of assessing variables in material and general construction, such as stud spacing. The extrapolation of the results to predict wall design has however been analysed in much greater detail than previously had been possible using major test programmes carried out on 4.8m long walls to determine the influence of parameters such as:
(i) length,

(ii) size and position of openings,

(iii) vertical load.

Secondary factors affecting wall performance such as:

(i) length of panels making up the walls,

(ii) continuity between panels, i.e. the vertical connection and the top plate,

(iii) base connection,

(iv) special forms of construction,

have had to be covered in different ways, either:

(i) they have had to be ignored, basing the design on a minimum standard condition which will allow a conservative estimate of performance and which is fairly typical of practice or,

(ii) the construction has been interpreted in such a way that it can be incorporated into one of the main design parameters.

The work on the wall factors has had to be confined to the principal forms of construction for timber frame walls i.e. nailed plywood or mediumboard on vertical studs at 600mm centres. It is assumed that design factors appropriate to these walls can be used with all the other forms of construction covered in the plain panel tests. It is also accepted that the 4.8m restriction on wall panel tests does not cover full length walls and an extrapolation of test data is necessary. In both cases these extensions to
the test information can be more safely interpreted because the trends have been plotted over a much wider range of the variable than has previously been possible. However the computer based analysis described in Chapter 7 has been set up in order to establish more accurately the extrapolated results using the full range of available test data without the need for further expensive and time consuming tests which would be rather limited in their contribution to available information.

4.5 THE STANDARD PANEL TEST

In the past, tests on plain panels, between 2.4 and 3.6m in length, have been used in one of three ways:

(i) to show acceptable standards by comparison with a datum sheathing commonly used in timber frame construction,

(ii) to quantify the relative performance of sheathings,

(iii) to produce design data for wall racking strength calculations.

The test becomes progressively more complex in form as greater emphasis is put on design data and the number of variables covered by the panel test increases. The term standard panel test is applicable if the test allows the primary material variables to be evaluated. The ASTM E-72 test (ASTM, 1980) was the first standard panel test; it was designed to prove acceptable standards but was later used to provide design data and is thought to be the source of the TRADA figures. The E-72 test measures one particular form of sheathing behaviour; by restricting uplift of the panel it implies a very large vertical load which would normally be unattainable in practice. Work in Britain (and more recently the ASTM E-564 test) has departed from this concept and checks performance at practical levels of
vertical load, including allowances for holding down restraints which can be modelled by vertical load. A range of vertical loads is tested to cover the variation in situation in which the wall type may be used. The ASTM tests are very much failure oriented whereas the British tests have attempted to provide design data based on:

(i) the failure load, with an overall safety factor applied,

(ii) a deflection limitation.

The early British test work has been developed in this investigation to provide a standard panel test which, by defining the racking loads for a range of vertical loads for a single wall type of fixed length with no openings, can be used to determine the racking resistance for a particular combination of materials and form of construction. By reducing the vertical load data to a single design value the racking resistance can be used as the starting point for evaluating wall design by including modification factors for the wall parameters previously noted. The single design value is termed the basic racking resistance (BRR) and includes for all materials variables. The terminology is therefore inappropriate in comparison with sawn timber design but as it is now commonly used (BSI, draft to be published) it will be retained in this thesis.

The basic racking resistance includes a number of variables; these are:

(i) type of board,
(ii) thickness of board,
(iii) size and spacing of nails,
(iv) layout of sheathing and frame members,
(v) frame material.

In order to widen the scope of the design method and reduce future test requirements the majority of the parameters
could be covered by the use of modification factors. Test data from this investigation has enabled factors to be determined for (ii) and (iii) above and, although not tested in detail, advice is given concerning (iv) and (v). Incorporating these modification factors, the starting point for design becomes a standard use for a given type of sheathing. It has been found to be more suitable to relate the materials parameters to a standard use of the board type governed by past experience of its use rather than to use a standard thickness, nail spacing etc. common to all types of board. Thus the modification factors for the materials parameters define percentage changes from the normal condition and can be limited for different boards by exclusion clauses based on the information available and/or the common usage of the board. The design value for a board type is termed, in this thesis, as the datum racking resistance (DRR) and defines the orientation and thickness of the board, the size and spacing of the nails and the framework details.

The standard panel test is used to determine both DRR and BRR values. It is from these test results alone that the modification factors for the materials parameters are evaluated. It is an acceptable starting point for determining the racking resistance of buildings because it can be performed quickly and economically to extend design information to cover new materials or new combinations and it is sufficiently flexible to allow interpretation of new forms of construction. The major problem with the standard panel test is that it does not analyse the parameters primarily affecting building performance and therefore the extrapolation of the results using data for different wall types should be treated with care. Increasing the variation in the standard test panel configuration from the normal reduces the reliability of the wall design (the variation can often be defined by the change in DRR value). It may prove necessary to restrict the use of standard panel test data to an upper value of BRR unless checks are made using wall length panel combinations with openings.
4.6 SMALL SCALE TESTS

Small scale tests can sometimes be substituted for standard panel tests to provide design information. Normally they depend on a correlation being established between the test parameter and the design value achieved in the full scale tests.

Griffiths (1978 b) showed that a very simple tensile failure test, using the desired sheathing, nails and frame material with a proportionately reduced nail spacing, could predict basic racking resistance values. The method was suitable for interpolation between known BRR values and could cover small variations in type of material. The method relied on an interaction line being established, based on a wide range of test cases where both full scale and small scale results were available.

Patton Mallory's tests (1983) were more complex, using miniature racking panels, and were intended as a direct replacement for the full scale ASTM test.

In principle the small scale tests could provide a direct entry point into the building design procedure. They would act as a substitute for standard panel testing but would need to be used in conjunction with a wider range of standard panel test data and, unless they could fully model the racking test (in which case they would need to be extremely complex), they could never totally replace the full scale test.

A further use of small scale tests is to provide data for a computer based analytical design method. Here information is required independently for each form of structural behaviour experienced when racking the timber frame wall, e.g.:
(i) the tensile and compressive moduli for the frame material,

(ii) the shear modulus of the sheathing,

(iii) the nail performance data for deflections parallel and perpendicular to the grain of the frame timber and also, possibly, parallel and perpendicular to the grain of the sheathing.

In practice a number of these parameters have such limited affect on the analysis as to preclude testing, however, these will only be known following a parameter study using the analysis. Many of the tests can be based on existing procedures and in some cases data will already be available. Little work has been carried out on the board types commonly used as sheathings or linings to timber frames. It has therefore been necessary within the current investigation to develop less sophisticated tests so that rough data could quickly be made available to test the acceptability of the method of analysis over a wide range of material variables. More accurate data can be substituted as available should the analysis prove satisfactory. Details of these particular tests are given in Chapter 7.

4.7 DESIGN BY COMPUTER BASED ANALYSIS

The proposed wall design method is empirical, based on test experience. It should substantially improve the accuracy in calculation of wall performance due to the quantity of test data on which it is based. It should prove to be better than the present simple design methods (such as used by TRADA) because it is more rigorous in its approach and the results can be confirmed by tests. But it will also require more work on the part of the designer. The method is suitable for incorporation in a simple computer based package which would require only the materials and wall details to be entered. Furthermore the design approach will
be more accurate than the majority of the theoretical analyses noted in Chapter 3 because it does not make assumptions or simplifications concerning the behaviour of the wall. Compared to the analytical methods it will also be easier to use.

The empirical design method is not a perfect solution and, itself, has to incorporate many simplifications. However, to achieve an appreciable improvement in both accuracy and applicability it will be necessary to use a comprehensive analytical approach such as a computer based finite element analysis. This would allow a very accurate model of the timber frame system to be set up which could identify all the modes of behaviour present in the wall deformation pattern.

The analysis could be used to evaluate the performance of individual walls or could be incorporated into a larger programme modelling complete buildings when the behaviour of horizontal diaphragms and the interaction of building elements will also have to be considered. Such a system will remain the ultimate objective for timber frame designers, but the initial wall design stage has significant value because it will allow checks to be made on the empirical design factors allowing their extension to cover cases for which tests have not been possible.

The basis of the analysis is to calculate the resistance of each structural element in the wall from its deformation due to its location within the overall structure. In this way the total internal resistance of the wall can be calculated and related to the external forces. By applying successive increments of load the wall racking test procedure can be modelled and the design loads established based on either deflection limitations or failure strength. The materials parameters and the method of analysis can be checked using the standard panel test data and then a more rigorous series of checks are applied by comparing the results of the computer design method, the empirical design
method and tests for the 4.8m wall units. Should there be good correlation between the analysis and the test data it will be possible to extrapolate the results using the computer programme enabling the empirical design factors to be evaluated for a more comprehensive coverage of wall designs.

The information required by the analysis is more complex than that of the empirical design method. The materials parameters detail the behaviour of a single structural element, thus board thickness is important as it describes the sheathing element but nail spacing now becomes a function of the whole structure, i.e. a wall design parameter. The materials factors too are more complex due to the need to assess individual element behaviour rather than considering the structural form as a single unit. The information will be taken from two sources, firstly standard properties such as moduli of elasticity and rigidity and cross sectional dimensions will be based on Codes of Practice and manufacturer's literature, and secondly nail fixing behaviour will be determined using small scale tests, as described in the previous section, covering not only the fixing itself but also the sheathing and the frame material. The wall design parameters will also be different with the programme requiring details of the location of every structural element thus the wall will be defined by the position and size of every frame, sheathing and fixing element rather than the overall length, height and location of openings proposed for the empirical design approach. Applied loads will be treated as before but the computer based analysis will be better able to include secondary restraints such as return wall fixing and holding down straps.

4.8 SUMMARY

The principal design approach adopted in this investigation is an empirical method to assess the racking resistance of individual full length walls. The design data is provided by tests on standard panels, the results of
which are then multiplied by modification factors, which have been determined in specialised test programmes, to predict the performance of walls. The modification factors will be divided into material and wall design factors which effectively break the design into three stages:

(i) the datum racking resistance; the resistance for a single specific use of the sheathing material suitable for incorporation into a standard design document, in this case the Code of Practice,

(ii) the basic racking resistance; the resistance of the sheathing material based on its particular use in the wall unit,

(iii) the wall design load.

A proposal for the empirical design procedure is shown in Figure 4.1. It notes different entry points to the design method based on small scale tests, standard panel tests and full length wall tests which by pass parts of the design procedure and thereby exclude the use of some of the modification factors. The method is similar to that presented in the draft Code of Practice (BSI to be published ) but includes additional factors covered only in the current investigation. A number of secondary factors examining special conditions of wall use are noted together with proposals for their incorporation in the design method. The test results and the evaluation of the individual design variables are detailed in Chapter 6. The factors are then assembled and checked with the results of the computer analysis in Chapter 8. The design of wall units is covered in detail and thought is given to the development of the work to cover whole house design. This will include suggestions for dealing with the following:

(i) the contribution of plasterboard,
(ii) the box effect of buildings,
(iii) the shielding effect of the brick wall.
A simplified design procedure is shown in Figure 4.2. This represents a more conservative approach that could adequately be applied to the vast number of houses that will present no racking resistance problems. Its advantage is that it can be used by a less specialised designer and would make the design of timber frame houses more comparable with the "deemed to satisfy" regulations appropriate to its competitors. The procedure reduces the number of modification factors and would simplify them in form. These changes make the method less efficient, thus where it could not be used to prove the adequacy of a structure it would be necessary to resort to the main design method.

A proposal for a computer based finite element analysis design procedure is shown in Figure 4.3. This relates to the details given in Section 4.7. However, the work covered by this investigation, detailed in Chapter 7 is restricted to the development of the programme to prove its viability for timber frame wall design.

Table 4.1 details the notation used for the principal design parameters encountered in the investigation. Table 4.2 lists the modification factors required initially for the empirical design methods; those for the standard design method start at K100 and those for the simplified method start at K200 to avoid confusion with BS5268 part 2.
Entry Point 1
Datum Racking Resistance Design
Values from Code Defines sheathing, thickness, frame & fixings

Entry Point 2
Standard Panel Test Materials as used in wall
Research Tests For Code Use

Entry Point 3
Full Scale Test On Wall As Used In Practice

Alternative Design Method Using Performance At a Tested Vertical Load equivalent to BRR x vert.load mod factor. Value cannot be modified to give other vertical load cases.

Commercial Testing

Basic Racking Resistance kN/m

Wall Design Modification Factors
1. Vertical Load
2. Length
3. Openings
4. Height

Special Conditions of Use of Wall
1. Holding down method return wall effect (iii)
2. Combined brick wall (iv)
3. Wet Panels
4. Horizontal Loads other than wind

Alterations to design equation

Wall Design
BRR x length x Mod.Factors

Wall Racking Load kN

Notes
(i) Not included in the Code of Practice.
(ii) Vertical load could affect modification factors for length and openings.
(iii) The practical holding down of the panel and return wall effect are accommodated as alterations to applied vertical load.
(iv) The brick wall effect is covered separately. See Chapter 8.
(v) Some use may be required of wall end modification factors and special conditions alterations depending on the complexity of the wall test.
(vi) Conditions alterations depending on the complexity of the wall test.

Figure 4.1 Procedure For Empirical Wall Design
Datum Racking Resistance defining minimum standards of board thickness and quality and stud size and quality

Materials Modification Factors
(i) Nail size
(ii) Nail spacing

Basic Racking Resistance kN/m

Wall Design Modification Factors
(i) Vertical Load
(ii) Method of Holding Down
(iii) Length
(iv) Openings
(v) Plasterboard Lining
(vi) Brick Outer Skin

Wall Design
BRR x Length x Mod. Factors

Wall Racking Load kN

Figure 4.2 The Simplified Empirical Design Procedure
Materials Parameters, Standard design figures M of E, G, etc.

Details of Structure, Defines size and location of every member and joint

Small Scale Tests, Results define nail performance, etc.

Non Linear Finite Element Analysis Computer Programme

Results

(i) Standard Panel

(ii) Full Length Walls

Results

Use

Check with standard panel tests
Results may be used to modify parameters in programme.

Check with wall tests.

Check with design wall racking loads.

Predict wall racking loads which are out of the range of tested modification factors.

Whole House Model, Links wall action to interaction with rest of structure

Checks with whole house tests.

Figure 4.3 Finite Element Analysis Design Method
Wall Design

(i) Racking Loads

WRL = Wall racking load
RL = Panel racking load
TRL = Test racking load for wall
FRL = Racking load at a floor level, usually the summation of the wall loads at a given level

The following subscripts are used with WRL and may also apply to RL and TRL

SS = Structurally sheathed walls
SP = Separating walls
PL = Plasterboard linings
BW = Brickwork or brick walls
primary board
secondary board
additional effect of secondary board
combination design
test

(ii) Racking Resistances (loads per metre length of wall)

BRR = Basic racking resistance
MRR = Median racking resistance
DRR = Datum racking resistance
TRR = Test racking resistance

The subscripts noted in (i) above may also be applied.

(iii) Loads

V = Stud load
F = Uniformly distributed load
F_p = Concentrated load

(iv) Dimensions

L = Wall length (which may be subscripted for additional detail)
h = Wall height
L = Part lengths of wall
a = Dimension from load to trailing edge of wall

(v) Modification Factors

K = Modification factor
and will be subscripted by a number, as defined in Table 4.2 or by:-

L = Length
VL = Vertical load
O = Opening

Building Design

(vi) Wind Forces - V_x and V_y

Where subscripts x and y denote direction of force.

(vii) Applied Wall Forces - R_F, R_B, R_L and R_R

Where subscripts are:

F = front wall
B = back wall
L = left wall
R = right wall

The forces may be further divided to ground floor, first floor and roof diaphragm loads using subscripts G, F and R which precede those noted above i.e. R_GF, R_FF, R_RF where:

G = ground floor
F = first floor
R = roof

(viii) Wall Racking Loads - WRL_F, WRL_B, WRL_L and WRL_R

using the subscripts noted in (vii) above.

(ix) Moments (due to unbalanced forces) - M_x and M_y

Dimensions

L = face length
B = side length
h = height
A = area

dimensions may be used with the subscripts noted above.

Note

The list of terms and subscripts is not comprehensive, other terms will be described as and when used.

The double usage of some characters should not prove confusing in practice due to the conditions to which they are appropriate.

Table 4.1 Notation Used For Design Parameters
<table>
<thead>
<tr>
<th>Modification Factor</th>
<th>Draft Code</th>
<th>Numerical Designation of Modification Factor</th>
<th>Standard Method</th>
<th>Simplified Method</th>
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<td>Materials Parameters</td>
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<td></td>
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<td>Nail size</td>
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<td>K202</td>
<td></td>
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<tr>
<td>Nail spacing</td>
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<td>Board thickness</td>
<td>(4)</td>
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<td>-</td>
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<td>Board orientation</td>
<td>(5)</td>
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<td>Wall Design Parameters</td>
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<td>Length</td>
<td>(8)</td>
<td>K112</td>
<td>K212</td>
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<tr>
<td>Openings</td>
<td>(9)</td>
<td>K113</td>
<td>K209</td>
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<tr>
<td>Height</td>
<td>(10)</td>
<td>K200</td>
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<td>Holding down</td>
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<td>External brick skin</td>
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<td>Wet panels</td>
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<tr>
<td>Load duration</td>
<td>(13)</td>
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<td>K114</td>
<td></td>
</tr>
</tbody>
</table>

(i) Variation factors included, but not termed as Modification Factors Required in the Empirical Design Methods.
CHAPTER 5
THE EXPERIMENTAL RACKING TEST

5.1 TEST PROCEDURE
5.1.1 Introduction

In 1976 when timber frame wall tests were first carried out by the author at Surrey, a decision had to be taken concerning the method of test to be adopted in the programme. Four options were available.


(iii) The Princes Risborough Laboratories test (PRL test) specifically developed to measure racking resistance.

(iv) To develop an independent method of test.

At the time the requirements for the test had not been fully defined but the research on racking test methods indicated the following points to be essential.

(i) The procedure should be developed for the standard panel test to provide the initial design data for testing wall racking resistance. It should also be adaptable to work on wall units to allow the determination of design factors.

(ii) The procedure should concentrate on the initial stiffness of the panels which was considered likely to be the governing factor in the majority of design cases.
failure performance, however, remained an important part of the test.

(iii) As design was likely to cover a variety of load conditions more than one test was likely to be required to assess a construction type. This could result in the need for a more complex method of reducing the results to provide design values.

(iv) The results should clearly indicate any deficiency in the form of construction tested.

(v) The test should be capable of showing consistent results for similar panels and the procedure should not favour any particular panel construction noting that there is a wide range of performance to be covered by the test.

(vi) The test should be kept as simple as possible but, in view of the size and cost of test specimens, as much information as possible should be obtained from each wall.

(vii) Tests should reflect the fact that racking forces are caused by short term loads; thus there should be no requirement from a 24 hour test.

(viii) The test should be carried out independent of proprietary fittings used by different builders in the assembly of the walls but should be capable of assessing the effect of these fittings on the performance of wall units.

Considering the test options it was noted that the standard test procedure was primarily intended to prove the adequacy of a structure under a given set of conditions. It has been
adapted for timber frame wall tests (Reardon, 1980) but is inherently unsuited for use in determining design values for a construction form over a wide range of uses. The ASTM test had already been found unsuitable by researchers in Britain (FPRL, 1971 a and 1971 b) because it did not cover practical vertical load conditions and was unsuitable for producing design values because it had been developed as a comparison test. The development of a new test was rejected in favour of the PRL test which had been based on many of the requirements noted above only a few years earlier. The other advantages of the PRL test were that:

(i) it was actively being promoted as a design test for timber frame walls in Britain,

(ii) results were available for construction forms relevant to the current investigation and the detail of the data was such that any weaknesses in the procedure could readily be assessed,

(iii) it was in the development stage, which would enable justifiable changes to be quickly implemented.

A decision was taken to adopt the PRL test procedure and to develop it, as a result of first hand experience, so that it could be used both for the standard panel tests and the research programme to determine wall design factors.

5.1.2 The Original Princes Risborough Laboratories Test

In the early Princes Risborough reports on racking tests,(FPRL, 1971 a and b), standard eight foot square panels were tested with up to six variations in vertical load. The following procedures were observed.
a) For panels subjected to racking loads only, i.e. zero top load, a new panel was used for each test. A preload of 225N was applied independently by a pulley system and maintained throughout the test. The deflection data was set and the racking load was applied in six increments to a total deflection of 7.5mm, the load was then removed and the residual deflection noted. Finally, the panel was loaded to failure.

b) For panels subjected to both racking and vertical load, stiffness tests to 7.5mm deflection were carried out on one panel under a number of vertical loads. The 225N racking preload was applied and retained through the tests. A 22.5kN vertical seating load was applied and removed and the datum deflection readings taken. A racking load of 1.35kN was applied followed by the required vertical load which was maintained throughout the test. The racking load was raised in increments until a total racking deflection of 7.5mm was reached. The racking load and vertical loads were removed and the residual deflection measured still with the preload applied. The second test commenced with the application of the vertical seating load and the re-zeroing of the gauges. Finally, after all the stiffness tests had been completed, the panel was racked to failure under one of the vertical loads.

In both cases the design values for racking load were taken direct from the test results using the lower of:

1. the load causing a 7.5mm deflection in the stiffness test,
2. the factored down failure load from the strength test.

The tests clearly showed the importance of stiffness because for each panel all the stiffness tests were performed although failure could be tested at only one condition of vertical load. Thus, often, the stiffness test load could be based on several results whereas the strength test load represented a single test.
The major difficulty experienced with the procedure was that each test had a different datum. The high deflection limit enhanced the problem resulting in large residual deflections causing the change in datum. Figure 5.1 shows the effect of increasing the racking load if the datum is reset after each load cycle for a single vertical load condition. A similar result is likely if the vertical load is changed between cycles as the zero reading point will be ahead of the true datum and the racking load will therefore be higher than if the panel was tested initially at that vertical load.

At the time of these early tests there was no agreed deflection limit for timber frame wall tests. Initially 7.5mm had been suggested but 0.002 times panel height (approximately 5.0mm) was suggested based on Scandinavian reports which indicated damage to wall fittings at higher levels (e.g. windows sticking, cracks in wall paper etc.). The limit was later increased to 0.003 times the panel height for the following reasons.

(i) It had become evident that the damage noted above was due to more prolonged load applications than the gust loading simulated by the test.

(ii) The results at 5mm deflection were low compared with those already used by designers which had not presented practical problems.

(iii) It was recognised that the increased stiffness of the timber frame house shell would mean that in use the wall panels would never reach the full deflection limit.

The uncertainty concerning the practical deflection limit suggested that a separate test limit based on the test parameters and procedure should be adopted. A limit of 0.002 x panel height was chosen to reduce the problem of
the non recoverable deflection (the panel set) measured between load cycles. The change in deflection limit reduced the damage done to the panel by each load cycle and thereby enabled cycling of racking load under a constant vertical load to be incorporated in the procedure. This allowed any secondary weakness in the sheathing such as poor elasticity to be noted and is of particular value in view of the cyclic nature of the practical wind loading condition. Panel weaknesses are measured either by the drop in racking load during cycling, or by the panel sets between load cycles.

Four load cycles were chosen for the standard test using a fixed datum, by which time a stable cycle had been reached or was close to being reached. The lowest maximum load achieved during these four cycles was then used in design. The original single cycle test represented a once in fifty year wind return load, thus the four cycle test could be analysed as a once in twelve-and-a-half year wind return load. The Code of Practice on wind loading (CP3, Chapter V, Part 2) noted this level of loading to be 80% of the maximum design value. Reference to previous tests showed that on average, the load at 5mm deflection was between 75 and 80% that at 0.003 times the panel height and thus the test limit and the practical design limit were safely compatible.

These changes in the test procedure resulted from the early test work carried out at Surrey (Griffiths, 1976) and soon after were incorporated into the draft for the timber frame wall Code of Practice. They improved the consistancy of the results but, as anticipated, reduced the stiffness results at higher vertical loads. Both cyclic loading and the method of application of vertical load presented problems in achieving a stable datum for each panel. The 225N preload had been eliminated from all Surrey tests to simplify the procedure but, in concept, needed to be replaced. The second series of improvements concentrated on establishing a stable datum, to improve the uniformity of results, and on fixing other secondary details. The following points were considered.
(i) The provision of a vertical settling load to compress all joints in the panel once it had been fixed in the test rig and before testing commenced.

(ii) The inclusion of a racking preload procedure for each vertical load condition in order to take up any slack in the panel caused during the application of vertical load in the event that there should be a small component of the load acting in the opposite direction to the racking load. The preload, fixed by a deflection limit, represented only a small percentage of the design racking load, and was determined so as not to cause any significant set in a normal panel.

(iii) The method of application of vertical load and its removal, if necessary, during the period of cyclic testing. Here greater simplicity in the test procedure was the aim.

(iv) The rate of loading.

(v) The recovery period between load cycles.

The procedure was detailed with relation to a standard panel test programme which normally required at least two panels to be fully tested in order to provide failure values for the extreme conditions of vertical load.

5.1.3 The Procedure for a Standard Test Programme

As a result of the preparatory tests the following procedure was adopted for all standard panel tests at Surrey.
a) The first panel is assembled in the test rig and care is taken to ensure that the base plate is level, the holding down bolts (if used) are tightened and any vertical connections are properly made. (The final point assumes that the standard panel can be made up from smaller modules joined together).

b) The vertical settling load of 0.75kN/stud is applied briefly to the panel for five minutes, released and the panel allowed five minutes to recover in order to minimise the effect of any distortions in the panel caused by manufacture or transportation. During this time the deflection gauges may be fitted in position.

c) The zero vertical load stiffness test is conducted. First a preload test is carried out racking the panel to a 1.25mm deflection and then allowing it to recover. The deflection gauges are then zeroed and the panel cycled four times to a deflection of 5mm, measured from the initial datum, at a rate not greater than 1.0mm per minute. Ten minutes are allowed between each cycle with the panel set, the non recoverable deflection recorded at the end of the period. A set greater than 1.5mm after the first cycle is indicative of a problem in testing or a particularly weak panel. During the load cycling, the racking deflection at the top of the panel is continuously monitored and a set of readings from all gauges is recorded for every millimetre of deflection.

d) The 2½kN/stud test is started ten minutes after the final zero cycle racking load has been removed. The vertical load is applied and held constant throughout the test. The racking preload is carried out and the deflection gauges re-zeroed. One cycle of load to 5mm deflection is conducted. Care is taken to ensure the vertical load does not vary during the test due to vertical movement in the panel. The panel set is recorded ten minutes after the racking load has been removed.
e) Once the panel set for the 2.5kN/stud test has been recorded the vertical load is increased to 5kN/stud, the racking preload is carried out and the deflection gauges re-zeroed. Four cycles of load are conducted to the 5mm deflection limit but in this case the initial panel set should not exceed 1.0mm. The set is not required after the last cycle and the top load can be removed immediately. Throughout the stiffness tests the deflection rate is kept constant.

f) The failure test at zero vertical load is carried out. The deflection gauges are re-zeroed and the racking load continuously applied at a deflection rate of 2.5-3.0mm per minute. By monitoring the racking deflection full sets of data are recorded every 2.0mm. Loading is stopped after the racking load can be seen to be dropping continuously or after a deflection of 75mm has been reached. The panel is checked and damage zones noted before releasing the load and removing the panel from the test frame.

g) The second panel is mounted in the frame and steps (a) to (e) repeated.

h) The failure test on the second panel is performed under a 5kN/stud vertical load. Although it would be acceptable to carry out the test directly the 5mm deflection is reached in the fourth load cycle of the 5kN/stud stiffness test, the normal procedure is adopted because it is necessary to change the gauges measuring the racking deflections to longer stroke versions. Step (f) is repeated re-zeroing the gauges after the vertical load has been applied. Great care is necessary in maintaining the vertical load constant.

Some parts of the above procedure were specific to the Surrey tests and were altered for inclusion in the Code of Practice. Alterations were made either to allow the procedure to be more general in its application or to reduce ambiguities which could be accepted in the Surrey tests as a result of the experience gained in using the test method. The changes included:

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(i) not specifying the vertical loads and requiring four load cycles for each condition,

(ii) defining the racking deflection to be the difference in movement between the top and the bottom of the panel and restricting this movement to 0.002 times the panel height in the stiffness test with the rider that slip at the base of the panel is limited to 0.5 millimetres,

(iii) defining the vertical settling load as either 1.5 kN/m or 0.75 kN/stud (if placed at approximately 600mm centres), thus the load is related to the panel length and not the spacing of the studs,

(iv) noting the racking load to be applied either continuously or in equal increments (based on deflection) with a minimum of four increments in each stiffness test cycle,

(v) rationalising the loading rates such that; for the stiffness test the maximum load should be reached not less than four minutes and not more than ten minutes after the start of the cycle and for the failure test the rate of loading should not increase the racking deflection at a rate exceeding 3.0mm per minute,

(vi) giving a more detailed assessment of panel failure conditions noting the likelihood of temporary reductions in loads as fixings break and the load is redistributed,
(vii) allowing the failure test to be carried out as a continuation of the fourth load cycle if there is no change in vertical load,

(viii) noting the need to monitor moisture content in the timber.

In the Surrey tests the panels were always vertically loaded at 600mm centres allowing a standard stud loading to be quoted. Assuming uplift forces to be resisted by holding down fastenings the extreme vertical load conditions for normal domestic use were assessed to be zero and 5kN/stud. One intermediate condition was tested at 2.5kN/stud and, to reduce the amount of work done on each panel and thus fatigue, the number of load cycles was reduced to one for this case noting that loss of load and panel sets could be interpolated from the other two vertical load cases. The racking deflection was determined solely by the movement at the top of the panel and was restricted to 5.0mm and not the correct 4.8mm. Additionally the point of measurement was 50mm below the top of the panel thereby underestimating the deflection by 2%. However the total overestimation of the racking deflection would have been negated by a sliding of 0.3mm. In practice sliding varied between 0.1 and 0.2mm and so the method of assessing deflection was well within standard requirement.

In its final form the test method has become known as the PRL/U of S test (BBA, 1983) recognising the work of Princes Risborough Laboratories in formulating the method of test and the work of the University of Surrey in developing it into a practical standard test procedure. The test is now solely recommended for the production of design data for British use and has also been adopted for use in other countries concerned with wall testing.
5.1.4 Variations for Investigative Test Programmes

The standard procedure defined in 5.1.3 has been applied to all the routine tests and, wherever possible, investigative programmes. In the latter case, however, it is not always possible to keep to the procedure in full due to the additional requirements of the research programme. It is then necessary to exercise judgement in the relevance of the data when reducing the results and making comparisons. Fortunately research programmes include a greater number of tests and it is normally possible to balance the loss in procedural accuracy with the greater volume of information needed to indicate trends in performance. It is not necessary at this stage to identify every departure from the standard procedure, but the more significant changes are noted below. Other variations which may have affected the results are included with the test results in Chapter 6.

A minor departure in test procedure will occur when it is necessary to achieve a full range of information from one panel, i.e. obtain two sets of racking strength data. Normally this is only done in cases where panel stiffness is expected to govern design and the strength tests are purely to confirm that an adequate factor of safety can be achieved. In this type of test the zero vertical load case is carried out first as less damage is done to the panel due to the localised brittle nature of the failure. The minimum strength load required is calculated based on the stiffness results and the safety factors applicable. The panel is then loaded to that value, unloaded and checked for damage. Normally the leading two to three nails in the bottom rail would be replaced before finally testing the panel to failure under 5kN/stud. Inability to achieve a satisfactory level of safety in the second test does not prove inadequacy in the panel and the test would then need to be repeated independently on a second panel. However, achievement of an adequate factor of safety means that design can proceed based on the stiffness results.
Major departures from procedure are necessary in panel combination tests where one panel may be reused in a number of stiffness test combinations and then finally in more than one failure configuration. A number of such programmes are included in this investigation, each following a slightly different test procedure. The two main approaches are either:

(i) to carry out all zero vertical load stiffness tests followed by all 5kN/stud stiffness tests before failing selected combinations, or

(ii) to conduct all stiffness tests on a panel or combination before proceeding with the next wall; failure tests are again left to last.

Both methods have proved successful, but the second is quicker as it requires fewer changes of panel in the test rig. In these tests the four load cycles are only necessary the first time a panel is tested at a particular vertical load, when it will normally be tested on its own. In later tests, a single load cycle is adequate. This has been confirmed by the last programme when two load cycles were included and, on average, the second cycle load was as high as the first. Failure tests on panel combinations were often conducted under two vertical loads using the method noted above; panels would be reused in further tests after checks and minor repairs had been made. This became necessary as a result of the variety of combinations that could be tested in stiffness using a relatively small number of panels.

Tests on wetted constructions also required a departure from normal procedure because the likely variation in the performance of panels meant that an accurate comparison of dry and wetted results could not be made unless tested on the
same panel. Thus a panel which was to be tested in either the wet state (immediately on removal from a water bath) or the dry after wetting state (after the panel timber had dried to its original moisture content following immersion in the water bath) had first to be tested dry. To avoid complications the tests were carried out under a 2.5kN/stud vertical load only. All tests started with a single stiffness cycle in the dry condition. After immersion, and drying where necessary, a standard four cycle stiffness test followed by a failure test was conducted. The effect of the wetting could be found by comparing the first cycle stiffness results and by analysing the factor of safety achieved by the panel.

5.2 REDUCTION OF TEST RESULTS

5.2.1 Introduction

The reduction of results for structural acceptance is relatively simple in most test methods. For instance, in the standard timber test, deflection limits are quoted for design loads and requirements for factors of safety given, related to the lowest failure load and the number of similar tests performed.

The details of the method of interpretation of the ASTM E-72 results are not clear. They state that graphs are to be drawn of the 3.5kN, 7.0kN, 10.5kN and failure load cycles and the sets between each test noted as a percentage of the total deflection of the immediately preceding test, but no advice is given on how to relate these values to design loads. Sherwood (1980) described requirements by statutory bodies with regard to the racking test. The first case was the Housing and Urban Development Minimum Property Standard HUD-MPS which was based on an earlier version of the test. Considering dry panel testing only, the requirements for an 8ft panel were:

(i) maximum load 5200 lb - no failure,
(ii) 1200 lb load - total deflection less than 0.2 inches
    - maximum set less than 0.1 inches

(iii) 2400 lb load - total deflection less than 0.6 inches
    - maximum set less than 0.3 inches

Combinations of materials that satisfied these requirements were then structurally acceptable for use in timber frame construction. Wall design was unnecessary as stability was covered by "deemed to satisfy" rules similar in practice to those for brick and block housing in Britain. The guide criteria for "Operation Breakthrough" was that for an applied load representing either 0.9 times the total of dead load plus wind load, or the dead load plus gravity live load plus 0.8 times the wind load then the deflection should not exceed 0.002 times the wall height. This method, by including vertical loads, must refer to the ASTM E-564 test although this is not clear from the reports.

The E-564 test itself (ASTM, 1976) however, is better defined. Using the averaged test results, the ultimate shear strength is defined as the maximum load divided by the length of panel. The shear stiffness is worked out, at a suggested reference load one-third the maximum value to be:

\[ G' = \frac{RL \times h}{\Delta \times b} \]  

where \( G' \) is the shear stiffness,
\( RL \) is the racking load,
\( \Delta \) is the deflection caused by RL
\( h \) is the panel height, and
\( b \) is the panel or wall length.

Thus, the shear stiffness is a secant value. However apart from the "Operation Breakthrough" guidelines, no requirements are given for either shear stiffness or failure load.

The racking test used in the current investigation and adopted for use in the Code of Practice BS 5268 part 6
(BSI, to be published) has been determined primarily to provide design values (see section 5.1). It is more complicated than other test procedures involving different load conditions, multiple panel investigations and separate design and deflection limits. The reduction of the results is therefore relatively complex and fundamental to the test procedure, as described below.

5.2.2 The PRL/UoFS Test

The original PRL test procedure presented few problems, the design load was found directly from either the stiffness test, which used the maximum allowable deflection, or by applying a factor of safety to the failure load.

The present method of reduction was derived at the same time as the cyclic test and the reduced stiffness deflection limit were introduced. The method requires the determination, at a particular vertical load, of both the racking stiffness load, which is calculated from 5mm deflection test results, and the racking strength load, which is based on failure performances. The racking design load is then calculated using the stiffness and strength values and the overall factor of safety which is dependent on the type of sheathing material. Where strength and stiffness values cannot be directly compared, either one or both values may be linearly interpolated from measured values, as shown in Figure 5.2; however, extrapolation is not allowed. The design values must be quoted with the vertical loads to which they apply. They cover, therefore, not only the construction of the panel, but also its use in a structure. Design values from a standard test may be used with modification factors for length and openings to cover wall design, although it should be noted that both factors may also be dependent on the vertical load.

Tests on standard panels had shown a reliable relationship between racking performance and vertical load for the normal range of sheathing materials (see Chapter 6).
Thus it was possible to consider a single racking performance figure with a common set of vertical load modification factors without too great a loss in performance. This result became known as the "basic racking resistance" (BRR). It is now quoted for the performance of the sheathing under a zero vertical load, but its value is determined so that in combination with the vertical load modification factors the tested standard panel design loads are not exceeded. It is important that the tests cover the maximum range of vertical loads under which the sheathing will be used and that for safety the design cases do not exceed these limits. Furthermore, there is no short cut to determining the basic racking resistance because, in general, it is difficult to predict the stiffness and strength behaviour of a sheathing/fixing combination. To explain this, two cases are considered representing either end of the spectrum of results. Firstly plasterboard, which being brittle has a low failure performance at zero vertical load which governs not only the design value but also the basic racking resistance. Secondly bitumen impregnated insulation board, which is a relatively weak sheathing and shows less improvement with vertical load than other stronger boards. Here, the basic racking resistance will be governed by the design value for the maximum vertical load. See Figure 5.3 a and b.

The basic racking resistance is very important to the simplicity of the wall design process, reducing the behaviour of the type of construction (i.e. the sheathing/fixing/frame combination) to a single value. It is the only design value quoted in the Code of Practice, and therefore the objective of most standard tests will be its determination. The cost of achieving simplicity, however, is loss of efficiency, particularly if the sheathing is either brittle or weak. The designer with access to test data could reduce the inefficiency by working directly from the results but this will present complications in wall design if the vertical load factor is not totally independent of other modification factors (see Chapter 6).

Two further points are notable concerning basic racking resistance. Firstly, the values contained in the
Code have been based on maximum test loads of 5kN/stud (equivalent to 10.4kN/m run), thus there is no increase in vertical load modification factor above this value. Secondly, it was only in March, 1986 that the basic racking resistance (BRR) was altered to refer to the zero vertical load case rather than that at 2½kN/stud (5.2kN/m). The reasons for the change were to make the vertical load modification factor greater than unity for all loads, achieving safety if omitted in error, and because zero vertical load is the only case when stud loads and uniformly distributed loads are the same regardless of panel length. Robertson and Griffiths (1981) had originally proposed the 2½kN/stud load as datum as they wished to have a mid-range practical datum point for all modification factors. However the safety limitations to this approach were soon noted and the change was made when the design method was adopted for use in the Code of Practice. The test results included in Chapter 6 were all reduced using the earlier definition of basic racking resistance. To reduce confusion, the name given to the 2½kN/stud performance level has been changed to the median racking resistance (MRR). The change in definition of basic racking resistance affects only the factors used in its calculation and the vertical load factors. Overall performance does not change and there is a direct relationship between the old and the new values, viz:

\[ \text{BRR} = 0.7 \times \text{MRR} \] (5.2)

The abbreviated version of the results used in Chapter 6 has been updated to show the present version of basic racking resistance but the data used in the test reports listed in Appendix B are all based on the old definition.

5.2.3 The Reduction Procedure for a Standard Panel Test Programme

The procedure for the reduction of results used to provide the standard panel test data given in Chapter 6 is detailed below with reference to Figure 5.4.
a) The load versus deflection results of the stiffness tests and failure tests for all the similar panels are plotted and are tabulated together with the significant readings of the other gauges.

b) The stiffness test load is determined for each condition of vertical load. To achieve this, the lowest racking load of the four cycles is found for each similar test and the average calculated. This is multiplied by the appropriate factor $K_{300}$, from Table 5.1 to take account of the number of similar panels tested, and by 1.25 to predict the once in fifty year performance figure. The $K_{300}$ factor is the standard modification factor for similar tests and is based on the $K_{73}$ factor used in BS 5268 part 2 (BSI, 1984). The 1.25 factor is the statistical factor linking the once in fifty year return wind load to the once in twelve and half year load where the four racking cycles represent the latter case. The value has been based on factors given in CP3 ChV part 2 (BSI, 1972).

At 2.5kN/stud, when only one racking cycle had been performed, the worst cycle load is estimated by calculating the loss in performance during cycling at zero and 5kN/stud, taking the average and subtracting it from the 2.5kN/stud test load.

c) Checks are made on the panel sets measured at the end of the first and third load cycles which should satisfy the following conditions:

(i) the set at the end of the first cycle should not exceed 1.2mm,

(ii) the set at the end of the third cycle should not exceed 2.4mm,

(iii) the magnitude of the increase in set due to each load cycle should reduce for successive cycles unless the increase is not greater than 0.12mm, when it may be ignored.
In tests where panels fail to comply with requirements (i) or (ii), the measured minimum racking load must be reduced by the factor $K_{301}$ where

$$K_{301} = \frac{\text{Maximum permissible set}}{\text{Measured set}}$$

The reduced value should be used when averaging the results for similar panels to calculate the stiffness test load.

The panel set restrictions were based on experimental experience and are unlikely to affect many panel tests when the correct racking preload procedure has been used.

d) The strength test load is calculated using the minimum racking load to cause failure in similar panels tested under the same vertical load. The minimum load is reduced by the appropriate $K_{300}$ factor (Table 5.1), to take account of the number of similar tests, and is divided by an overall factor of safety of 1.6. The required safety factor is reduced from the 2.0 value traditionally used (BS 5268 part 2) because the racking load is considered to be caused by wind effects.

e) The design racking load for a particular vertical load is deemed to be the lesser of the stiffness test load and the strength test load. Where necessary, the stiffness and strength loads may be linearly interpolated to give values at intermediate vertical loads. Such values may then be used in the determination of design values (Figure 5.2).

f) The median racking resistance, expressed as a load per metre run of panel, for a combination of materials may be determined from the test racking design loads, assuming values are known at zero and 5kN/stud. The median racking resistance is the lowest value found when the test design loads are divided by the appropriate modification factor $K_{303}$ given in Table 5.2. The $K_{303}$ factors have been determined from
the vertical load results for all standard panel tests as discussed in Chapter 6.

The major differences between the reduction procedure in the tests and that now proposed in the Code of Practice are the following.

(1) Basic racking resistance refers to the performance at zero vertical load. The implications of this change have already been noted and it has been shown that the change has no affect on design values. The value of BRR is calculated using modification factor $K_{302}$ (Table 5.2) instead of $K_{303}$.

(ii) In nomenclature; test racking stiffness is the same as stiffness test and test racking design is the same as design racking. However, test racking strength and strength test loads are different; the latter includes the overall factor of safety which is omitted by the former. The reason for the difference is explained in (iii) below.

(iii) The overall factors of safety vary with the type of material, whereas the current tests have used a constant value of 1.6. The Code requires that for plasterboard or any unclassified boards, or for any enhancement in performance due to these boards, the factor of safety is increased to 2.4. The higher value relates either to the traditional use of plasterboard as a non-structural material and to worries over its susceptibility to damage by wetting or impact, or to uncertainties concerning the behaviour of unclassified boards.

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As modification factors are included in the Code to cover change in vertical load there is no advantage, and even a possible disadvantage (see Figure 5.3 c) in carrying out standard panel tests at other than zero and 5kN/stud (10.4kN/m). However, the tests carried out at 2½kN/stud in the research programme were essential because they enabled the values of all the vertical load modification factors (K_{110}, K_{302}, and K_{303}) necessary to the definition of basic racking resistance to be determined.

5.2.4 Presentation of Standard Test Results

A typical set of results for a standard test is shown in Tables 5.3a, b, c and d, referring to a two panel test on 9mm medium board with standard fixings and SPF studs. The significant results are tabulated in Tables 5.3a and b covering the first and lowest stiffness cycles (denoted worst stiffness cycle in the Tables) and the failure test. The stiffness results are plotted in terms of racking load versus racking deflection, shown by \Delta, in Figures 5.5a and b. Similarly the failure plots for both panels are shown in Figure 5.5c. The factors of safety quoted on the latter graph refer to the panel performance only and are not used in design. The ultimate load of a panel is divided by its first cycle 5mm deflection load at a similar vertical load. The factors indicate the behaviour of the panel: values greater than two indicate that stiffness will probably govern the design value.

The overall results are quoted in Table 5.3c and are averaged for the two panels. The 'source of data' information in Table 5.3d indicates the appropriate K_{300} factor for use in calculating the stiffness and strength test loads, e.g. 0.8 for the single failure tests and 0.87 for the pair of stiffness tests. The calculated stiffness and strength values are shown in Table 5.3c and the lower value carried forward as the design load. In this case the panel strength governs at all vertical loads as predicted by the factors of safety recorded in Figure 5.5c. The racking performance
Table (Table 5.3 c) covers the information required by the designer. Firstly the design load and resistance (the load divided by the panel length) are quoted for the test results. In the next column, the median racking resistance required for each result are noted, i.e. the design load divided by the appropriate factor $K_{303}$. The lowest value is then carried forward and multiplied by the $K_{303}$ factor to give firstly the theoretical design resistance and then the theoretical design load. Thus in this example the median racking resistance is 3.20kN/m and the basic racking resistance is 2.24kN/m. It is notable that because the performance of the mediumboard is governed by strength and the 2.2kN/stud value is therefore interpolated, the median or basic racking resistance has been limited as shown in Figure 5.3 c. If the 2.2kN/stud results were ignored, as would now be acceptable, the theoretical resistances in some cases will be improved. However, the difference will be small and furthermore it can be shown that the maximum possible loss is only 2.5%.

The differences between the test and theoretical values indicate the accuracy in modelling performance using the basic racking resistance and the vertical load modification factors. Mediumboard, typical of stronger sheathings, achieves a high degree of accuracy indicating that its vertical load response conforms with the theoretical equation. Plasterboard and BIIB efficiencies are much lower.

Table 5.3 d gives additional details of the test which are of use in assessing the performance of the panels and the reliability of the design values. The details included are the following:

a) The consistency of the stiffness results which should show a variation about the mean of less than 10% and this figure would normally be less than 5% for reliable sheathings.

b) The resistance to cyclic loading showing the loss in load between the first and the lowest cycles; this value rarely exceeds 10% and should decrease with vertical load for
strong boards. Boards which give values consistently greater than 10% are likely to be unsuitable for use as sheathings, however, the cyclic behaviour is closely linked to the panel set and in this case the sets would undoubtedly be very high and would further reduce the design performance.

c) The panel set which is the maximum set after three cycles. The equally important first cycle set is not included and it is the responsibility of the designer to check for any infringement of design rules and then to calculate the reduced stiffness test load for use in Table 5.3c. However, the omission is not serious as excessive panel sets are very rare.

d) The failure details which are a summary of the results plotted in Figure 5.5 c.

e) The performance at 90% of maximum load which, due to the variability of panel performance at failure, is a better level at which to compare the behaviour of different wall types. Here, similar tests show more consistent results.

The results of all the standard panel tests covered in Chapter 6 have been reduced in this manner. Although the results are then presented in abbreviated form due account has been taken of the secondary factor, noted above, which can indicate the reliability of the results.

The method of presentation is included here as a recommendation for test practice in comparison with the minimum requirements at present detailed in BS 5268 part 6. As noted below, this method of reduction has not been applied to the tests identifying wall design parameters.

5.2.5 Investigative Test Programmes

The principal problem associated with the use of the method of reduction of results on investigative tests is one of comparison. The use of the $K_{300}$ partial safety factor
may be advantageous in commercial testing as it allows the manufacturer, who has comprehensively tested his product, to achieve higher performance levels, but for impartial research comparison becomes a problem unless each variation is treated in an identical way. Further problems derive from the different treatments of stiffness and failure results: using the mean of the former and the lowest value of the latter.

In tests to evaluate material parameters, two panels have generally been tested so that comparison has not been a problem. Where three or more panels have been tested and a comparison is required, the two results closest to the mean have normally been used. This implies that the stiffness results are of prime importance. The panels are selected to give results as close as possible to the overall mean, although it is essential that failures at both ends of the vertical load spectrum are covered. Where only one panel has been tested it can be directly compared using the safety factors for two tests where necessary if there are only small differences in design between it and the normally tested configuration.

Comparisons with other research or test programmes require more care and full details of the number of panels tested in stiffness and strength must be known when the individual panel performances are not available.

Derivation of overall design values for a material type and comparison with Code of Practice design values are also difficult. The Code values have not been established from a single extended test programme on one board, but are the result of many two panel programmes on similar boards. Combining all the test results will probably allow the maximum value of $K_{300}$ to be used. However, the derivation of design values for a range of boards also requires a knowledge of whether or not the lowest quality board has been tested. Thus the design values should, in principle, include two safety factors, i.e.:
Design value = test value x factor for knowledge of material

Considering the Code value for a material type, the test factor will be unity if many tests have been conducted, but the material factor will have to be less than unity if it is possible that a weaker board, which still meets the general specification for the material, has still to be tested. A standard panel test on a particular board is different. Here the factor for test will be low if only two panels are tested but the material factor will be unity because the results will apply to that material alone. Hence, it can be seen that the standard test results and the Code values may be similar but for very different reasons. Furthermore, it is quite possible that a standard two panel test on a board may yield design values lower than those quoted in the Code.

The same problem, based on safety factors, also applies to investigative test programmes when many different tests are carried out on panels or walls of one type. In theory, only one test may have been carried out on each variation necessitating a low safety factor, but the trend line linking the results could be treated with a unity safety factor if it represents more than five tests. In practice, if the research tests are used solely to determine modification factors for design variables, the problem of comparison will be eliminated. However, care is needed if the results are compared with design values for similar panels.

Individual problems related to the reduction of test results will be considered, along with the panel performance values, in Chapters 6 and 8.

5.3 THE RACKING TEST RIG

5.3.1 General Introduction

Timber frame walls may be tested in either a horizontal or a vertical mode. One advantage of the former case is that the test apparatus can be quickly set up, however, for more
prolonged work the greater loss of laboratory floor space may present problems. A second benefit is that direct loads on the face of the panel can more easily be included with the standard vertical and horizontal racking loads. The need for this test is extremely limited and the advantages of the horizontal test are outweighed by doubts concerning non-standard behaviour and secondary resistances due to the method of test.

The type of frame required for testing a panel in the vertical mode depends on the method of test and the means of applying vertical load. In New Zealand, where vertical load testing is not common and any requirement for light loads can be met by dead weights, the rigs need only have a base to which the panel can be attached and a braced vertical member to react against the racking load. The same arrangement may also be suitable for the American ASTM E-72 racking test (as shown in Figure 5.6), where the holding down force reacts through additional tie rods which can be attached to the base of the test rig.

TRADA have recently drawn up a proposal for a test rig which uses dead weights for vertical load application in the PRL/UofS test (Figure 5.7). The vertical loading system is extremely versatile allowing:

(i) freedom in extension of panel length,

(ii) easy alteration of load points;

(iii) variation of load along the panel length, and

(iv) low longitudinal resistance, even in failure tests.

The one major disadvantage is the need to manhandle the weights particularly in the standard panel tests, which will require 2.5kN per loading point to achieve the 5kN per stud vertical load. The weights could be reduced by adopting an unequal lever arm in the loading bar, but this is more likely to lead to instability.

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The more common method of carrying out a racking test with vertical load is to use a self reacting frame and hydraulic jacks. Due to the size of the panel and the jacks, the frame opening needs to be at least 3.0m by 2.8m to test a standard panel and it is obvious that the length of panel that can be tested is restricted by the frame. However, depending on the means of fixing, it is possible to alter the loading points as desired, but variations in vertical load along a panel length are difficult as each load needs a separate hydraulic system. One of the major problems that could be encountered with a genuine self reacting frame is that its distortion under load may affect the wall panel due to the need for base fixity. It is essential, whatever method of test is adopted, that the application of horizontal or vertical loads do not in any way affect the position of the base of the test rig to which the panel is attached.

A derivation of the self reacting frame, using smaller sections and providing external bracing to overcome the need for large moment resisting joints, may also be used with regard to the relatively light loads applied to the panel and the narrow depth of construction which allows the braces to pass either side of the panel. The method is very suitable for use on a structural laboratory floor, allowing braces and ties to be fixed direct to the stiff floor foundation and not to the base of the rig, which itself can be reduced in size if it is attached at intervals along its length to the floor. The University of Surrey test rig, detailed later, is an example of this type of construction.

In addition to the method of loading, other details need to be considered in the design of the apparatus. Firstly, base fixing of the panels, which is normally achieved by bolting, where the base of the frame will either have to be accessible to allow location and tightening of bolts (as in the TRADA rig, Figure 5.7), or will have to be drilled and tapped at fixed intervals to take the bolts. The latter method clearly reduces the versatility and ease of use of the rig as all panels will have to be accurately drilled to
suit the test frame. Secondly, the panel must be laterally restrained. Unless the test frame is itself transversely braced, a form of connecting arm linking the panel to a stiff support, such as a structural wall, will be required. The TRADA system uses short stiff arms with a guideway through which the head plate of the panel passes. Other systems use longer radius arms attached to the panel.

The loading system for the racking force will need to be hydraulically operated. It can either apply a direct compression force on the panel, or work through a tension jack, linked by cables to the panel. The load is normally applied at the leading end of the wall or through a substantial head plate to fully motivate the panels. As previously noted, the vertical load can either be applied by an hydraulic system or by dead weights. It is essential that whatever method is used, no significant restraints on the wall, in terms of planar movements, are imposed. Thus the horizontal load must be transmitted through a universal joint, which allows the panel to uplift and rotate, and the vertical loading system should incorporate "frictionless" bearings, unless the loads are allowed to move with the panel.

A datum frame is used to mount the deflection gauges. Two gauges are required as a minimum; the first to measure the racking deflection of the front of the panel and the second to measure sliding of the base. Two further gauges are normally included, one to record the racking deflection of the rear of the panel, which will detect closing up of the panel(s) under load, and the other to measure the uplift of the leading stud which helps in assessing panel performance.

The test requirements for the PRL/UoF method noted in the draft Code of Practice (BSI to be published) are given in Appendix A.

5.3.2 The University of Surrey Test Rig

When the University test rig was designed, the only
other frames specifically used for timber frame walls had a maximum limit on panel length of 2.4m. This constraint excluded experimental work on the effects of wall length, panel combination and openings within walls. Thus the first requirement of the test rig was that it should accommodate walls up to 4.8m long, which covered the longest panel then being manufactured by the sponsors, Guildway Ltd.

The standard height of panels for dwellings was 2.4m; a second height 2.7m was commonly used in public buildings and there was a possibility of 2.1m panels being introduced for mass housing in developing countries. All three heights had to be accommodated in the frame which made it necessary to have an adjustable top member for vertical load tests.

A survey of the test frames used for racking tests in Great Britain at the time, showed a high degree of variation. The most commonly used rig was that of the Building Research Establishment's Princes Risborough Laboratories which, being Government owned, was accepted as producing authoritative results and furthermore had been used in the first work establishing a British test method. The requirement for early accreditation of the Surrey test results entailed the fundamentals of the BRE frame being incorporated in the Surrey design.

The final version of the test rig is shown in Figure 5.8. The frame consists of a rectangular steel frame made from 150mm deep channel and box sections. The bottom rail of the frame was bolted down, initially, to the heavy floor of the laboratory, but later when the frame was relocated, rawlbolts were used in the concrete floor slab. It is essential to testing that the base plate is level. This requirement was achieved by casting a concrete plinth supporting the bottom rail, which rested on compressible mediumboard packers. Perfect level was achieved by careful torqueing of the holding down bolts. The bottom rail was strengthened and then drilled and tapped to take M15 panel
fixing bolts at strategic points along its length. This method of panel fixing is a weakness in the test equipment, but has not presented undue problems, except that the M12 bolts initially used stripped their threads in positions where panel uplift was high.

The top beam was made from two spaced channels so that the vertical load jacks could be mounted on a rotating base fitted between the channels. It was hoped that the pivot length of the jacks, approximately 300mm, would be sufficient to allow movement of the panel without motivating a significant horizontal component in the jack load. In failure tests where panel movements of 60mm or more were recorded, this was clearly impossible. The bases were then fixed with the jacks vertical and the load was applied through a roller train to reduce friction to within acceptable limits. The design of the jack supports has also been noted as a weakness, making tests with load points other than at 600mm centres difficult. A better method would have been for a jack baseplate to have been fabricated which could slide along the bottom edge of the channels making the loading position infinitely variable. The vertical columns were drilled to allow the top rail position to be adjusted, depending on the panel height. 60mm diameter holes were drilled through the columns at fixed heights to take the racking jacks, thus enabling maximum advantage to be made of the distance between columns for accommodating panels. The weakness of this system is that the jack heights are limited to the number of holes drilled and cannot cover any small variations in wall height which may be necessary to allow specialist base fixings.

The whole surround frame was braced by six steel tie rods to provide the reactions to the jacking loads and to allow the horizontal load to be applied from either end of the frame. The ties were threaded at one end and, as they acted eccentric to the frame, torqueing of the nuts enabled the rig to be accurately lined and levelled.
The standard method of holding down panels was to bolt them to the base of the test rig. The holes in the base plate were drilled at 200mm centres with the first hole sited 170mm from the front of the panel. A soleplate, initially of softwood, but later of hardwood, extended in two pieces along the length of the base plate. Each 38 x 89mm section was held down by countersunk bolts in addition to the panel bolts. Further bolts were included in tests that did not require the panels to be bolted down. Wide clearance holes were drilled in the soleplate for the panel bolts. These M12/M15 bolts were provided with 50mm square washers cut from 6mm steel plate which allowed the bottom rail to be rigidly fixed and prevented it cupping under the eccentric action of the load transferred from the sheathing.

Provision was made on the test frame for two specialised panel fixing methods. The first allowed simulation of the ASTM E-72 test and consisted of a pair of 20mm diameter tie rods, fixed to the base of the test frame but allowed to rotate in the plane of the panel. The rods acted either side of the leading stud and were linked at the top by a steel bar which could be tightened down onto the top of the panel to prevent uplift. The rods were strain gauged so that the force motivated in the rods due to the racking action of the panel could be monitored. The rods were never used for ASTM testing, except in tests to show that the holding down force would be very much higher than could be obtained from any practical restraint. However a series of tests was conducted in which an additional stud was nailed in front of the leading stud and held down by the restraining rods. This test attempted to model the practical situation in which a panel would always be preceded or would be fixed to a return wall either of which would provide some resistance to uplift. The results of these tests are discussed in Chapter 6.

The second fixing allowed anchor straps to be nailed to the panel studs at 600mm centres to simulate direct attachment of the wall to the foundations. The fitting consisted of a mild steel block welded to the base of the
test rig, drilled and tapped to take the anchor strap fixing bolt. 3.3mm diameter holes were drilled in the strap so that up to six 65mm long nails could be used to attach it to the stud of the panel, or studs if the strap was positioned at a panel joint as shown in Figure 5.9. Details of tests using this arrangement are included in Chapter 6.

The fixtures at the top of the panels consisted of the header plate and the lateral restraint arms. The header plate, of 38 x 89mm softwood, was fixed to the top rail of the panel either with nails at 300mm centres, or with 10mm diameter coach screws, which simplified the re-use of the test panel. Where possible the header plate was a single unit, but on long panels, a break was necessary. This was situated at least 600mm away from any panel joint.

Lateral restraint of the panel was achieved by using 2m long transverse radius arms of steel angle bolted to the header plate and top rail of the panels and pinned to supports mounted on the structural wall of the laboratory. At least two radius arms were used in a wall test. A third and fourth arm were added for walls longer than 2.4m and 3.6m respectively.

5.3.3 The Loading System

The loading equipment may be split into four section covering:

(i) the racking load,

(ii) the vertical load,

(iii) the hydraulic system, and

(iv) the monitoring system.

The racking loads were applied through an 8 tonne capacity single action ram, with a 300mm stroke, rigidly attached to the uprights of the test frame. The line of action was along the top rail of the panel and the load was
applied via:

(i) a load cell to measure the applied force,

(ii) a ball socket and roller assembly, allowing the panel to move without restriction in its plane, and

(iii) a steel plate nailed to the frame of the panel.

In early tests, the 250mm long steel plate partly covered the sheathing contrary to present test requirements. Checks were made that showed this had no influence on results.

On panels longer than 3.0m, a second jack was used to apply load at the centre of the panel to obtain a better simulation of the loading effect of the horizontal diaphragm. The second jack, identical to the first, was mounted on the rear upright of the frame and loaded a sledge notched into the header plate, approximately 2.1m from the leading edge of the panel. The ram and sledge were linked by a yoke and two prestressing wires running either side of the panel. A load cell was mounted between the ram and the yoke. Adjustments could be made to the yoke to equalise the initial tensions in the wires and avoid eccentricity of loading. When used, the second racking jack was hydraulically linked to the first, which approximately equalised the loading at the two points.

Vertical loads were applied by 3.5 tonne capacity single action rams with a 120mm stroke. In all the tests included in this investigation, the rams were fixed in a vertical position and the system incorporated roller trains to reduce friction. The leading stud ram was independent of the others and was mounted 50 - 75mm behind the leading edge of the panel to allow for the large failure test deflections. It applied load through a roller train and a steel plate nailed to the wall header plate, which effectively prevented any
interference with the panel sheathing. Subsequent rams were coupled together hydraulically and each loaded two studs through steel I section distribution beams. This system presented a problem on walls with an even number of loading points, e.g. 0.6 and 1.8m long, etc., which was overcome by hanging a dummy reaction point from the top of the test frame to support the rear end of the last distribution beam.

The hydraulics of the jacking system were controlled from a custom built panel. A single pump and reservoir unit maintained the three separate but identical circuits controlling the racking jacks, the single ram over the leading stud and subsequent vertical load jacks. The hydraulic circuit shown in Figure 5.10 allowed fine adjustment of loading and unloading and incorporated a pressure gauge calibrated to show the jack load.

In the case of the vertical loads, the pressure gauges were used to monitor load and adjustments could be made during testing to maintain a constant value. It should be noted that the distortion of the panel under racking load, tends to increase the load on the leading jack and to reduce the load in subsequent rams so that careful control of the inlet and outlet valves is necessary. The racking load circuit included a pressure gauge to give an indication of load, but the main readings were taken from the load cell mounted on the jack. In early tests, the load cells were read using a standard digital output box converting the electrical resistance directly to load. However, for the majority of the tests covered in this investigation, the load cell drove a channel of the data logger, allowing the load to be continuously monitored and stored.

Details of the loading systems are shown in Figure 5.11 and are illustrated in Appendix C.

5.3.4 Deflection Measurement

Initially deflections were measured using dial gauges. The majority of results covered by this investigation,
however, have been recorded using linear variable displacement transducers (LVDTs) feeding into a conventional data-logger. This has reduced the necessity for incremental loading and in standard tests the deflection can be continuously monitored using a digital volt meter with a scan taken of all load cells and all LVDTs at required deflection intervals. The four measuring points for the standard tests described in 5.3.1 and shown in Figure 5.11 have been supplemented in the research programmes to gain additional information on panel behaviour, e.g. relative movement of the studs at panel joints. Two particular systems of measurements are detailed below.

The first has been used to monitor the movement of sheathing boards and consists of perspex blocks with 50mm square sides bolted to the sheathing so that deflections can be measured in the horizontal and vertical directions. These blocks can be placed close to the perimeter of each sheathing board, preferably along diagonals, and the deflections used to interpret board movements (see Figure 5.12). The number of measurements needed for this type of investigation has meant that on all but very simple walls with few sheathing boards dial gauges have had to be used in place of LVDTs.

The second system covers nail slip by measuring the relative movement between the board and the frame at various locations on the panel. The method of measurement is shown in Figure 5.13. It consists of a double headed nail fastened to the stud through a 20mm diameter hole in the sheathing. Demec pips are then attached to the nail and to the boards as shown, and the relative movement between the stud and the board position recorded during the racking test. To best define the nail movements, it was necessary to put three sets of demec points along each of the frame members attached to the sheathing measuring movements both parallel and perpendicular to the frame member. Readings were taken at three stages during the stiffness test at equal increments of racking deflection. However this method was too slow and laborious relative to its value to the test programme and further problems were encountered with creep in the panel.
affecting the overall test performance. (A full set of readings on a standard panel took thirty minutes to record, so that a standard deflection cycle took two hours compared with a Code recommended duration of ten minutes). The number of readings were substantially reduced by:

(i) ignoring deflections perpendicular to the frame as these were found to be small in the initial tests,

(ii) taking only the central reading on the frame member, (the three readings previously used showed little difference and were normally averaged by the central value),

(iii) ignoring the internal stud where readings were always very small.

The demec gauge positions subsequently used on a standard panel are shown in Figure 5.13 d.

A data logging system was used to record all test data, but the results were computed on a separate system, which included a high quality printer and plotter for outputting the final results. Only in recent tests, when the movement of individual boards has been monitored with up to twenty two LVDTs, has it been necessary to interface the data collection and processing systems. Even then the data from the demec gauges has had to be entered separately and by hand.

The details given above cover all the equipment associated with the full-scale racking tests except that equipment used in very specialist tests, which will be covered later, in conjunction with the test data. Further details of the racking test rig and the ancillary equipment are shown in the photographs included in Appendix C. The equipment used in other investigations, such as the small-scale predictive tests, is detailed separately.
5.4 TEST PANELS

5.4.1 General Details

The test panels are normally designed to include the main features of the construction forms noted in Chapter 2. Thus in American tests stud spacing is often 400mm. In Britain a 600mm module is used with studs at approximately equal centres and sheathings to plain walls using 2.4m x 1.2m boards laid vertically and jointed on a single stud within panels. The form of construction of test panels has changed little since the early PRL tests, thus this section deals with the method of construction and general details of the panels used in the current test programme. Further specific details of construction, including descriptions of the materials used, will be included with the test results. The general details covered here are:

(i) the layout of panels, including framing around openings,

(ii) the range of materials used in the panels,

(iii) fixings, both to the base of the test rig and vertically between panels,

(iv) attachment to brick walls.

5.4.2 Panels for Standard Tests

The standard panel used for comparative testing of materials for use in wall construction is 2.4m square overall, typical details are shown in Figure 5.14. The frame consists of top and bottom rails separated by five studs at nominal 600mm centres, all using the same section size of timber. Throughout this work CLS timber has been used in its most common finished size of 89 x 38mm which thus fixes the length and spacing of studs. The stud width represents the narrowest
dimension for successful jointing of two boards on one stud. In practice, the depth could be reduced for use in internal partitions, when great care would be necessary in predicting behaviour, but may also be increased to achieve better standards of insulation without adversely affecting performance. Either Canadian construction grade or British GS graded timber has been used for the frames, although the presence of knots has little significance on racking performance, which is very much more dependent on the nail resistance and thus the density of timber. The species of timber has varied through the investigation; most common (and in historic order) have been: Canadian hem-fir, Canadian spruce/pine/fir (SPF) and Swedish redwood/whitewood, although the first of these is not now used. As with the grade of timber, the type of wood is not directly relevant to performance, as variation within a species grade is as great as between grades or between species. Care has been taken in the selection of frame timbers to avoid split, twisted or bowed wood, which could either cause problems in seating the panels squarely in the test rig, or in joining boards on centre studs. Except in special tests with horizontal sheathings, no noggins have been used in the frames.

The frame fixings have varied over the years from three 75mm long 3.25mm diameter wire nails to two 100mm long 4.0mm diameter coated nails, although the latter are more common. The nails for the test frames have been hand fixed, although in practice they would be gunned. Holes have been pre-drilled in the top and bottom rails for the end studs only to prevent splitting.

The sheathings most commonly used in the research programmes were initially 9.0mm thick medium density fibreboard (MDF) and latterly 9.5mm Canadian softwood plywood (CSP). To cover a wider range of typical walls, 12.5mm bitumen impregnated insulating board (B11B) and 12.5mm plasterboard have also been extensively tested. Within the contract tests however, a very much wider range of types and thicknesses of materials have been considered and are
detailed in Chapter 6. The maximum board size has always been 1.2m long by 2.4m high, except for very specialised tests, using chipboard, although the boards have often been trimmed from imperial sizes. Stud positions in plain panels have been fixed by the 1.2m dimension.

Sheathing fixings have varied through the investigation. Initially staples were used at 100mm centres, but they have not been used since 1977 in investigative tests and they have not been recommended for use in the Code of Practice BS 5268 part 6 (BSI, to be published). Nail sizes for wood based sheathing have varied from 2.87 to 3.25mm in diameter and from 50 to 63mm in length, although the standard size nail of 3.0mm in diameter noted in the Code of Practice has not been tested. The lengths all conform to standard guidelines for end penetration. Protective treatments have been hot dip galvanising on the earlier hand driven clout head nails and electro-galvanising on the now standard gun-driven wire nails. Spacing of nails has been in accordance with standard recommended practice, except when investigating nail spacing, viz.: 150mm centres around board perimeters and 300mm to internal studs, except for B11B, where both distances are halved. Other methods of fixing, such as screwing or gluing and nailing, have been used in commercial tests.

Insulation material, which is normally laid between studs, has been omitted from all test panels as has the polythene vapour barrier and the breather paper.

Where tested, internal linings have been of plasterboard nailed, at the standard centres quoted above, with special 40mm long 2.65mm diameter hot gip galvanised hand driven nails. Further specialised uses of plasterboard, such as in separating walls, are noted in Chapter 6.

Panel construction for research tests has been carried out in the laboratory specifically to achieve consistency in quality, particularly in the spacing of nails and in the depth to which they are driven, i.e. with the head just
penetrating the board. The frames and sheathings were assembled on the laboratory floor prior to erection in the test rig. Where a lining board was used in addition to the sheathing, it was normally fixed after the panel had been correctly bolted onto the base plate and the lateral braces had been fixed. The exception was when the panels were to be re-used; then the lining was nailed before erection and holes were cut in the board to allow holding down and connection bolts to be inserted and tightened.

5.4.3 Panels for Investigative Work

Plain panels, other than standard panels, were built to a standard 600mm grid with internal studs positioned, either to allow two boards to join on the stud or to be central to the board. Top and bottom rails were continuous even in the 4.8m panels, which were the maximum length tested. Sheathing boards were normally 1.2m long with one 0.6m board used to make up odd lengths (1.8m, 3.0m, etc.).

Window and door panels have incorporated standard details wherever possible. However, some experimental work has necessitated the use of unusual features in order to simplify the overall programme. Details of such panels are given in the sections dealing with their testing. Solid lintols have been used throughout, either a single 90mm wide timber or, more commonly, a section built up from two 38mm wide timbers and a piece of 9.5mm ply. Sheathing details depend on the requirements of the test, but in general the simplest method, using rectangular sheets, has been adopted which has been seen to be the weakest practical solution.

External fixing of the panels was standardised throughout the tests. However, an investigation was carried out on support techniques to relate the testing standard to the practical situation. The standard base fixing comprised M15 bolts, one in each bay of the panel with a minimum of two bolts per panel. The first bolt in a wall was always placed as near the leading stud as possible, which with the panel
in standard position in the test rig, was 170mm from the front of the panel; the Code of Practice requires this distance to lie between 150 and 200mm. In recent years, a 50mm square steel washer has successfully been used with the bolts to give better bearing which prevents the bottom rail from twisting due to the lifting action of the sheathing under the racking forces. Early tests used much smaller washers, which affected the mode of failure, although not the performance of the wall. The bolted method of base fixing achieved the requirement that the bottom rail(s) of the wall were firmly fixed to the test rig and exhibited no significant deflection, in terms of sliding or uplift, during the test. Blocking the end of the panel was not necessary when the bolts were tightened so that the washer just started to bite into the bottom rail.

In all tests, the panels were laid onto a soleplate made up of 89 x 38mm timber which, for greater repeated use, was made of hardwood. The soleplate was separately bolted to the test rig and those fixings were increased in the specialist tests where the effect of different holding down methods were investigated if there was no direct fixing between the panel and the test rig.

The standard method of joining panels in series has been to use three M12 bolts at heights as close as possible to 300, 1200 and 2100mm above the panel base. Window openings have occasionally made it necessary to adjust these heights. In earlier tests, 75mm long 4.0mm diameter double-headed nails were skew driven at 300mm centres to achieve the same effect. A header plate continuous over the joint by at least 600mm has been used in all standard tests. A special programme of tests, however, has investigated the effect of reducing the vertical fixings and also of omitting the head plate.

In the special tests, investigating the combined action of plain timber framed walls with a brick skin (see Figure 5.15), the two 2.4m long timber frames used were of standard construction;
only one being externally sheathed with mediumboard. The internal wall faces were then free to take other claddings which could be replaced after testing to achieve a greater diversity of results. The brickwall was approximately 5.2m long, with 225mm end returns, by 2.5m high and consisted of a single leaf of fletton bricks bonded with a 1:1:6 cement/lime/sand mix. The brickwork was built up from the floor of the laboratory on a mortar bed covered with a bitumenous damp proof course. The minimum separation of the walls was 38mm where the panel was externally clad and the two leaves were connected by 'Chevron' flexible wall ties at 600mm centres, both horizontally and vertically. Each tie was fixed to the timber frame with two 50mm long 3.25mm diameter galvanised clout head nails. The spacing for the ties was taken from the Timber Frame Housing - Design Guide (TRADA, undated), although it is now evident that the 600mm vertical spacing was incorrect and had been reduced to 400mm. However by using too few ties, the tests gave a conservative estimate of the combined action and made allowance for bad building practices.

The two timber frame panels were separated in the test rig by 40mm to allow individual testing from either end. The panels could be linked by inserting a 90 x 40mm timber and bolting through the whole assembly with three M12 bolts in the standard manner.

In conclusion, the detailed requirements for test panels given in the draft Code of Practice BS 5268 part 6 (BSI, to be published) are included in Appendix A.
A 7.5mm deflection limit causes a large residual deflection.

During the second cycle, the load at 7.5mm deflection from the original datum will be less than the first cycle but under continued loading the panel will stiffen slightly, quickly reaching the unique load deflection relationship.

The second cycle load will represent a deflection of 7.5mm plus the residual panel set of the first cycle.

As the deflection has increased there will be even greater residual set after the second cycle so that the third cycle will again be higher and the performance will slowly stabilise with further cycles.

Notes

(i) A 7.5mm deflection limit causes a large residual deflection.

(ii) During the second cycle, the load at 7.5mm deflection from the original datum will be less than the first cycle but under continued loading the panel will stiffen slightly, quickly reaching the unique load deflection relationship.

(iii) The second cycle load will represent a deflection of 7.5mm plus the residual panel set of the first cycle.

(iv) As the deflection has increased there will be even greater residual set after the second cycle so that the third cycle will again be higher and the performance will slowly stabilise with further cycles.
Figure 5.2 Interpretation of Design Loads for Racking Test Results

(a) Typical results for brittle sheathing e.g. plasterboard (see next page for full details of Figure 5.3)
(b) Typical results for weak sheathing e.g. B11B

(c) Performance lowered by using interpolated strength result for design at 2.5kN/stud. Curve (iii), as drawn, may safely be used for design.

Key to (a), (b) and (c)
- design load by test
- design load by basic racking resistance
- racking resistance curves based on design loads

at: (i) 5kN/stud
(ii) 2.5kN/stud
(iii) 0kN/stud

A Basic racking resistance value (BRR)
B Median racking resistance value (MRR)

The shaded area represents the loss in efficiency of performance when using BRR for design.

Figure 5.3 Basic Racking Resistance Performance
1. First cycle stiffness load (at deflection 0.002 x the panel height).

2. Fourth or lowest cycle stiffness load.

3. Factored lowest cycle stiffness load. (Averages load 2 with results from similar panel tests and applies partial safety factor $K_{300}$ to account for the number of panels tested).

4. Stiffness test load (equal to 1.25 x load 3 and assumed equivalent to a safe racking load at 0.003 x panel height deflection).

5. Panel set prior to second cycle.

6. Panel set prior to fourth cycle.

7. Failure load.

8. Factored failure load or strength test load. (Takes the lowest failure load from similar panel tests, applies a partial safety factor $K_{300}$ for the number of similar tests and divides by an overall factor of safety of 1.6).

9. Design racking load (the lower value of the stiffness and strength criteria 4 and 8).

10. Test racking strength load, as defined by the Code takes the lowest failure load x $K_{300}$ only. This allows variable overall factors of safety to be used in the determination of the design racking load.

Figure 5.4 Racking Load Versus Deflection Graph For A Single Vertical Load Case
<table>
<thead>
<tr>
<th>No. of similar panels tested under the same conditions</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_{300}$</td>
<td>0.80</td>
<td>0.87</td>
<td>0.93</td>
<td>0.97</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table 5.1 Modification Factors For Similar Tests

<table>
<thead>
<tr>
<th>Vertical Load (kN/stud)</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>2.5</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_{303}$ for median racking resistance</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.7</td>
<td>0.82</td>
<td>0.94</td>
<td>1.00</td>
<td>1.05</td>
<td>1.15</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td>$K_{302}$ for basic racking resistance</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td>1.18</td>
<td>1.35</td>
<td>1.43</td>
<td>1.50</td>
<td>1.65</td>
<td>1.77</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.2 Modification Factors For Point Loads On Studs At 600mm Centres On A Standard Test Panel
U of S: Tests on 9mm Medium hardboard (Karliti).

**Stiffness test**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Front Deflection</strong></td>
<td><strong>Worst cycle</strong></td>
<td><strong>First cycle</strong></td>
<td><strong>First cycle</strong></td>
<td><strong>Worst cycle</strong></td>
</tr>
<tr>
<td>0.00</td>
<td>0.00</td>
<td>1.01</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.00</td>
<td>0.00</td>
<td>1.01</td>
<td>0.00</td>
<td>1.00</td>
</tr>
<tr>
<td>2.00</td>
<td>4.63</td>
<td>2.00</td>
<td>2.42</td>
<td>2.00</td>
</tr>
<tr>
<td>2.00</td>
<td>4.63</td>
<td>2.00</td>
<td>2.42</td>
<td>2.00</td>
</tr>
<tr>
<td>4.00</td>
<td>6.70</td>
<td>4.00</td>
<td>5.22</td>
<td>4.00</td>
</tr>
<tr>
<td>5.00</td>
<td>7.22</td>
<td>5.00</td>
<td>6.54</td>
<td>5.00</td>
</tr>
</tbody>
</table>


2.25 7.23 2.51 6.54 1.73 8.63 1.27 11.58 1.11 11.23

Sliding at end of cycle

0.15 7.17 0.14 4.73 0.09 4.33 0.09 11.35 0.16 11.57

**Ultimate load test**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Front Horiz. Rear Uplift slide</strong></td>
<td><strong>Defl. Load</strong></td>
<td><strong>Defl. Load</strong></td>
</tr>
<tr>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2.00</td>
<td>3.45</td>
<td>2.00</td>
</tr>
<tr>
<td>6.00</td>
<td>7.17</td>
<td>6.00</td>
</tr>
<tr>
<td>10.00</td>
<td>8.79</td>
<td>10.00</td>
</tr>
<tr>
<td>12.00</td>
<td>9.27</td>
<td>12.00</td>
</tr>
<tr>
<td>14.00</td>
<td>9.86</td>
<td>14.00</td>
</tr>
<tr>
<td>16.00</td>
<td>10.44</td>
<td>16.00</td>
</tr>
<tr>
<td>20.00</td>
<td>10.56</td>
<td>20.00</td>
</tr>
<tr>
<td>22.00</td>
<td>10.77</td>
<td>22.00</td>
</tr>
<tr>
<td>24.00</td>
<td>9.52</td>
<td>24.00</td>
</tr>
<tr>
<td>26.00</td>
<td>8.33</td>
<td>26.00</td>
</tr>
</tbody>
</table>

---

**Panel UKS-2: 2.4m x 2.4m plain on SPF studding.**

63mm long 3.25mm dia. gunned nails at 150/300mm centres.

**Stiffness test**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Front Deflection</strong></td>
<td><strong>Worst cycle</strong></td>
<td><strong>First cycle</strong></td>
<td><strong>First cycle</strong></td>
<td><strong>Worst cycle</strong></td>
</tr>
<tr>
<td>0.00</td>
<td>0.00</td>
<td>1.19</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>1.00</td>
<td>2.32</td>
<td>1.19</td>
<td>0.00</td>
<td>1.00</td>
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<tr>
<td>2.00</td>
<td>4.00</td>
<td>2.00</td>
<td>1.85</td>
<td>2.00</td>
</tr>
<tr>
<td>2.00</td>
<td>4.00</td>
<td>2.00</td>
<td>1.85</td>
<td>2.00</td>
</tr>
<tr>
<td>4.00</td>
<td>6.47</td>
<td>4.00</td>
<td>5.20</td>
<td>4.00</td>
</tr>
<tr>
<td>5.00</td>
<td>7.17</td>
<td>5.00</td>
<td>6.78</td>
<td>5.00</td>
</tr>
</tbody>
</table>

4.90 7.17 4.89 4.78 4.95 9.95 4.95 11.78 4.99 11.57

2.64 7.17 2.61 6.78 2.17 9.95 2.17 11.78 1.46 11.57

Siding at end of cycle

0.15 7.17 0.14 4.73 0.16 4.92 0.00 11.78 0.16 11.57

---

**Ultimate load test**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Front Horiz. Rear Uplift slide</strong></td>
<td><strong>Defl. Load</strong></td>
<td><strong>Defl. Load</strong></td>
</tr>
<tr>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2.00</td>
<td>3.45</td>
<td>2.00</td>
</tr>
<tr>
<td>6.00</td>
<td>7.17</td>
<td>6.00</td>
</tr>
<tr>
<td>10.00</td>
<td>8.79</td>
<td>10.00</td>
</tr>
<tr>
<td>12.00</td>
<td>9.27</td>
<td>12.00</td>
</tr>
<tr>
<td>14.00</td>
<td>9.86</td>
<td>14.00</td>
</tr>
<tr>
<td>16.00</td>
<td>10.44</td>
<td>16.00</td>
</tr>
<tr>
<td>20.00</td>
<td>10.56</td>
<td>20.00</td>
</tr>
<tr>
<td>22.00</td>
<td>10.77</td>
<td>22.00</td>
</tr>
<tr>
<td>24.00</td>
<td>9.52</td>
<td>24.00</td>
</tr>
<tr>
<td>26.00</td>
<td>8.33</td>
<td>26.00</td>
</tr>
</tbody>
</table>

---

*September 1984*

---

*Table 5.3a*

---

*September 1984*

---

*Table 5.3b*
U. of S: Tests on 9mm Medium hardboard (Karlit).

PANEL UKS-1: 2.4m x 2.4m plain on SPF studding.
63mm long 3.25mm dia. gunned nails at 150/300mm centres.
5mm Deflection test: 0, 2.5 and 5kN/stud vertical load.

PANEL UKS-2: 2.4m x 2.4m plain on SPF studding.
63mm long 3.25mm dia. gunned nails at 150/300mm centres.
5mm Deflection test: 0, 2.5 and 5kN/stud vertical load.

September 1984 Figure 5.5a

September 1984 Figure 5.5b
U. of S: Tests on 9mm Medium hardboard (Karlić).

PANELS UKS-1 & UKS-2: 2.4m x 2.4m plain on SPF studding.
63mm long 3.25mm dia. gunned nails at 150/300mm centres.

Ultimate load tests: 0 and 5kN/stud vertical load.

<table>
<thead>
<tr>
<th>Vertical load (kN/stud)</th>
<th>0</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate load (kN)</td>
<td>10.77</td>
<td>19.93</td>
</tr>
<tr>
<td>Factor of safety</td>
<td>1.49</td>
<td>1.69</td>
</tr>
</tbody>
</table>

(Failure/ First cycle 5mm deflection)

DESCRIPTION OF RESULTS: TEST A5

The behaviour of the two panels was typical of medium hardboard and there was very good consistency in performance. Standard failures were noted with bottom rail nails rotating and withdrawing from the timber before finally shearing through the board material. Design values were governed by strength and the theoretical design figures very closely matched the test results.
U.of S: Tests on 9mm Medium hardboard (Karlit).

**TEST RESULTS**

<table>
<thead>
<tr>
<th>Panel No.</th>
<th>Length (m)</th>
<th>Vertical Load (kN/stud)</th>
<th>First cycle</th>
<th>Worst cycle</th>
<th>Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>UKS-1</td>
<td>2.4</td>
<td>0.0</td>
<td>7.23</td>
<td>6.54</td>
<td>10.77</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>9.43</td>
<td>9.114</td>
<td>11.25</td>
<td></td>
</tr>
<tr>
<td>UKS-2</td>
<td>2.4</td>
<td>0.0</td>
<td>7.17</td>
<td>6.78</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>9.95</td>
<td>9.653</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**MEANED RESULTS**

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>Vertical Load (kN/stud)</th>
<th>Average worst</th>
<th>Cycle load (kN)</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Test load (kN)</th>
<th>Design Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>6.66</td>
<td>7.24</td>
<td>5.39</td>
<td>5.59</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.4</td>
<td>9.28</td>
<td>10.20</td>
<td>7.68</td>
<td>7.458</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>11.40</td>
<td>12.40</td>
<td>9.97</td>
<td>9.97</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**RACKING PERFORMANCE**

<table>
<thead>
<tr>
<th>Vertical Load (kN/stud)</th>
<th>Tested Design</th>
<th>Tested Design</th>
<th>Effective Design</th>
<th>Theoretical Design</th>
<th>Theoretical Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load (kN/m)</td>
<td>Resistance</td>
<td>Resistance</td>
<td>Resistance</td>
<td>Resistance</td>
<td>Resistance</td>
</tr>
<tr>
<td>0.0</td>
<td>5.39</td>
<td>2.24</td>
<td>2.21</td>
<td>2.24</td>
<td>5.37</td>
</tr>
<tr>
<td>2.5</td>
<td>7.68</td>
<td>3.20</td>
<td>3.20</td>
<td>3.20</td>
<td>7.68</td>
</tr>
<tr>
<td>5.0</td>
<td>9.97</td>
<td>4.15</td>
<td>3.32</td>
<td>4.00</td>
<td>9.59</td>
</tr>
</tbody>
</table>

* The worst cycle load and the meaned result for the 2.5kN/stud vertical load case include interpolation of data from tests at the other vertical loads.

**U.of S: Tests on 9mm Medium hardboard (Karlit).**

**DATA INTERPRETATION**

**Source of Data.**

<table>
<thead>
<tr>
<th>Load/stud (kN)</th>
<th>Stiffness</th>
<th>Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel 1</td>
<td>2.5</td>
<td>5.0</td>
</tr>
</tbody>
</table>

**Consistency of racking stiffness results.**

<table>
<thead>
<tr>
<th>Test Load/stud (kN)</th>
<th>Panel 1</th>
<th>Panel 2</th>
<th>Racking Loads (kN)</th>
<th>Mean from mean (%)</th>
<th>Max. Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>First cycle 0.0</td>
<td>7.23</td>
<td>7.17</td>
<td>7.20</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>2.5</td>
<td>9.63</td>
<td>9.95</td>
<td>9.79</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>5.0</td>
<td>11.59</td>
<td>11.78</td>
<td>11.68</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

**Resistance of panel to cyclic loading.**

<table>
<thead>
<tr>
<th>Test Load/stud (kN)</th>
<th>Panel 1</th>
<th>Panel 2</th>
<th>Racking Loads (kN)</th>
<th>Percentage loss</th>
<th>Average loss (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First cycle 0.0</td>
<td>7.23</td>
<td>6.54</td>
<td>10</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>9.63</td>
<td>9.111</td>
<td>54</td>
<td>41</td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>11.59</td>
<td>11.78</td>
<td>11.23</td>
<td>3</td>
<td></td>
</tr>
</tbody>
</table>

**Panel set during cyclic loading (mm).**

<table>
<thead>
<tr>
<th>Test Load/stud (kN)</th>
<th>Panel 1</th>
<th>Panel 2</th>
<th>Racking Deflections (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>2.5</td>
<td>5.0</td>
<td>14.00</td>
</tr>
<tr>
<td>1.0</td>
<td>0.99</td>
<td></td>
<td>0.72</td>
</tr>
</tbody>
</table>

**Failure details.**

<table>
<thead>
<tr>
<th>Load/stud (kN)</th>
<th>Max. Load (kN)</th>
<th>Factor of Deflection at Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel 1</td>
<td>10.77</td>
<td>1.49</td>
</tr>
<tr>
<td>Panel 2</td>
<td>5.0</td>
<td>1.69</td>
</tr>
</tbody>
</table>

**Failure performance at 90% of Maximum load.**

<table>
<thead>
<tr>
<th>Load/stud (kN)</th>
<th>90% of Max. Load (kN)</th>
<th>Racking Deflections (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel 1</td>
<td>10.77</td>
<td>Start 14.00 End 22.00 Total 8.00</td>
</tr>
<tr>
<td>Panel 2</td>
<td>5.0</td>
<td>Start 17.74 End 24.00 Total 8.00</td>
</tr>
</tbody>
</table>

Table 5.3c

September 1984
Figure 5.6 Equipment for the ASTM E-72 Racking Test
Figure 5.7 The Proposed TRADA Racking Test Rig
Figure 5.8 Details of the University of Surrey Racking Test Rig
Figure 5.9 Details of Holding Down Straps

Alternative strap has two rows of holes at 25mm centres for use at board joints

90x40mm stud
Sheathing
Holes to suit 8 gauge nails

150x3ft3mm Anchor straps
10mm dia bolt
25mm M/S block welded to strap
H/S block welded to base of test rig

Base of test rig

Figure 5.10 Hydraulic Circuit for Loading Rams

Circuit 1

Hydraulic rams
Pressure gauge
Fine control value
Shut off release valve
Shut off loading valve
Direction of flow
Feed to circuits 2 and 3, both identical to 1

Oil sump
Electric pump
Deflection gauges mounted on a separate frame

1.2x2.4m panel representing the standard use of a sheathing board

Perspex block position relative to corner of sheathing board

(a) Layout of Gauges

(b) Details of Perspex Block

Figure 5.12 Deflection Gauges Monitoring Sheathing Movement
Figure 5.13 Measurement of Nail Slips Using Demec Gauges

(a) Plan of demec positions at panel corner
(b) Section A-A
(c) Optimum number of demec points on a single sheathing
(d) Reduced demec locations on a standard panel

Single stud (exaggerated for clarity)
Timber frame: 90 x 40 members
Brick wall stopped short of timber frame for access

2.4m high timber frame panel sheathed externally with mediumboard

38mm minimum, cavity

Five chevron flexible brick ties at nominal 600mm vertical centres in staggered pattern on studs at 600mm centres

Plasterboard internal lining applied and removed as required

15mm dia. holding down bolts at 600mm centres

90x40 hardwood sole plate

150x75 channel test rig base

Brick wall on DPC and mortar bed on concrete laboratory floor

Figure 5.15 Section Through Brick Wall and Timber Frame Showing Test Details
CHAPTER 6

TEST RESULTS AND DESIGN DATA

6.1 INTRODUCTION

This chapter covers the results of the tests conducted on timber frame wall panels at the University of Surrey since 1977 when the use of nails for fixing the sheathings and the omission of non structural elements such as breather paper and insulation became normal test practice. The standard method of test detailed in Chapter 5 has formed the basis for all the work but has often included the additional procedures, also noted, to extend the use of the data collected in the investigative programmes. The results have been reduced by means of the procedures outlined in Chapter 5. Direct comparison of results has sometimes proved difficult due to the differences in the standard materials used in the panels and the slight changes made to the test method during the ten year test period.

During this time, the test data has been available for use in the preparation of design methods. (Griffiths, 1978 b; Robertson and Griffiths, 1981 and in reports to the BSI sub-committee on timber frame walls (see Appendix B)). The design values have been progressively updated as further data became available both widening the scope of the method and improving its accuracy. However, as the design method is already in use major changes have been avoided wherever possible and the design philosophy discussed in Chapter 4 has been adhered to. The latest published design method based on this work: BS 5268 part 6 draft for publication (BSI,to be published), included in Appendix A, represents the state of knowledge in 1984 and is essentially a lower bound interpretation of the test results. Since then a great volume of data has been processed necessitating changes to that design method. The current investigation includes all available test data and in non controversial areas, where design factors are based wholly on test performance, details
the changes that should be incorporated into the Code of Practice. Such factors for wall design are included in this chapter together with a number which are out of the scope of the Code. The more controversial extension of wall design to cover the racking resistance of buildings is then detailed in Chapter 8 when the inclusion of limitations on the structural use of certain types of wall construction will be discussed. Thus in Chapter 6 the design values proposed represent the maximum capacity that could be attributed to a wall based on test results alone and are independent of the structural acceptability of the material.

This investigation has run very closely with the author's work for the Code of Practice and thus the design methods are similar and in places the current information has been simplified to match the requirements of the Code, for example the data for the standard use of sheathings is classified into two groups, as set down in the Code, which reflect the current use of the sheathings in the British market. It is considered that the requirements for both design methods are identical and should take account of the following points:

(i) design values should be verifiable by test, thus in the initial wall design special construction factors should not be included to cover restraints found within a building which cannot be applied in the standard test procedure,

(ii) where possible design factors should be based on appropriate theoretical considerations,

(iii) factors of safety should be consistent throughout the procedure,

(iv) where test data is limited a greater factor of safety is to be expected,
(v) extrapolation of test data should be avoided,

(vi) modification factors should include an increased factor of safety as use diverges from the normal; the increase will be inversely proportional to the amount of knowledge on which the modification factor is based,

(vii) modification factors should be analysed independently so that safety factors are not compounded,

(viii) a wider range of material than those tested may need to be considered,

(ix) the presentation and application of the procedures must be kept simple.

The test results have been divided into three sections for analysis based on the design requirements noted in Chapter 4. The first covers the factors affecting the basic racking resistance of 2.4m long plain panels and details datum racking resistance values and materials modification factors. The second section examines the major factors affecting the racking resistance of walls; covering length, vertical load and openings. The third includes special factors which will have a secondary influence on the design of walls and in general have not been included as variables in the Code of Practice. In all these cases the design values are based on the stiffness and strength performances of the test panels which are a very condensed form of the test data. Thus a fourth section has been included to cover more comprehensively the behaviour of the panels and walls under test.
SECTION A
STANDARD PANEL TEST RESULTS AND
FACTORS AFFECTING BASIC RACKING RESISTANCE

6.2 RESULTS OF STANDARD PANEL TESTS

The variables considered in this section relate to the materials used in the construction of the panels and include:

(i) sheathing: board type, thickness, density, etc.,

(ii) nails: type, diameter, length and spacing,

(iii) frame: stud material, quality and size.

The results are all gained from standard tests on 2.4m long plain panels and thus the only other major variable included in the tests is that of vertical load. Here, in nearly every case, the tests have been conducted for stiffness under zero, 2½ and 5kN/stud vertical loads and for strength at the two extremes.

The most significant variable is that of the sheathing type. Therefore, to allow direct comparison of the sheathings, the frame material and nail type and size were held constant wherever possible. However, minor variations in materials have occurred through the test period as described below.

Up to 1978 the standard frame timber used in tests was construction grade hem-fir from Canada. This was obtained from a single source and was thus of uniform quality. The timber came from an area noted for its slow growth and quality of wood. As neither property is directly judged by visual grading, it must be noted that within the construction grade this timber was of particularly high quality. The density of the wood and the excellent nail holding properties noted during assembly and dismantling of panels would certainly have benefitted panel performance. The standard nails used at
this time were hand driven 50mm long, 3.25mm diameter hot dip galvanised clout head nails. Two nail spacing models were used; 150mm on board perimeters and 300mm internally for denser sheathings (which henceforward is shown as a spacing of 150/300mm) and 75/150mm for low density sheathings, principally bitumen impregnated insulation board.

In 1979 the hem-fir became unobtainable as a result of difficulties experienced in Canada with the boron diffusion treatment process used by the main supplier. It was replaced by the more commonly used spruce/pine/fir (SPF) amalgamated species group. Construction grade material was again used which according to BS 5268 part 2 should have been a direct replacement for the hem-fir in terms of allowable stresses. The quality of the wood was also good but the nail holding properties of the timber were noticeably reduced in comparison to the hem-fir. Later in the programme a different supplier brought in a much lower quality of SPF although its grade was identical to that previously used. These differences in the stud material can be seen to have significant influence on the test results.

In 1984 a further change was made in stud material. The introduction of the GS grade European redwood-whitewood imported from Sweden was due to financial considerations on the supplier's behalf, following the rise in value of the dollar. The wood was of high quality within the grade and exhibited good nail holding characteristics. The wood was finished to Canadian lumber sizes (CLS) and thus throughout the tests the frame material size has been constant. The nominal size of 90 x 40mm reduces in practice to 89 x 38mm and is now standard for use in external timber frame walls.

The method of nailing was changed in 1981 when a-gun nailing tool was introduced to simulate more closely conventional industrial practice. The gun fired electro-galvanised wire nails with smaller diameter heads than those of the clout nails. At that time, one of the most influential suppliers of timber frame houses had introduced a nailing
standard which allowed 50mm long by 2.87mm diameter nails on boards of 9.0mm thickness or less, but required 63mm long by 3.25mm diameter nails on thicker boards. This standard was followed for a period, but only affected the tests on 9.0mm mediumboard and by 1984 the 3.25mm diameter nail was used in all tests allowing direct comparisons once more. The length of 63mm was considered unnecessary, 50mm being adequate according to BS 5268 part 2 guidelines. However the 3.25mm diameter and 50mm length combination was difficult to obtain in quantity, making it unsuitable for standard tests. Standards such as the one noted will be unnecessary with the introduction of a design method covering nail sizes. General limitations may be placed on the diameter of nails and, together with head size, may be specifically limited by the type of sheathing. Nail length however should always be governed by the depth of penetration required in the frame timber as determined from BS 5268 part 2.

Throughout the tests plasterboard has been specially fixed using hand driven 40mm long clout head nails with a hot dip galvanised protection treatment. Initially the nails were 2.87mm in diameter, but since 1984 this has been reduced to 2.6mm. This specification follows the board manufacturer's guidelines for the most suitable type of nail which is based on factors other than racking resistance.

Frame nails have varied throughout the test period. They were considered to have no influence on frame properties and thus no record has been kept of sizes used in specific tests. The nails have varied in length, from 90 to 100mm, and in diameter, from 3.25 to 4.1mm. Treatment has been either electro or hot dip galvanising.

The boards covered by the standard panel tests are:

a) Plywood: Canadian and North American Origin
8.0 to 12.5mm in thickness douglas fir or Canadian spruce ply (CSP) in either sheathing or select sheathing grades.
b) Mediumboard (MDF): Scandinavian origin, different manufacturers 9.0 to 15.5mm in thickness, variations in density and surface finish.

c) Bitumen impregnated insulation board (BIIB): European origin mainly Scandinavian. 12.5 to 15.5mm in thickness standard and high density boards.

d) Tempered Hardboard: Scandinavian origin 4.8 to 6.4mm in thickness.

e) Chipboard: European origin 12.0mm in thickness Type 1 and Type III grades.

f) Flakeboard: European origin 5.0 to 12.0mm in thickness.

g) Cement Bonded Particle Boards and Building Boards: European origin different manufacturers. 3.5 to 12.5mm in thickness Variations in constituent materials and quality.

h) Plasterboard: British origin 9.5 and 12.5mm in thickness Standard and moisture resistant grades.

Currently they represent the boards most commonly used in timber frame construction and others which are actively being promoted for use.

The results for the individual board types are recorded in Tables 6.1 to 6.6. The results have been subdivided into similar groups of test, e.g. standard materials, variation
in nail size, etc., and also include single panel tests. The information given in the tables includes the following:

(i) full details of the construction, i.e. sheathing, frame, fixings and special details where the panel is at variance from that described in Chapter 5,

(ii) test details, the number of panels tested and the vertical loads,

(iii) test results, the averaged first cycle load for the similar tests and the failure loads,

(iv) the intermediate values in the reduction method, viz. the stiffness test load and the strength test load,

(v) the design values, viz. the racking load, the racking resistance and the basic racking resistance (the single figure value used in conjunction with accepted vertical load modification factors).

The results are discussed separately for the principal board types in the following sections. The design values are then analysed considering all the appropriate results for each sheathing type together.

6.3 DISCUSSION OF RESULTS

6.3.1 Plywood

The standard use of plywood is considered to be - 9.5mm thick board fixed with 3.25mm diameter nails. Differences in sheathing grade and country of origin have so little effect on performance that they can be omitted as design parameters. However, the range of boards tested is
limited and engineering judgement based on stress and modulus levels for different plywoods may be necessary to decide their suitability for use with the proposed design values. The standard usage results (P4 to P9) are very varied depending on the stud material, however, they effectively bracket other test results notably those of COFI in Vancouver (Parasin, 1980). The results were the first indication of the importance of the frame material to racking resistance; a factor which had previously been ignored in all investigations and will be considered in greater detail later in this section.

The design values for plywood are based heavily on panel stiffness. The reason being that the board itself is very unlikely to fail due to the lay up of the plies. Failure is due mainly to nail withdrawal by which time the panel has achieved an adequate factor of safety over the stiffness load. Consequently if deflection limitations were relaxed the design load for plywood could be increased.

In view of the importance of plywood as a sheathing material the secondary tests on standard 2.4m long plain panels are comprehensive and cover combinations of boards and all the important material variables. The horizontally laid sheathing tests are unique to plywood and were carried out to model standard North American sheathing practice. They indicate firstly that if a full size sheathing board is fixed all around its perimeter its orientation has little significance and secondly that loss of structural continuity greatly reduces the performance of panels under high vertical loads.

6.3.2 Mediumboard

The mediumboard tests extend over a much longer period than those on plywood, although the results are more limited as less specialist cases have been examined. Much of the project work covered later in the chapter has been carried out using mediumboard thus it is probably the most tested and best understood of the sheathing materials. The standard tests are covered by results M1 to M8 (Table 6.2a).
Not all the standard tests have been included and results have been omitted where they were similar to an earlier test, or they did not represent an extreme, either in value or type of material. The results include all changes made in frame and fixings. They also include a range of nail types, but as the diameter is constant and penetration is adequate, direct comparison is acceptable. Framing material has varied through the tests although no results are available for the very low density timber noted during the plywood tests. The sheathing boards have been subject to change and thus the results may be considered representative of HM grade medium density fibreboard or mediumboard as it is now termed.

The standard use of mediumboard combines a 9.0mm thickness of board with 3.25mm diameter nails at 150/300mm centres. Noting the fixings to be identical, the standard thickness board has very similar design values to those of plywood so that the two boards are well suited to being linked such that they will be the lower bound to the strong Category 1 sheathing group.

A closer examination of the mediumboard and plywood results shows the former to be initially more stiff but to fail in a more brittle manner, thus in general, mediumboard results are based on a failure performance and plywood results on stiffness. This behaviour is a result of the higher density of the mediumboard and the short, randomly laid up board fibres compared with the oriented plies of the plywood. Failure in mediumboard is always due to the edgefailure of the board about the perimeter nails when a vee shaped wedge of board is sheared by the nails at the leading end of the bottom rail joint where the stress perpendicular to the edge is greatest. Mediumboard results are less variable than those of plywood but in part this is because the tests on the low density frames did not include mediumboard.
6.3.3 Bitumen Impregnated Insulation Board

Bitumen impregnated insulation board (BIIB) is a much less dense and therefore weaker board than all the other sheathing materials. In its standard use, a normal density (270 kg/m³) board approximately 12.5mm thick is combined with a 3.25mm diameter fixing spaced at 75/150mm centres. Many insulation boards, mainly of Scandinavian origin, have been tested using different types of nail and frame material. The results are rather variable but, due to the inherent weakness of the board this is due mainly to variations in board manufacture rather than fixings or frame material. The average performance, even with the closer nail spacing, is very much lower than that of plywood and mediumboard and, because BIIB is widely used in the timber frame industry, is ideally suited to being the basis for the lower bound of the weak Category 2 sheathing group. Boards weaker than normal density BIIB may be taken to be of little commercial use as sheathings.

Test variables are limited in that no condition weaker than the standard use has been tested. BIIB requires, however, two extra variables to be considered, firstly density of sheathing and secondly the waterproofing constituent of the board. Density is seen to greatly affect performance but, because the density is nowhere near as high as mediumboard, the results do not approach the strong category of sheathing. The waterproofing compound had little effect on performance such that the term BIIB can be extended for racking resistance to cover other forms of insulation board such as resin bonded. In fact the latter board was slightly stronger which is probably due to a better bond being achieved between the resin coated fibres.

The weakness of the BIIB affects its behaviour. Firstly there is noticeably less improvement in racking resistance with vertical load and secondly while stiffness governs design at high vertical loads strength is critical at zero vertical load.
BIIB has not been tested in combination with other boards except plasterboard. This is not a practical limitation except in assessing some Scandinavian constructions which combine BIIB with an internal mediumboard lining.

6.3.4 Tempered Hardboard, Chipboard and Waferboard.

Table 6.4 shows design values for the other types of board tested during the current investigation which would normally be considered as strong sheathing materials and placed in the same category as plywood and mediumboard.

The hardboard tests are all dated and reflect the limited use now made of hardboard due to its density (which makes nail driving difficult) and its thinness (which causes worries about board buckling between studs). The panels were constructed using high quality hem-fir frames and hand driven fixings. Throughout the tests racking resistance was high and for the normal use of the board (thickness 6.0mm with 3.25mm diameter fixings at 150/300mm centres) was 20% above the standard plywood results. Thus tempered hardboard may be classified as a category 1 material.

The chipboard results cover three programmes; two on standard type 1 board and one on type III moisture resistant board. Thickness and fixings were standardised at 12mm and 3.25mm diameter nails at 150/300mm centres respectively. Results C3 show the Type III board to be suitable for inclusion with plywood and mediumboard as a Category 1 sheathing and the board has also been deemed suitable for inclusion in BS 5268 part 6 based on durability considerations. The type 1 board is weaker than the type III but is just adequate for Category 1 status, however at present, durability requirements do not allow this board to be promoted as an externally fixed structural sheathing.

In general chipboard was rather brittle in its behaviour such that failure performance governed the design value.
The waferboard tests (F1 to F4) comprise four different thicknesses of urea glued flakeboard from a single manufacturer. The earlier tests on 5.0 and 6.0mm thick boards produced higher levels of performance than the later tests on 9.0 and 12.0mm boards. This apparent discrepancy must have resulted from the differences in the framing material because other tests have shown little difference between the two types of nails used in the tests. All the boards exceeded the requirements for the strong sheathing category and the initial tests indicated the flakeboard to be equivalent to tempered hardboard in terms of racking resistance. The flakeboards tested have not been included in the Code due to durability worries concerning the urea based glue. Many waferboards are, however, moisture resistant and it is probable that they could be accepted immediately into the plywood category. A suitable datum size for the board would be 9.00mm to match the plywood and mediumboard with 3.25mm diameter nails at 150/300mm centres. The larger flakes used in waferboard improve the edge strength of the board so that it is much less brittle than the chipboard. It is assumed that oriented strand board would behave in much the same way as waferboard.

The only material variable covered in this series of tests was board thickness and here only the tempered hardboard results can be included in the thickness factor analysis because of the problems already noted with the flakeboard results.

6.3.5 Plasterboard

The draft for publication of BS 5268 part 6 (BSI, to be published) treats plasterboard differently in comparison with all other cladding boards both in design and in testing. The differences in wall design have been noted in Chapter 4. In standard panel design a minimum nailing standard is applied and no enhancement is allowed for improvements to this datum. The smallest acceptable plasterboard nail is that recommended by British Gypsum, a 40mm long 2.65mm diameter hand driven hot dip galvanised clout head nail, and the maximum spacing for structural use is 150mm. The differences in test requirements
relate to the factor of safety that is applied to the failure load. In the case of plasterboard this must be increased from 1.6 to 2.4 (Chapter 5). The last change is very recent and is only meant for use with new or specialist tests. The tests on plasterboard carried out during this investigation (see Table 6.5) have been reduced in the normal manner and so can be compared directly with the other sheathing boards. The basic design values in the Code have been based on these figures and therefore include a factor of safety of 1.6 (in practice the overall safety factor is 2.0 as the partial safety factor for similar tests is 0.8 in all cases). The brittle nature of the plasterboard means that in every case design is governed by the failure strength, thus if the test factor of safety had been applied the design load would have been reduced by 33%.

The standard thickness of the board is taken to be 12.5mm but thickness is of less consequence in plasterboard as strength is gained, for the most part, from the thickness of the lining paper and its bond with the gypsum. For a similar reason no difference is anticipated between the standard board and the moisture resistant board.

The behaviour of plasterboard sheathed panels is greatly influenced by the brittle nature of the board which results in very low factors of safety being achieved by the failure loads at zero vertical load. Factor of safety increases with vertical load but failure load continues to govern design. The variation in factor of safety affects both the basic racking resistance values for the board and the accuracy of the vertical load factor. Because of the special nature of the board the design values will be considered separately when the points noted above will be analysed quantitatively.

Few tests have been carried out varying the normal material parameters, however, new factors have been considered, firstly gluing the boards to the framework and secondly taping the central board joint to achieve a continuous 2.4m square sheathing.
Tests G7 and G8 cover the former case. PVA glue was used to attach the boards in addition to the standard nailing. The G7 results are not reliable as one panel was excessively stiff and strong in comparison with the second which behaved very like the horizontally sheathed panels of Test G8. It is thought that the design values for the two tests should be similar as in the G8 tests there was little difference between the panels with and without horizontal noggins. When the panels were dismantled it became evident that the quantity of glue used was critical, as it affected the penetration into the paper lining of the plasterboard which in turn affected the strength of the joint. A thin glue line resulted in a rolling shear failure within the lay up of the paper, whereas a thick glue line penetrated the paper directly bonding the plaster to the framework.

The glued panel results are not important to the overall consideration of plasterboard. The results show some improvement on panels where nailing is the only fixing, but are too dependent on the quality of gluing which may be difficult to control and monitor on site. The results, however, are the only ones included in this thesis which cover gluing and do show the advantages that could be obtained if the glue was used with a primary sheathing and was applied under carefully controlled factory conditions. It should be noted that the improvement resulting from the gluing will probably reduce with stronger sheathings and that different types of failure, particularly delamination in medium density fibreboards, could also reduce the benefit particularly under cyclic load conditions.

Result G9 shows the effect of taping the central joint between the boards which effectively converts the sheathings into a single 2.4m square board as no movement between the boards was detectable during testing. The improvement in performance increased from 6% at zero vertical load to 16% at 5kN/stud. It is noticeable during failure tests on standard panels at zero vertical load that there is little relative movement between the boards and so it
is to be expected that the difference in performance would be small. The taped joint wall is more representative of standard practice and an argument could therefore be made for using the result in determining the basic racking resistance of a single lining. However, there is no evidence at present of the behaviour of tape jointed boards over long lengths. This could differ markedly from that of standard panels and it would be unwise to attempt to justify any increase on the basis of the standard panel test alone.

Plasterboard has been tested in combination with the main sheathing materials, noting that if it is to contribute to racking resistance then in external walls it will always be providing additional resistance to that of the standard sheathing. Thus the design value will be determined from the equation:

\[
\text{Design contribution} = \text{Design value for plasterboard} - \text{Design value for sheathing plus lining} - \text{Design value for sheathing alone}
\]

and not from the design value for the plasterboard on its own. Plasterboard is also used in internal walls and separating walls when further design methods will have to be considered. In the former case the contribution of the second board can be determined if the behaviour of the combined linings and single lining are known and in the latter case the wall is a special construction from which the basic racking resistance can be calculated. The design approach for combined panels which effects the use of plasterboard in particular is covered in detail in section 6.4.

6.3.6 Building Boards

Results 01 to 06 (Table 6.6) cover three different types of building board which are all solid, strong and suitable for use as either a sheathing or, if of adequate thickness, as a lining. The boards are all similar to plasterboard in their brittle behaviour and susceptibility to damage from close to edge nailing. However, unlike
plasterboard, the damage caused can be seen because there is no cardboard covering to afford visual protection. In general the thicker the board the greater the problem of edge nailing and in every test a recommendation could be made that the holes be predrilled, particularly when joining two sheathings on a single stud. Here standard nailing would not be practical in mass production and, in general, problems with fixing may preclude these boards from commercial viability.

Table 6.6 shows the results when a 1.6 factor of safety is used. The boards would then all qualify as weak category sheathings. However, as the Code of Practice now requires a 2.4 factor of safety to be used, and all designs are governed by failure, then both design and basic racking resistance values should be reduced by 33%. Only the 12mm cement bonded particle board would qualify directly. It is probable that all the boards could achieve this performance level if their thicknesses were increased. To be considered as linings a minimum thickness of 12mm would be necessary to span studs at 600mm centres, then cement bonded particle board is noticeably stronger than plasterboard and could provide a suitable alternative.

In general, special care is needed with all building boards, including plasterboard, because of their inherent brittleness.

6.4. BASIC RACKING RESISTANCE FOR THE STANDARD USE OF THE SHEATHINGS IN TESTS

6.4.1 Single Sheathings

The first stage in determining design values for an empirical procedure such as that outlined in Chapter 4 is to define a standard use for each type of sheathing when used on its own to provide racking resistance. The standard panel, showing layout of frame timber and sheathing boards, was noted in Chapter 5. The quality of the frame material has had to be taken as the average quality used in the tests,
this wrongly assumes the frame timber to be of little importance but does not affect the results because, in general, the timber has been of consistent quality and typical of that used in practice. For simplicity factors such as board thickness, fixing size and fixing spacing should be identical for each board so that only the board material is compared. This is not practically possible due to the differences in standard thickness of the boards and their level of performance. Instead it is necessary to define the standard use of the board and then relate modification factors for board thickness etc. to percentage changes from the standard situation. The standard uses of the boards tested have been defined for generic types in section 6.2; they are:

<table>
<thead>
<tr>
<th>Board Type</th>
<th>Thickness</th>
<th>Nail Size</th>
<th>Nail Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>(i) Plywood</td>
<td>9.5mm</td>
<td>3.25mm dia.</td>
<td>150/300</td>
</tr>
<tr>
<td>(ii) Mediumboard</td>
<td>9.0mm</td>
<td>3.25mm dia.</td>
<td>150/300</td>
</tr>
<tr>
<td>(iii) BIIB</td>
<td>12.5mm</td>
<td>3.25mm dia.</td>
<td>75/150</td>
</tr>
<tr>
<td>(iv) Plasterboard</td>
<td>12.5mm</td>
<td>2.65mm dia.</td>
<td>150/300</td>
</tr>
<tr>
<td>(v) Tempered Hardboard</td>
<td>6.0mm</td>
<td>3.25mm dia.</td>
<td>150/300</td>
</tr>
<tr>
<td>(vi) Chipboard</td>
<td>12.0mm</td>
<td>3.25mm dia.</td>
<td>150/300</td>
</tr>
<tr>
<td>(vii) Flakeboard</td>
<td>9.0mm</td>
<td>3.25mm dia.</td>
<td>150/300</td>
</tr>
</tbody>
</table>

It can be seen that there are only two fixing variations from the norm. Firstly the nail size for plasterboard; this is fixed at 2.65mm with no allowance for larger diameters. It will therefore be possible to relate nail size modification factors to 3.25mm nails only. Secondly for nail spacing the weak BIIB sheathing requires the standard nail spacing to be halved to achieve a reasonable performance level. It will therefore be possible to relate the nail spacing factor to a standard spacing of 150/300mm but to note that for BIIB (and any other similar boards) the spacings must be halved. It must be noted that in practice the spacing of nails in plasterboard on internal studs will be reduced from 300mm to 150mm, however this will not affect the design values.

The final factor necessary in defining a single design value for each board is the vertical load. The basic racking
resistance at zero vertical load has been defined in Chapter 5 but in order to evaluate it from the test results standard vertical load factors must first be determined. They can be calculated for the 2½ and 5kN/stud test cases by dividing the design resistance at these vertical loads by the design resistance at zero vertical load. Table 6.7 shows the averaged values firstly for the groups of standard tests on each board type and secondly for non standard tests for board types where only board thickness or nail size has been altered. The results are uniform with the exception of plasterboard which, due to its brittle nature, had a very low zero vertical load failure affecting both vertical load factors. The average values have therefore been calculated without the plasterboard figures. They are seen to be very similar to those included in the draft Code of Practice (BSI, to be published) which are based on figures first proposed in 1978 (Griffiths 1978 b). No change is therefore proposed to the vertical load factors for standard panels shown as Modification Factor K302 in Table 5.2 and they will be used firstly in the calculation of basic racking resistance (BRR) from the test values and secondly in calculating design values at other vertical loads for standard panels using BRR. This result confirms the design values given in the last column of Tables 6.1 to 6.6. The accuracy of the vertical load factor is important to the efficiency when determining the basic racking resistance from test results. The method of calculation (Chapter 5) ensures adequate safety but if the test results exhibit lower vertical load factors than K302 then the low vertical load racking resistance will be underestimated whereas if they are higher, as in the case of plasterboard, the high vertical load case will be underestimated.

The test results can now be used to determine the design value of BRR for the standard test use of plywood, mediumboard, BIIB and plasterboard. In each case the results may be meaned over a number of sets of data all based on two panel tests. Two methods have been used to determine the mean values. The first assumes all the tests to be of separate identity and averages the design racking load for each vertical
load condition. Thus the results include the 0.87 partial safety factor for stiffness and the 0.8 factor for strength. The second method considers all the tests as one group, which enables a partial safety factor of unity to be used but necessitates the use of the lowest failure results which could prove critical to design. For both reduction methods the basic racking resistance is calculated and the test values for the vertical load modification factor. The results for the four boards are shown in Tables 6.8 to 6.11 together with the design values quoted in the draft for publication of BS 5268 part 6 (BSI, to be published). In assessing a suitable design value for BRR the following points were considered:

(i) it should not exceed either of the meaned results,

(ii) it should not exceed the lowest test result by more than 15%,

(iii) plywood and mediumboard are so similar that they should be classified together and, being the most common types of strong sheathing, they should represent the lower bound value for such boards (thereby achieving maximum efficiency in practice),

(iv) BIIB and plasterboard could also be classified together in order to reduce the number of design values to a minimum,

(v) the final design values, the datum racking resistance (DRR) will have to be based on 3.0mm diameter nails because the tested nails are non standard in relation to BS 5268 part 2, this will not affect the plywood and mediumboard category but will affect that for BIIB
and plasterboard because, while reducing the load for BIIB, the values for plasterboard will remain unchanged since they are fixed for the standard 2.65mm diameter plasterboard nail.

The basic racking resistances determined for the boards, as tested, were:

(i) plywood and mediumboard  -  1.82 kN/m,
(ii) BIIB  -  0.98 kN/m,
(iii) Plasterboard  -  0.90 kN/m.

The values may be judged for each board separately. The plywood figure is that used in the draft Code of Practice; it represents an 8% reduction on the "meaned pairs" performance. It infringes slightly the second requirement noted above; if the weakest set of test data is used with safety factors of unity to allow direct comparison then racking loads of 4.47, 6.15 and 7.75 kN are obtained, all based on stiffness results. This can be accepted because the failure performance, and thus overall safety, is adequate and the frames were particularly weak and would probably have been deemed unsuitable if a regulation on density of timber had been included.

The mediumboard value was chosen to be identical to plywood and is also used by the draft Code of Practice. The safety margin is much greater because overall the mediumboard results were 4% higher than plywood. However, because mediumboard was not tested on weak frames, the design value is realistic.

The BIIB value is reduced from the draft Code figure which is seen to be unsafe in Table 6.10. The BRR figures based on both averaging methods are low due to one result; the 5kN/stud performance in Test B3. The 0.98 kN/m design value is a compromise which achieves the same reduction on the "meaned pairs" performance as the plywood but cannot
accommodate the B3 results when matched with partial safety factors of unity. The design value is safe for use with resin bonded boards as well as bitumen impregnated.

The plasterboard results included the four results G1 to G4 and ignored effects of nail size and board thickness. The latter, as previously noted, is unlikely to have affected performance but it is likely that the use of 2.87mm nails did increase the performance of tests G1 to G3 in relation to that expected for the standard 2.65mm diameter nail. The BRR value of 0.9 kN/m was chosen to be the same as BIIB when used with 3.0mm diameter nails. It is perfectly safe in comparison with the results reduced using the 1.6 safety factor but is unsatisfactory if the safety factor is increased to 2.4. Fortunately the latter factor is for testing only to ensure that in future tests standard design values which have been carefully appraised will not be exceeded. The change in requirement relates only to plasterboard due to its brittleness and it being used traditionally as a non-structural material. It should be noted that the BRR value is governed by the weak zero vertical load performance so that designs at higher vertical loads using BRR will have increased safety margins, however, plasterboard is normally used on internal walls where the vertical load will be very low.

The other sheathing materials have not had their results analysed in the same way because of limited data. From a design standpoint they only need to be classified into either the strong or the weak category. It is clear from the test results that the standard thicknesses of tempered hardboard, chipboard and flakeboard all achieve strong category status but that high density BIIB and all building boards must be classified as weak. Where the sheathing shows a marked improvement over the standard values for the category to which it is assigned, design may be based directly on test data to achieve maximum efficiency but this will restrict its use because materials modification factors should not be used in conjunction with tested BRR values.
6.4.2 Datum Racking Resistance

The standard tests on the wood based sheathings have all used 3.25mm diameter nails which are a non standard size according to BS 5268 part 2. It has been decided, therefore, to quote the principal design value, the datum racking resistance (DRR), for 3.0mm diameter nails which conveniently interpolates the two nail sizes commonly used in testing. In order to calculate DRR using the value of BRR for the standard test, it is necessary to use a modification factor for nail diameter (K101). This should be determined from tests on the main sheathings but only if all other factors are identical. The results are shown in Table 6.12 and Figure 6.1 but are not conclusive due to the limited number of tests. It must be noted that variability in identical panel performance is normally ±5% and can be as much as ±10% about the mean whereas the change in nail size is only -12%. Furthermore the comparative tests were often carried out at different times so that it is unlikely that the frame timber was consistent. Theoretically the predominant factor in nail design should be bearing between the nail and the sheathing so that racking resistance is proportional to the diameter of the nail and the thickness of the board. This hypothesis assumes racking resistance to be based on stiffness because failure performance could be related to nail withdrawal, nail bending or board failure and could be independent of nail size. With no detailed evidence to the contrary it is proposed that panel performance be directly related to nail diameter such that the nail modification factor K101 is given by:

\[
K101 = \frac{\text{Proposed nail diameter}}{\text{Standard diameter}}
\]

This relationship is shown in Figure 6.1 and can be seen to deal adequately with the test data.

The datum racking resistances can now be calculated by applying a factor of 3.00/3.25 to the tested values of BRR. Thus the datum racking resistances for the principal sheathing
materials (i.e. for use with 3.00mm diameter nails) are:

(i) Category 1 strong sheathings, including 9.5mm plywood and 9.0mm mediumboard, 
    DRR = 1.68 kN/m

(ii) Category 2 weak sheathings, including 12.5mm BIIB, DRR = 0.90 kN/m.

The DRR value for plasterboard will also be 0.90 kN/m but this value refers to the use of 2.65mm diameter plasterboard nails. The values are tabulated in Table 6.13.

6.4.3 Combined Sheathings

The combined sheathing case covers the standard wall where the timber frame is sheathed (or lined) on both sides. It is immediately clear from the test results that the racking resistance cannot be taken as the sum of the parts. Furthermore as the zero vertical load results show a similar improvement to those at 5 kN/stud it is not possible to design the wall as the sum of the two sheathing cases when the vertical load is divided between the two parts. The behaviour (see Chapter 6 Section D) is more complex if the two boards are of different material which is typical of external walls where the brittle plasterboard is combined with a wood based sheathing which is more ductile in its load resistance. The present method of wall design (BSI, draft to be published) allows only a partial contribution from some boards. It is therefore advisable to analyse the combined sheathing case as the principal sheathing carrying its full load plus the contribution from the secondary board. Hence

\[
\text{Combined Sheathing Resistance (A)} = \text{Primary Sheathing Resistance (B)} + \text{Secondary Sheathing Resistance (C)}
\]

A and B can be determined by test allowing C to be calculated. The number of combinations available are many, although the majority will not be practical, and the test data is limited;
it is proposed therefore to deal with combinations using the two categories of sheathing and considering plasterboard to be a weak category board. Then the only combinations are:

1. **category 1 + category 1**
   
e.g. ply + ply which is occasionally used to strengthen walls with a large proportion of openings,

2. **category 1 + category 2**
   
e.g. ply + plasterboard, a typical external wall,

3. **category 2 + category 2**
   
e.g. BIIB + plasterboard, a typical external wall or plasterboard both sides, an internal wall,

4. **category 2 + category 1**
   
e.g. BIIB + mediumboard, often used in Scandinavia, but for design purposes identical to (ii) above.

The test results for board combinations are detailed in Table 6.14 but are not conclusive. Thus a design method was proposed such that the performance of the secondary board was a fixed percentage of its basic racking resistance dependent on the category of the principal board. The factors were selected to give a conservative assessment of the combined panel, they are:

1. **50%** where the category of the secondary board is the same as the principal board

2. **30%** where the secondary board is in a weaker category.

To avoid non standard calculations when joining a strong secondary board to a weak principal board, it is necessary to
define the principal board, as either:

(i) the sheathing material in a sheathing/lining combination or

(ii) the stronger board in combinations of sheathings (or linings).

The application of these factors for datum racking resistance combinations is shown in Table 6.13. Using this method it is possible to adjust each board's performance by the materials modification factors appropriate to its use. Table 6.13 values, together with the nail size factor, have been used to determine design values for basic racking resistance of the combination (i.e. BRR of principal board + BRR of secondary board) for all the test cases. The design values are then compared with the test values and the same comparison is made for the principal board in the combination, the results are shown in Table 6.15. In theory the percentage improvement of the combination should be similar to that of the principal board where the principal sheathing is a category 1 material this is seen to be true, the improvements are high but this is because the combined panel tests were carried out on relatively strong frames. In general the weak category combinations could accept a higher percentage improvement factor except for the plasterboard combination at zero vertical load (see Table 6.14). Because this is the most likely use of weak combination boards and for the sake of uniformity the 50% factor is retained and can be seen in Table 6.15 to provide safe results.

A different assessment of plasterboard and sheathing combinations is shown in Table 6.16. Here the plasterboard is taken as the primary board and the improvement gained from the sheathing related to the plasterboard performance is measured. Apart from the very brittle combination of two plasterboard sheathings at low loads the uniformity of category 1 and 2 materials is notable. Unfortunately the method is not suitable for design purposes because:
(i) plasterboard is structurally unsuited for use as the primary sheathing

(ii) the large improvement factors make the use of material modification factors on the sheathings more significant.

It is of note that the improvement factor increases at a lesser rate than the basic racking resistance (i.e. 73% for 0.98 kN/m and 112% for 1.82 kN/m). This indicates that the stronger the sheathing the less the effect of the plasterboard when used in combination. Returning therefore to the design method, care is needed if the basic racking resistance of the sheathing material is much greater than its datum racking resistance due either to the use of material modification factors or the substitution of test data. It is then possible that the 0.28 kN/m performance level for the plasterboard will be compromised. This can be avoided by the inclusion of two design rules viz:

(i) secondary board values may only be used if the basic racking resistance of the principal sheathing is not greater than a given value, say 2.50 kN/m

(ii) secondary board values may not be used with BRR values determined by test unless the combined sheathing case has also been tested and can be shown to be adequate.

Two further special forms of sheathing combinations should also be considered, both relate to two sheathings on the same face of the frame. The first case has a common use in separating walls where the internal boards are replaced by horizontal planking and here test information is available (G6). In the second case each sheathing is attached
separately using the standard nailing pattern such that in theory the internal boards would be doubled nailed and thus of higher strength however the secondary board has then been displaced from the frame by the thickness of the internal board and the resultant flexibility in the nails may not allow its full strength as a secondary sheathing to be motivated. To compensate for this it is suggested that design is based on the standard sheathing combination rules previously noted, i.e. no benefit is given to the extra nails in the internal board but the external panel is allowed the full contribution of a secondary sheathing. It will be seen (Section 6.4.4) that the performance will exceed that of the internal sheathing designed alone for double nailing. A third board could in theory be considered as an additional secondary material but here it is essential that the first design rule (noted above) concerning the basic racking resistance of the, now combined, principal sheathing, is applied, e.g. a panel comprising two layers of plywood each fixed with 3.25mm diameter nails at 150/300 centres on the external face of the frame when combined with an internal plasterboard lining would have a theoretical resistance of 3.0 kN/m \((1.5 \times 1.82 + 0.27)\) in practice this should be limited to 2.77 kN/m (the greater of 1.5 x 1.82 and 2.5 + 0.27).

Returning to the first case, the horizontal planks are not supported along their horizontal edges and thus provide little racking resistance themselves whilst reducing the benefit of the main sheathing boards by separating them from the frame. The test results may be used to show that a suitable DRR value for this combination would be approximately 1.25 kN/m (here the test results and the design values are compared with the standard combination of two plasterboards, G5). In the draft Code a further factor of safety has been applied to this form of construction, because it may be used as a total replacement for a wood based sheathing board, and it is given the same DRR value as standard plasterboard i.e. 0.9 kN/m. The merits of this extra safety factor will be discussed in Chapter 8.
It is noted that the values now included in Table 6.13 differ from those in the draft Code of Practice (BSI, to be published). The values for BIIB have been reduced to bring the overall factor of safety for their use more in line with those applied to plywood and mediumboard. The method of dealing with secondary sheathing has been substantially altered and the contribution of the secondary board reduced by approximately 15% for similar category materials.

6.4.4 Materials Modification Factors

The datum racking resistance values evaluated in the previous section may be multiplied by materials modification factors to give basic racking resistance values for plain 2.4m square wall panels. The test results may be used to identify the following factors:

(i) nail size \( K_{101} \),

(ii) nail spacing \( K_{102} \),

(iii) board thickness \( K_{103} \),

(iv) board orientation \( K_{104} \).

The factors should be independent of other factors, in particular the board material, although it is possible to exclude their use with certain materials. Test data is limited but is normally spread over a range of materials, although some extrapolation of results is possible it will normally be necessary to put limitations on the use of the factors. Materials modification factors covering the frame timber have not been covered in this investigation but their use will also be discussed. Finally, the vertical load modification factors noted in the calculation of datum racking resistance have not been classified with materials and will be examined in Section B.
a) **Nail Diameter Modification Factor**

This factor has already been covered in Section 6.4.2 noting the factor to vary linearly with nail diameter. Because the DRR value relates to a nail diameter of 3.0mm the modification factor can be written as:

\[ K_{101} = \frac{\text{Proposed nail diameter (mm)}}{3.0} - 6.2 \]

Although tests were only carried out on 2.87 and 3.25mm diameter nails it is thought that the factor can safely be extrapolated to cover nails between 2.25 and 3.75mm diameter (±25%). The factor should only be applied to wood based sheathing materials. Design values for plasterboard are based on 2.65mm diameter nails, no enhancement is allowed for larger diameter, and smaller diameter nails are not acceptable.

The length of nail is determined by the thickness of the board and depth of penetration in the frame wood required for the nail diameter based on guidelines given in BS 5268 part 2 (BSI, 1984).

The size of the nail head has not been covered in detail. In practical terms it should be as large as possible, noting the requirement for gun nailing, especially in weak sheathing boards like BIIB. In most cases the underside of the nail head should be driven flush with the board face but in the case of BIIB and plasterboard the nail head should just penetrate the board.

b) **Nail Spacing Modification Factor**

The results of the limited number of nail spacing tests are detailed in Table 6.17. The improvements when halving
the spacing for plywood and mediumboard are seen to be very low (approximately 26%) however the datum values for the standard test were very high and compared with the average basic resistance values for the boards the improvements were 44% for plywood and 37% for mediumboard. Compared with the basic racking resistance value for the standard spacing the improvement in both cases was 56%. For BIIB the improvements on the average test results and the BRR value were 63% and 66% respectively for a spacing reduction of 67%.

The nail spacing variation factor used in the draft version of BS 5268 part 6 (BSI, to be published) had been derived from information presented by Robertson and Griffiths (1981) and TRADA (undated and 1980 b). The TRADA values indicated a substantially increased enhancement factor of 67% for half spacing and although the basis for this figure was unreliable its value had been widely used and would therefore strongly influence any successive design value. Robertson and Griffiths' survey of previous work indicated large differences for BIIB but much smaller changes for category 1 boards. In the draft for BS 5268 part 6 Griffiths proposed a design solution based on the parameter 'A' which related the actual spacing to the normal spacing such that:

\[ A = \frac{\text{Proposed nail spacing around perimeter}}{\text{Standard nail spacing around perimeter}} - 6.3 \]

The modification factor was determined by the equation:

\[ K_{102} = \frac{1}{xA + (1-x)} - 6.4 \]

which best suited the trend of the available data. An intermediate solution was obtained taking the value of 'x' to be 0.67 when the factor for halving the nail spacing was 1.5. These results are all shown in Figure 6.2. The Code modification factor would therefore just be acceptable for the test results in the current investigation when used with the datum racking resistances.
However, it is considered that the factor is too generous based on the current work, the Robertson and Griffiths report and the doubts surrounding the TRADA figures. The value of 'x' in equation 6.4 has therefore been changed to match the improvement factors for the category 1 boards based on the averaged test results such that the nail spacing modification factor is given by

\[ K_{102} = \frac{1}{(0.6 A + 0.4)} \quad -6.5 \]

Use of this factor gives a BRR value of 2.6 kN/m for the test panel, underestimating its performance by 9%.

Further reduction of the value of 'x' of 0.5 gives a design value of 2.4 kN/m achieving a 15% safety factor. However, this improvement in safety for a nail spacing closer than standard is gained at a cost of reduced safety for wider spaced nails.

In view of the lack of reliable test data and the nature of equation 6.4, which increases the relative strength as 'A' diverges from the datum spacing it is proposed that the equation be used only if 'A' lies between 0.5 and 2.0 and that further consideration be given to the value of 'x' based on the results of the computer analysis (Chapter 7) or future test work.

c) **Board Thickness Modification Factor**

Theoretically if board performance is governed by either nail bearing or edge failure of nails then it should be directly related to board thickness. However as board performance increases, nail and frame behaviour become more influential in the racking capacity of the panel. The draft Code noted these facts in a conservative estimate of the modification factor whereby BRR reduced in direct proportion to the loss of standard board thickness but no enhancement
was allowed for thicknesses greater than the standard value. The results of tests carried out in the current investigation (see Table 6.18) showed that the proportional reduction was the correct lower bound solution for these boards but that an enhancement of 15% in performance could be accepted for a 30% increase in standard board thickness (Figure 6.3).

An equation has been derived based on a factor $B$ where:

$$B = \frac{\text{Proposed board thickness}}{\text{Standard board thickness}} - 6.6$$

to meet these requirements, such that:

$$K_{103} = 2.8B - B^2 - 0.8 - 6.7$$

This equation is safe within the range of the boards tested and will give increased safety if the results are extrapolated in either direction. However the nature of the equation suggests that the limits for its use should be $0.7 < B < 1.3$. For simplicity and to avoid confusion the board thicknesses used to determine the value of $B$ should be the nominal thickness and not the minimum thickness as noted in the draft Code.

d) Board Orientation Modification Factor

The board orientation factor is not included in the draft Code and has been introduced on the basis of the plywood results alone where two thicknesses of board (7.5 and 9.5mm) were tested. The results (P16 to P19) cannot be directly compared with vertical board tests due to a change in nail size when using the 7.5mm board and a different frame material (although of similar grade and species) for the 9.5mm board. The results are detailed in Table 6.19 making adjustment for the change in nail size. They show the horizontal unblocked sheathing to be so weak in shear resistance, compared with its resistance to rotational uplift, that very little advantage can be taken of vertical load. The blocked panel results are similar to those of the vertically
sheathed panels and for design purposes no differentiation need be made.

The board movements at failure under a 5kN/stud load are shown in Figure 6.4 when the difference in behaviour is at a maximum. Under the same condition a vertically clad panel, with a racking deflection of 70mm, would have uplifted approximately 30mm and the relative movement between boards on the centre stud would be approximately 12mm. These figures confirm the similarity in behaviour noted above.

For design purposes if a horizontal sheathing is laid on a standard frame with noggins such that all edges of the board can be supported and fixed then the board orientation factor will be given as:

\[ K_{104} = 1.0 \text{ (all edges fixed)} \]

If support is removed from the horizontal joint both the overall performance and the vertical load performance must be reduced. A suitable equation for the modification factor has been analysed as:

\[ K_{104} = 0.9 / K_{110}^{0.5} \]

where \( K_{110} \) is the vertical load modification factor. Using this formula the design of a wall with unsupported joints to horizontally clad panels can be treated identically to any other wall. The \( K_{104} \) factor should be limited to 1.2m wide horizontally laid boards or a single horizontal joint in a panel height and must not be applied to horizontal planks of limited width.

6.4.5 The Frame

Throughout the standard panel tests the results have been considered independently of the frame although the results, particularly those for plywood, have shown major
differences where the only variable has been the frame itself. Two factors need to be considered:

(i) the section size,

(ii) the material quality.

In general the size of the frame member should have little effect on racking resistance. The width of the section, 38mm, is necessary to allow two boards to be joined on one stud: no enhancement should be allowed for wider studs and joints should be prohibited on narrower versions, effectively requiring two studs at board joints which would themselves have to be effectively connected (see section 6.5). The 89mm depth of section is the minimum recommended for external walls and again no enhancement should be allowed for deeper sections. For internal partitions the depth could reduce to 72mm. No data are available for such frames, however, a reduction factor is not suggested for the following reasons:

(i) such frames will normally only be used with plasterboard sheathings which already include a higher safety factor,

(ii) they are likely to be used in non load bearing situations when the racking loads will be relatively small and the board is the weakest governing factor.

The material quality relates to its resistance to nail movements and not its species or grade as defined by the presence of defects or its structural capability as a beam. At present there is no accepted method of grading timber for use where the behaviour of the connectors is the governing criterion and therefore no attempt was made in the full scale tests to investigate quantitatively this form of variability. Clearly worries about differences in timber quality refer to mean values for different consignments rather than variability between individual pieces because
the timber frame wall is a true load sharing system (as distinct from the Code definition of load sharing) because the connecting elements, the sheathing boards, are strong in the direction of loading.

The tests show firstly that there are small differences between individual panels, but these are averaged by the test method and, due to the number of panels used in a building, the average performance level is appropriate. Secondly there can be major differences between stocks of material even though they could be similarly graded. More work is needed to find the property of the frame wood which causes this variation. If this was found to be, say, density then a $K_{105}$ modification factor could be included in the standard panel design. The present design method would benefit if limitations were put on timber density viz:

(i) a minimum density of say 425 kg/m$^3$, to which the datum racking resistance values could be applied,

(ii) a maximum density of say the true mean value for the grade of the species, for frame timber in standard panel tests to determine basic racking resistance values.

The effect of frame material is one of the few areas where further work could greatly benefit the timber frame designer particularly as the results would be applicable to all the design methods outlined in Chapter 4.

6.4.6 Summary

The design method has simplified the presentation of data by choosing two boundaries. The first, the lower limit for strong sheathing (category 1), is based on the materials most commonly used in timber frame walls viz 9.5mm
plywood and 9.0mm mediumboard (noting all thicknesses to be nominal) which are very similar in their tested racking resistances. The second is the lower limit for weak sheathings (category 2) and lining boards which is governed by 12.5mm BIIB and 12.5mm plasterboard. Other materials can be placed in these categories based on their test results and should include increased factors of safety. The materials modification factors are, with minor exceptions, adjusted so as to be identical for all categories. This procedure will simplify the design process and reduce confusion, but will make many results more conservative as the differences between board types mean the single relationship cannot be justified in terms of accuracy. One trend that has been noted throughout the tests is that response to improvement to a panel decreases as the datum racking resistance of the panel increases. This effect is illustrated in Figure 6.5 which shows a hypothetical response curve for racking resistance. The 'y' axis plots racking resistance and the 'x' axis allows the designer to determine the effect of the modification factor. The datum resistance is used to determine the standard ordinate for the modification factor. This is multiplied by the change in modification factor and using this new ordinate, the panel resistance can be determined from the response curve. The graph illustrates the effect of doubling the modification parameter, e.g. by halving the spacing of nails. The nature of the response curve means that the percentage improvement in racking resistance will be much higher for weak sheathings in comparison with stronger sheathings. As drawn, an improvement in parameter of four times would be necessary to gain the same percentage improvement in basic racking resistance for the stronger sheathed panel. The response curve is typical of behaviour noted in the analysis of the test results. However, each parameter would have a different curve and the procedure is not suitable for the empirical design method covered in this investigation.

To summarise the work on modification factors for standard panels a list of proposals is given in Table 6.20 noting changes from those included in the draft for publication.

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of BS 5268 part 6. Modification factors are applied independently of board type although this has been noted as a simplification introduced to reduce the complexity of the design procedure. In determining the factors, greatest importance has been placed on their use with the most commonly used sheathing, 9.5mm plywood. This often has the effect of making the design using weaker sheathings more conservative. The factors have been established from very limited test data and rely on theoretical consideration in some areas. The use of nails both in their size and spacing could benefit from further tests although it is considered that the great majority of likely design conditions have been covered in testing. The most significant area of improvement that could be made is in the assessment of the frame material and its effect on racking resistance.
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Table 6.1b Standard Panel Tests: Plywood Sheathing
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Table 6.1c Standard Panel Tests: Plywood Sheathing
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<tr>
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<td>Karlit</td>
<td>G/Hay</td>
<td>150/300mm ccs</td>
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<td>19.93</td>
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**AVERAGED RESULTS FOR TESTS M1 TO M3**

|        |                          |       |         | 16                          | 0             | 5.33                             | 11.49         | 4.98                | 2.68              | 2.02        | 2.08           | 2.08                  |

**Table 6.2a Standard Panel Tests : Mediumboard Sheathing**
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<th>FRAME</th>
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<th>VERTICAL LOAD (kN/Stand)</th>
<th>AVERAGED DEFL. LOAD (mm)</th>
<th>FAILURE LOAD (kN)</th>
<th>STIFFNESS TEST LOAD (kN)</th>
<th>STRENGTH LOAD (kN)</th>
<th>DESIGN LOAD (kN)</th>
<th>DESIGN RESISTANCE (kN/m)</th>
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<td>G/Vay</td>
<td>150/300mm ccs</td>
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<td>6.72</td>
<td>15.84</td>
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<td>150/300mm ccs</td>
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<td>75/150mm ccs</td>
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Table 6.2b Standard Panel Tests: Mediumboard Sheathing
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<th>VERTICAL LOAD</th>
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<th>FAILURE LOAD</th>
<th>STIFFNESS TEST LOAD</th>
<th>STRENGTH TEST LOAD</th>
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**AVERAGED RESULTS FOR TESTS 31 TO 35**

| 31  | 12.7mm B118       | SPF   | hand clouts| 2                        | 0             | 2.88          | 5.42                          | 2.61         | 1.09                | 1.02              | 1.02        |            |               |
|     | Swan Asphalt      | G/Nay | 75/150mm ccs| 5                        | 4.61          | 9.58          | 4.82                          | 1.88         | 1.79                |               |            |            |               |

Table 6.3a Standard Panel Tests: Bitumen Impregnated Insulating Board Sheathing
### Table 6.4 Standard Panel Tests: Other Category 1 Sheathings

#### a: Hardboard and Chipboard

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<th>FAILURE LOAD</th>
<th>STIFFNESS TEST LOAD</th>
<th>STRENGTH LOAD</th>
<th>DESIGN RESISTANCE</th>
<th>BASIC RESISTANCE</th>
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<td>Non-fir.</td>
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<td>3-12 Type I Flakeboard</td>
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#### b: Flakeboard

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<th>FAILURE LOAD</th>
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Table 6.4 Standard Panel Tests: Other Category 1 Sheathings

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Table 6.5 Standard Panel Tests : Plasterboard Sheathings

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</tr>
<tr>
<td></td>
<td></td>
<td>LOAD</td>
<td>Kg</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 6.6 Standard Panel Tests : Building Boards
Table 6.7 - Vertical Load Improvement Factors from Standard Panel Tests

<table>
<thead>
<tr>
<th>Method of Analysis</th>
<th>Vertical Load kN/stud</th>
<th>Design Racking Load kN</th>
<th>Design Racking Resistance kN/m</th>
<th>Design Racking Resistance kN/m</th>
<th>Vertical Load Improvement Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test results</td>
<td>0</td>
<td>4.99</td>
<td>2.08</td>
<td>1.98</td>
<td>1.00</td>
</tr>
<tr>
<td>taken in pairs</td>
<td>2.5</td>
<td>6.96</td>
<td>2.90</td>
<td>1.00</td>
<td>1.39</td>
</tr>
<tr>
<td>and averaged</td>
<td>5</td>
<td>8.52</td>
<td>3.55</td>
<td>1.00</td>
<td>1.71</td>
</tr>
<tr>
<td>All test results</td>
<td>0</td>
<td>5.51</td>
<td>2.30</td>
<td>2.30</td>
<td>1.00</td>
</tr>
<tr>
<td>grouped together</td>
<td>2.5</td>
<td>8.23</td>
<td>3.43</td>
<td>1.00</td>
<td>1.49</td>
</tr>
<tr>
<td>5</td>
<td>10.02</td>
<td>4.17</td>
<td>1.82</td>
<td>1.00</td>
<td>1.82</td>
</tr>
<tr>
<td>BS5268 values</td>
<td>0</td>
<td>4.37</td>
<td>1.82</td>
<td>1.82</td>
<td>1.00</td>
</tr>
<tr>
<td>with correction</td>
<td>2.5</td>
<td>6.24</td>
<td>2.60</td>
<td>1.00</td>
<td>1.43</td>
</tr>
<tr>
<td>for nail diameter</td>
<td>5</td>
<td>7.80</td>
<td>3.25</td>
<td>1.00</td>
<td>1.77</td>
</tr>
</tbody>
</table>

Table 6.8 Safe Design Values and Vertical Load Improvement Factors for 9.5mm Plywood with 3.25mm Diameter Nails.
<table>
<thead>
<tr>
<th>Method of Analysis</th>
<th>Vertical Load kN/stud</th>
<th>Design Racking Load kN</th>
<th>Design Racking Resistance kN/m</th>
<th>Design Racking Resistance kN/m</th>
<th>Vertical Load Improvement Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test results taken in pairs and meaned</td>
<td>0</td>
<td>4.98</td>
<td>2.08</td>
<td>2.08</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>7.23</td>
<td>3.01</td>
<td>1.45</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>8.94</td>
<td>3.72</td>
<td>1.80</td>
<td></td>
</tr>
<tr>
<td>All test results grouped together</td>
<td>0</td>
<td>5.84</td>
<td>2.43</td>
<td>2.39</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>8.20</td>
<td>3.41</td>
<td>1.41</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>10.55</td>
<td>4.40</td>
<td>1.81</td>
<td></td>
</tr>
<tr>
<td>BS5268 values with correction for nail diameter</td>
<td>0</td>
<td>4.37</td>
<td>1.82</td>
<td>1.82</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>6.24</td>
<td>2.60</td>
<td>1.43</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>7.80</td>
<td>3.25</td>
<td>1.77</td>
<td></td>
</tr>
</tbody>
</table>

Table 6.9 Safe Design Values and Vertical Load Improvement Factors for 9.0mm Mediumboard with 3.25mm Diameter Nails.

<table>
<thead>
<tr>
<th>Method of Analysis</th>
<th>Vertical Load kN/stud</th>
<th>Design Racking Load kN</th>
<th>Design Racking Resistance kN/m</th>
<th>Design Racking Resistance kN/m</th>
<th>Vertical Load Improvement Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test results taken in pairs and meaned</td>
<td>0</td>
<td>2.61</td>
<td>1.09</td>
<td>1.05</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>3.69</td>
<td>1.54</td>
<td>1.41</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>4.52</td>
<td>1.88</td>
<td>1.73</td>
<td></td>
</tr>
<tr>
<td>All test results grouped together</td>
<td>0</td>
<td>3.13</td>
<td>1.30</td>
<td>1.02</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>3.74</td>
<td>1.56</td>
<td>1.19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>4.36</td>
<td>1.82</td>
<td>1.39</td>
<td></td>
</tr>
<tr>
<td>BS5268 values with correction for nail diameter</td>
<td>0</td>
<td>2.55</td>
<td>1.06</td>
<td>1.06</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>3.64</td>
<td>1.52</td>
<td>1.43</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>4.55</td>
<td>1.90</td>
<td>1.77</td>
<td></td>
</tr>
</tbody>
</table>

Table 6.10 Safe Design Values and Vertical Load Improvement Factors for 12.5mm Bitumen Impregnated Insulating Board with 3.25mm Diameter Nails.
Table 6.11 Safe Design Values and Vertical Load Improvement for 12.5mm Plasterboard with 2.6mm Diameter, or larger, Nails

<table>
<thead>
<tr>
<th>Method of Analysis</th>
<th>Vertical Load kN/stud</th>
<th>Design Racking Load kN</th>
<th>Design Racking Resistance kN/m</th>
<th>Design Racking Resistance kN/m</th>
<th>B.R.R (kN/m) including 2.4 safety factor</th>
<th>Vertical Load Improvement Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test results taken in pairs and meansed</td>
<td>0</td>
<td>2.45</td>
<td>1.02</td>
<td>0.68</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>3.83</td>
<td>1.60</td>
<td>0.56</td>
<td>1.56</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>5.20</td>
<td>2.16</td>
<td>0.22</td>
<td>2.12</td>
<td></td>
</tr>
<tr>
<td>All test results grouped together</td>
<td>0</td>
<td>2.59</td>
<td>1.08</td>
<td>0.72</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>4.20</td>
<td>1.75</td>
<td>1.62</td>
<td>2.44</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>5.81</td>
<td>2.42</td>
<td>1.00</td>
<td>1.43</td>
<td></td>
</tr>
<tr>
<td>BS5268 values with correction for nail diameter</td>
<td>0</td>
<td>2.35</td>
<td>0.98</td>
<td>0.65</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>3.36</td>
<td>1.40</td>
<td>1.43</td>
<td>1.77</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>4.20</td>
<td>1.75</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 6.12 - Summary of Test Results for Nail Diameter Changes

<table>
<thead>
<tr>
<th>Board Material</th>
<th>Nail Diameter (mm)</th>
<th>Performance Changes from Standard Test Case</th>
<th>Comments on Test Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Standard Diameter (3.25mm)</td>
<td>BRR OR Kn/ sq ft</td>
<td>2.5 Kn/ sq ft</td>
</tr>
<tr>
<td>Plywood</td>
<td>2.87</td>
<td>-12%</td>
<td>P14 v P7</td>
</tr>
<tr>
<td>Medium board</td>
<td>2.87</td>
<td>-12%</td>
<td>Averaged</td>
</tr>
<tr>
<td>Medium board</td>
<td>2.87</td>
<td>-12%</td>
<td>Averaged</td>
</tr>
<tr>
<td>811B</td>
<td>2.87</td>
<td>-12%</td>
<td>B6 v B4</td>
</tr>
</tbody>
</table>
Figure 6.1. The Effect of Nail Size on Racking Resistance

Key
(i) Plywood
(ii) B11B
(iii) Mediumboard (as tested)
(iv) Mediumboard (showing other tests)

Code Relationship : Performance $\propto$ nail diameter
<table>
<thead>
<tr>
<th>Board Thickness</th>
<th>Fixing</th>
<th>Datum Racking Resistance</th>
<th>Additional Contribution of Secondary Board on Timber Frame Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Principal Board on Timber Frame Wall</td>
<td>Plasterboard or Category 2 Sheathing</td>
</tr>
<tr>
<td>CATEGORY 1 SHEATHINGS</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.5mm Plywood</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.0mm Mediumboard</td>
<td></td>
<td>3.00mm diameter wire nails at least 50mm long</td>
<td>1.68kN/m</td>
</tr>
<tr>
<td>12.0mm Chipboard (Type III)</td>
<td></td>
<td>Maximum spacing 150mm on perimeter</td>
<td></td>
</tr>
<tr>
<td>6.0mm Tempered Hardboard</td>
<td></td>
<td>300mm internal</td>
<td></td>
</tr>
<tr>
<td>CATEGORY 2 SHEATHINGS</td>
<td></td>
<td>3.00mm diameter wire nails at least 50mm long</td>
<td>0.90kN/m</td>
</tr>
<tr>
<td>12.5mm BIIB</td>
<td></td>
<td>Maximum spacing 75mm on perimeter</td>
<td>150mm internal</td>
</tr>
<tr>
<td>LININGS</td>
<td></td>
<td>2.65mm diameter plasterboard nails at least 40mm long</td>
<td>0.90kN/m</td>
</tr>
<tr>
<td>12.5mm Plasterboard</td>
<td></td>
<td>Maximum spacing 150mm.</td>
<td></td>
</tr>
<tr>
<td>SEPARATING WALLS</td>
<td></td>
<td>2.65mm diameter plasterboard nails at least 60mm long and at 150mm spacing in each layer</td>
<td>0.90kN/m</td>
</tr>
</tbody>
</table>

Note: the principal board is either:
(i) the sheathing in a sheathing/lining combination
(ii) the stronger board in a combination of sheathings or linings

Table 6.13 - Datum Racking Resistance Values for Standard Sheathings and Linings

-225-
<table>
<thead>
<tr>
<th>Primary + Secondary Board</th>
<th>Test Comparison Combined v Single</th>
<th>Improvement on Single, Sheathing, 0kN/stud, 2.5kN/stud, 5kN/stud</th>
<th>Comment on Single Board Case</th>
<th>Comment on Combined Board Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plywood + Plywood Cat. 1 + Cat. 1</td>
<td>P20 v P7</td>
<td>+51% +49% +52%</td>
<td>P7 above average performance</td>
<td>P20 identical materials to P7</td>
</tr>
<tr>
<td>Plywood + Plasterboard Cat. 1 + Cat. 2</td>
<td>P21 v P7</td>
<td>+8% +9% +16%</td>
<td>P7 above average performance</td>
<td>P21 identical materials to P7</td>
</tr>
<tr>
<td>Mediumboard + Plasterboard Cat. 1 + Cat. 2</td>
<td>M13 v M2</td>
<td>+19% +21% +31%</td>
<td>M2 below average performance</td>
<td>M13 different materials but not noticeably better</td>
</tr>
<tr>
<td>B11B + Plasterboard Cat. 1 + Cat. 2 or Plasterboard + B11B Cat. 2 + Cat. 2</td>
<td>B14 v B5 (or B14 v G4)</td>
<td>+68% +75% +79% (or +75% +73% +71%)</td>
<td>B5 above average performance (or G4 above average performance)</td>
<td>B14 identical materials to B5 (or B14 identical materials to G4)</td>
</tr>
<tr>
<td>Plasterboard + Plasterboard Cat. 2 + Cat. 2</td>
<td>G5 v G4</td>
<td>+35% +60% +73%</td>
<td>G4 above average performance</td>
<td>G5 identical materials to G4</td>
</tr>
</tbody>
</table>

Table 6.14 - Summary of Test Results for Combined Sheathings
<table>
<thead>
<tr>
<th>Sheathing Combination</th>
<th>Design Value for BRR for Combined Sheathing</th>
<th>Test Value for BRR for Combined Sheathing</th>
<th>Percentage Improvement</th>
<th>Design Value for BRR for Principal Sheathing</th>
<th>Test Value for BRR for Principal Sheathing</th>
<th>Percentage Improvement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plywood + Plywood</td>
<td>2.73</td>
<td>3.38</td>
<td>+24%</td>
<td>1.82</td>
<td>2.28</td>
<td>+25%</td>
</tr>
<tr>
<td>Plywood + Plasterboard</td>
<td>2.09</td>
<td>2.50</td>
<td>+20%</td>
<td>1.82</td>
<td>2.28</td>
<td>+25%</td>
</tr>
<tr>
<td>Mediumboard + Plasterboard</td>
<td>2.09</td>
<td>2.32</td>
<td>+11%</td>
<td>1.82</td>
<td>1.95</td>
<td>+7%</td>
</tr>
<tr>
<td>B11B + Plasterboard</td>
<td>1.43</td>
<td>2.04</td>
<td>+42%</td>
<td>0.98</td>
<td>1.16</td>
<td>+18%</td>
</tr>
<tr>
<td>Plasterboard + Plasterboard</td>
<td>1.35</td>
<td>1.58</td>
<td>+17%</td>
<td>0.90</td>
<td>1.18</td>
<td>+31%</td>
</tr>
</tbody>
</table>

Table 6.15 - Comparison of Design Values and Test Results for Combined Sheathing Tests
<table>
<thead>
<tr>
<th>Vertical Load kN/stud</th>
<th>Improvement Gained Over Plasterboard Alone From the Combined Sheathing and Lining Panel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plywood</td>
</tr>
<tr>
<td>0</td>
<td>111%</td>
</tr>
<tr>
<td>2k</td>
<td>112%</td>
</tr>
<tr>
<td>5</td>
<td>115%</td>
</tr>
<tr>
<td>Average</td>
<td>113%</td>
</tr>
</tbody>
</table>

Table 6.16  Comparison of Combined Sheathing and Lining Panels with Those for Plasterboard Alone.

<table>
<thead>
<tr>
<th>Board Material</th>
<th>Nail Spacing</th>
<th>Nail Spacing Standard Spacing for Board (A)</th>
<th>Test Comparison</th>
<th>Performance Changes from Standard Test Case</th>
<th>Comments on Test Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>BRR</td>
<td>0kN/stud</td>
</tr>
<tr>
<td>Plywood</td>
<td>75/150</td>
<td>0.5</td>
<td>P15V P7</td>
<td>25%</td>
<td>41%</td>
</tr>
<tr>
<td>Medium board</td>
<td>75/150</td>
<td>0.5</td>
<td>M12V M6</td>
<td>27%</td>
<td>46%</td>
</tr>
<tr>
<td>BIIB</td>
<td>50/100</td>
<td>0.66</td>
<td>B13 v B5</td>
<td>41%</td>
<td>45%</td>
</tr>
</tbody>
</table>

Table 6.17  Summary of Test Results for Nail Spacing Changes
Racking Resistance (Expressed as a percentage of value for standard spacing)

Key

Proposed modification factors

<table>
<thead>
<tr>
<th>Test results</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_{102} = \frac{1}{(6A+4)}$</td>
</tr>
<tr>
<td>$K_{102} = \frac{1}{(5A+5)}$</td>
</tr>
<tr>
<td>$K_{102} = \frac{1}{(6A+3)}$</td>
</tr>
</tbody>
</table>

Where $A =$ \text{Nail spacing} \quad \text{Standard spacing for board}

Previous test results

- APA on BIIB
- TRADA on ply
- Robertson & Griffiths on Category 1 boards

Figure 6.2 The Effect of Nail Spacing on Racking Resistance
<table>
<thead>
<tr>
<th>Board Material</th>
<th>Board Thickness (mm)</th>
<th>Thickness Standard Thickness (%)</th>
<th>Test Comparison</th>
<th>Performance Change from Standard Test Case</th>
<th>Comments on Test Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plywood</td>
<td>12.5</td>
<td>+32%</td>
<td>P2 v P4</td>
<td>+16%  +16%  +19%  +15%</td>
<td>below average</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>identical to standard thickness</td>
</tr>
<tr>
<td>Plywood</td>
<td>7.5</td>
<td>-21%</td>
<td>P3 v P14</td>
<td>-7%  -10%  -6%  -3%</td>
<td>above average</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>identical to standard thickness</td>
</tr>
<tr>
<td>Medium Board</td>
<td>12.0</td>
<td>+33%</td>
<td>M11 v M4</td>
<td>+32%  +32%  +27%  +38%</td>
<td>below average</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>M11 v M5</td>
<td>+1%  -4%  +8%  +11%</td>
<td>above average</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Average</td>
<td>+16%  +14%  +17%  +24%</td>
<td>different type of board in both cases</td>
</tr>
<tr>
<td>B11B</td>
<td>15.5</td>
<td>+24%</td>
<td>B12 v B4</td>
<td>+32%  +30%  +24%  +32%</td>
<td>average</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>identical to standard thickness</td>
</tr>
<tr>
<td>Tempered Hardboard</td>
<td>4.8</td>
<td>-20%</td>
<td>H4 v H3</td>
<td>-15%  -15%  -15%  -19%</td>
<td>identical to standard thickness</td>
</tr>
<tr>
<td>Flakeboard</td>
<td>12.0</td>
<td>+33%</td>
<td>F4 v F3</td>
<td>+16%  +8%  -14%  +16%</td>
<td>identical to standard thickness</td>
</tr>
<tr>
<td>Plasterboard</td>
<td>9.5</td>
<td>-25%</td>
<td>G3 v G2</td>
<td>-9%  -9%  -8%  -8%</td>
<td>identical to standard thickness</td>
</tr>
</tbody>
</table>

Table 6.18 - Summary of Test Results for Board Thickness Changes
Figure 6.3 The Effect of Board Thickness on Racking Resistance

- **Proposed modification factors**
  - \( K_{103} = 0.8 + 2.8B - B^2 \)
  - \( a \)  
  - \( K_{103} \)  
  - \( B \) (\( B < 1 \))  
  - \( 1 \) (\( B > 1 \))
  - \( a(\text{ext}) = K_{103} = B \) (\( B > 1 \))

- **Test data**
  - (i) — Plywood
  - (ii) — B11B
  - (iii) — Flakeboard
  - (iv) — Tempered Hardboard
  - (v) — Plasterboard

Where, 
- \( B \) = Board thickness
- Standard thickness for the sheathing material
<table>
<thead>
<tr>
<th>Vertical Load kN/stud</th>
<th>Effect of Horizontal Sheathing Compared with Standard Vertical Sheathing Boards of same Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>9.5mm board unblocked</td>
</tr>
<tr>
<td>0</td>
<td>-24%</td>
</tr>
<tr>
<td>2½</td>
<td>-34%</td>
</tr>
<tr>
<td>5</td>
<td>-41%</td>
</tr>
</tbody>
</table>

Table 6.19 Comparison of Horizontal Sheathing Results for Plywood

(a) Unblocked

76mm (70) total deflection.

LOAD

bending of studs.

3mm (2) uplift.

63mm (53) relative movement.

(b) Blocked

80mm (70) total deflection.

LOAD

studs straight.

29mm (31) uplift.

6mm (10) relative movement.

Note: Main values are for 7.5mm board and those in brackets are for 9.5mm board

Figure 6.4 Behaviour of Panels at Failure in Horizontal Sheathing Tests
Racking Resistance
kN/m

Permissible Resistance
(Strong Board)

Basic Resistance
(Strong Board)

Permissible Resistance
(Weak Board)

Basic Resistance
(Weak Board)

Figure 6.5 Hypothetical Response of Panels to Improved Restraint
<table>
<thead>
<tr>
<th>Modification Factor</th>
<th>Value of Factor orGenerating Equation</th>
<th>Change From Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nail Size (K101)</td>
<td>Proposed Nail Diameter (mm) 3 for diameters between 2.25 and 3.75mm</td>
<td>None</td>
</tr>
</tbody>
</table>
| Nail Spacing (K102) | \[
\frac{1}{0.6A + 0.4} \\
A = \text{Proposed Nail Spacing} \\
\text{Standard Nail Spacing}
\] For 0.5 < A < 2.0 | Code uses \[
\frac{1}{0.67A + 0.33}
\] |
| Board Thickness (K103) | \[
2.8B - B^2 - 0.8 \\
B = \text{Proposed Board Thickness} \\
\text{Standard Board Thickness}
\] For 0.7 < B < 1.3 | Factor is B if B < 1 Factor is 1.0 if B > 1 |
| Board Orientation (K104) | Horizontal with Noggins = 1.0 Horizontal without Noggins = 0.9 \(\frac{1}{(K_{110})^{0.5}}\) | Not included in Code |
| Frame Material (K105) | 1.0 (insufficient data for any alternative proposal) | Not included in Code |
| Vertical Load (K110) | Zero kN/stud = 1.00 2.5 kN/stud = 1.43 5 kN/stud = 1.77 | None |

Table 6.20 Modification Factors for Use With Standard Panels
SECTION B

FACTORS AFFECTING WALL RACKING PERFORMANCE

6.5 THE MAJOR TEST PROGRAMMES

The proposals for the design method given in Chapter 4 showed (Figure 4.1) that the standard procedure for wall design was to use the basic racking resistance for the materials used in the wall construction together with modification factors for wall behaviour. Following the work on material properties in Section A, this section investigates the principal variables affecting wall racking resistance which are those of length, vertical load and openings.

Three test programmes were carried out to predict the behaviour of timber frame walls each incorporating significant differences which both extended the available information and made the comparison process more complicated. The wide range of the tests ensures that the modification factors proposed are acceptable to a general design situation and are therefore suitable for inclusion in the standard design method. For each set of tests full details of the panels are noted and procedural differences are stated (compared with those detailed in Chapter 5) where changes have been made necessary by the reuse of panels in a number of different combinations.

The 1977 tests used mediumboard sheathing and hem-fir frames. They covered the performance of plain walls (i.e. without openings) between 0.6 and 4.8m long. One of the principal objectives of the programme was to establish behavioural differences between combinations of smaller panels, where top and bottom rails were not continuous and boards started and ended on separate studs, and single units where separate sheathing was joined on a single stud. The second programme in 1979 also used mediumboard sheathing but on a different quality frame material. The panels incorporated openings and were based on a modular system. Tests concentrated on 4.8m wall lengths. Size and location of openings were based on experimental needs and consequently openings were not allowed within 0.6m of the start of the wall.
Some tests were carried out on fully-clad panels to enable comparison with the 1977 tests and, in both programmes, all tests were carried out at zero and 5 kN/stud. The final set of tests in 1985 used 1.2m modules but positioned the openings so that a much greater variety of opening size and shape could be achieved making the walls more typical of standard practice. Openings were allowed as close as 0.3m from the start of the wall. The panels were sheathed in plywood in recognition of it being the most important sheathing material. The majority of tests were conducted under a 2½ kN/stud vertical load as, at that time, basic racking resistance related to a mid range vertical load. Tests were carried out at zero and 5 kN/stud load and on plain walls to allow direct comparison with the previous work.

Before examining the results individually it is valuable to note the procedural difficulties experienced that were common to all three programmes. The first problem was that, for the sake of economy, panels had to be reused in many different stiffness tests and in up to three failure tests. To accommodate this approach to testing, joints between modules were made with bolted connections. Three M 12 black bolts were used at every vertical joint but, due to the large relative movement of the studs, the holes were drilled oversize to allow repetitive assembly and therefore the clamping action of the bolts was relied upon to transfer load through the joint. Further continuity was achieved in all tests, including the single wall units, by using head binders and bottom plates which were made continuous over joints. The effect of joints on wall construction is considered in more detail in Section 6.6. The joints used throughout the present tests are considered to represent a standard that should be required in normal construction. A later section details alternative ways in which this standard may be achieved.

The changes necessary in test procedure to reduce the number of load cycles carried out on each test panel has already been detailed in Chapter 5. It was concluded that in the combination tests, carried out after each panel had been individually tested, the single load cycle represented
the lowest (or worst) test cycle. The problem concerning the limitation on failure tests was also noted in Chapter 5. Fortunately it was found that for many walls, and in particular those with large openings, wall strength exceeded that required by the stiffness tests to provide an adequate factor of safety. However, there was still a need to fail a representative sample of walls which necessitated the reuse of panels. In order to reduce damage the failure tests were concluded when an adequate factor of safety had been achieved. The wall could then be retested at a higher vertical load or the panels reused in different combinations. Before reuse, all panels were carefully examined and any major board damage repaired.

The problem in the assessment of design loads was noted in Chapter 5. It is caused by the non standard test procedures and the similarity of tests within a programme without any one specific case being repeated. The partial safety factors to be used in the design are a subject open to conjecture and the following arguments could both be supported:

(i) where the tests are different, as with openings a safety factor of 0.8 should be used,

(ii) because the numerous tests are basically similar the design line linking the measured results could be used with a K300 safety factor of 1.0. Thus in practice each result includes the 1.0 factor.

It was decided however to limit the safety factor to 0.87 so that the results would be compatible with those of the standard panel tests. The situations are similar in that the basic racking resistances were taken from many tests which were closely related although not identical. By using a K300 value of 0.87 all the test results can be directly compared which is beneficial to the assessment of relative performance. The effect of the choice of safety factor on the general design equation will be examined prior to determining suitable values for the modification factors.
The failure results are also closely linked to the K₃₀₀ factor. In all tests where panel strength was examined without taking the wall to failure, an overall factor of safety of 2.0 was attempted.

The three test programmes are now detailed:

A THE 1977 PLAIN PANEL COMBINATION TESTS

Materials

Sheathing: Karlitpanel - Extra medium density fibreboard 9.0mm thick.

Framework: Hem-fir graded "construction" but of particularly good quality; 90 x 40mm nominal section size.

Fixings: 50mm long 3.25mm diameter hot dip galvanised clout head nails spaced at standard 150/300mm centres.

Panel Sizes

All panels 2.4m high

0.6m length: Four panels, A600, B600, C600 and D600

1.2m length: Five panels, E1200, F1200, G1200, H1200 and M1200

1.8m length: One panel, K1800 comprising a 1.2m wide sheathing followed by a 0.6m board. One panel, L1800 comprising a 0.6m wide sheathing followed by a 1.2m board.

2.4m length: Four panels, I2400, J2400, N2400 and O2400

3.6m length: One panel, P3600

4.8m length: One panel, Q4800
Panel Construction

Standard details incorporating 1.2m wide sheathings boards on 0.6m nominal stud centres were used wherever possible. In the longer panels, boards were joined on a single stud. Joints between panels were made with three M12 black bolts at heights of 0.3, 1.2 and 2.1m and by using top and bottom plates continuous over panel breaks.

Test Procedure

All panels and combination of panels were loaded once to a racking deflection of 5.0mm. The standard readings for sliding and uplift were recorded together with additional measurements for nail slip along the edge fixings of each board and sheathing rotation. All zero vertical load stiffness tests were completed before starting the 5.0 kN/stud tests. In the failure tests the panels and combinations were loaded until the maximum racking load was achieved. Panels were rarely reused after failure and then only if they had been the second panel in a pairing.

Test Results

The principal results, the racking loads to cause a 5mm deflection are tabulated in Table 6.21. They have been divided into individual unit and combined panel tests to help in assessing the effect of joints within walls. It can be seen that there are no significant differences in performance except at 4.8m length where the single result for the individual unit cannot be regarded as conclusive. The results show that walls which include head binders and bottom plates, to achieve continuity, and incorporate adequate vertical joints between panels may be designed without reference to their make up. Practical limitation must be applied to this assumption and it is intended that the standard module for walls should not be less than 1.2m although the results should allow for a few 0.6m panels to be included if necessary to the positioning of openings.

The strength test results are shown separately in Table 6.22 together with the factor of safety based on the
test load. Although the results exceed the required safety factor of 2.0, except in the case of Q4800 which had an extremely high test stiffness load, it is noticeable that the factor reduces as panel length and vertical load increase.

The stiffness test results are shown graphically in figures 6.6a and b. Two lines were originally fitted by the method of least squares to the results at each vertical load. The first showed a linear relationship between load and length. It was not possible to apply a single relationship to all the results and thus panels shorter than 1.2m are covered separately using the origin and the 0.6m results. The second line is a quadratic solution which allows all the results to be covered by one equation. The line has merits in dealing with walls up to 2.4m long but the rapidly increasing performance predicted for longer panels is not confirmed in the test results. It is likely therefore that for longer panels the relationship between length and load will tend to a linear pattern.

Table 6.23 gives the meaned results for all similar tests although it is noted that the number of tests for each length is variable. Assuming the safety factors to be adequate design values for the walls may be determined by multiplying the meaned 5mm deflection load by 1.0875 (i.e. the 1.25 factor predicting performance at 0.003 times panel height and the 0.87 factor for similar tests). The 2.4m wall results may then be used to show that the basic racking resistance of the material is 2.25 kN/m. The single design value is very efficient here because the vertical load factor for the tests is 1.80 compared with the predicted value of 1.77 used in the calculation of BRR. It is notable that the performance is identical to that of test M5 shown in Table 6.2a suggesting that the wall test results will be typical of a good quality mediumboard on high quality frame material. The meaned 5mm deflection load data is also used to show test values for length and vertical load factors related to 2.4m length and zero vertical load respectively.
This information will be used in the analysis of the wall length and vertical load modification factors. It is clear that the vertical load factor is not a constant and decreases for increasing panel length. Consequently the length factor can also be seen to diminish with increasing vertical load.

In this test programme all lengths included at least five separate wall tests except at 0.6m, where an error of ±3% in the average might be expected for the four tests, and at 3.0m, where an error of ±15% could be expected for the two tests. The authenticity of the results can be checked noting that single results should lie within ±25% of the average (see Section 6.6.1).

B THE 1979 PANEL COMBINATION AND OPENING TESTS

Materials

Sheathing: Karlitpanel - Extra medium density fibreboard 9.0mm thick.

Framework: Spruce/pine/fir (SPF) 90 x 40mm nominal section size, construction grade but weaker nail holding characteristics than hem-fir.

Fixings: 50mm long 3.25mm diameter hot dip galvanised clout head nails spaced at standard 150/300mm centres.

Panel Sizes (see figure 6.7)

All panels 2.4m high

0.6m length: Four panels A to D.
plain panels

1.2m length: Six panels, E to J.
plain panels
1.2m length: Two panels K and L with 2.18m high door panels by 1.04m wide opening.

1.2m length: Four panels M to P with 1.18m high window panels by 1.04m wide openings.

Panel Construction

Standard details applied throughout. Window and door panels had 180mm deep lintol units on cripple studs nailed to main studs at 600mm centres. No sheathing was applied over the lintols and below the windows the sheathing was vertically fixed to the main studs and the central stud only. Joints between panels were made with M12 black bolts, at heights of 0.3, 1.2 and 2.1m, and by top and bottom plates continuous over panel joints. The continuity of the bottom nail and bottom plate under the door panels was not typical of practice and was included to help in attaching the walls to the base of the test rig.

Test Procedure

The first series of tests was carried out at zero vertical load. All panels were first tested individually to 5mm deflection through four load cycles. The three 2.4m long wall combinations, followed by the thirty six 4.8m long combinations were then subjected to a single 5mm deflection test. All the tests were then repeated at 5 kN/stud vertical load. A very small number of walls with openings were then tested to failure. Standard measurements for racking deflection, uplift and sliding were taken throughout the programme.

Test Results

The principal results, those of racking load to produce a 5mm deflection and failure load are given in Table 6.24. The individual panel results include both
the first and the lowest cycle loads. The panel combinations are identified in Figure 6.8. The results of the plain panel tests for both vertical loads are plotted in Figure 6.9 and a linear solution similar to that used on the 1977 tests is applied to the results basing short panel behaviour on the 0.6m length results. No differentiation is made between walls with only 1.2m panels and those including the 0.6 panels. Failure tests were only carried out on 4.8m walls with openings. Nearly all the factors of safety achieved were greater than 2.5 inferring that strength is a lesser problem in the more usual perforate walls. The test results for the plain panels have been averaged for similar tests and the results included in Table 6.25.

Analysis of the 2.4m wall results gives a basic racking resistance value of 1.77 kN/m which is 21% down on the 1977 test results due entirely to the change in frame material. The vertical load factor for the standard panel length is 1.78 almost identical to the predicted value of 1.77. The basic racking resistance is below the 1.82 kN/m prescribed in Section 6.4 for this particular form of construction, however, as the test value includes a K300 factor of 0.87 and is within 15% of the design figure the results are perfectly acceptable and represent a below average quality of frame material. The vertical load and panel length factors are similar to those calculated for the earlier tests.

All the lengths were tested with five or more panels so that an average result should have been achieved except at 2.4m where an error of up to ±15% is possible as only two panels were tested. This may have influenced the basic racking resistance value but it is noticeable that the differences compared with the 1977 results were similar to those of the 4.8m walls where eight separate tests were carried out.

The wall opening results cannot be related to the other performance figures except in a comparison of the
vertical load performance. The average enhancement at 5 kN/stud for the 4.8m panels is 1.57 for results which are normally distributed and lie between 1.26 and 2.04. The higher values relate to door panels and represent the splitting of the wall into two separate small units. However, some of the low factors also include door panels so that no general trend for vertical load performance can be noted. The wall opening results will be reconsidered later together with the 1985 test results.

C THE 1985 PANEL COMBINATION AND OPENING TESTS

Materials

Sheathing: Canadian CSP sheathing grade plywood 9.5mm thick.

Framework: Spruce/pine/fir (SPF) 90 x 40mm nominal section size, GS grade with good nail holding characteristics.

Fixings: 63mm long 3.25mm diameter electro galvanised wire nails spaced at standard 150/300mm centres.

Panel Sizes (see Figure 6.10)

All panels 2.4m high

1.2m length: Eight panels A1 – A8.
plain panels

1.2m length: Two panels B1 and B2 with a 1.10m full length window panels high by 1.04m wide opening.
1.2m length: Two panels C1 and C2 with a door opening 2.10m high by 0.80m wide at the leading end of the panel plus: Two panels D1 and D2, as C1 and C2, but handed so that the door opening is at the trailing end of the panel.

1.2m length: Panel E9 with a window opening 0.90m high by 0.80m wide at the leading end of the panel, plus: Panel E11 as E9 but with a 1.10m deep window, plus: Panels F9 and F11 as E9 and E11 but handed.

1.2m length: Two panels G1 and G2 with a window opening 0.90m high by 0.50m wide at the leading end of the panel, plus: Two panels H1 and H2 as G1 and G2 but handed.

Panel Construction

Standard details applied throughout. Window and door panels had a 165mm deep lintol supported on cripple studs nailed to the main studs at 600mm centres. In all panels except G2 and H2 rectangular sheathing boards were used, with separate boards placed to the side and underneath openings; sheathing was not positioned over the lintols. A single 2.4m by 1.2m board was used to clad panels G2 and H2 with a window cut out leaving an "L" shaped sheathing. Joints between panels were made with M12 bolts at heights of 0.3, 1.2 and 2.1m, except in the case of openings where the top bolt was at a height of 2.3m, and by a head binder and bottom plate continuous over panel joints.

Test Procedure

Throughout the programme, tests were concentrated on the 2½kN/stud vertical loading with approximately 30%
of the walls further tested under zero and 5 kN/stud vertical load. Unlike the previous tests the three vertical load tests on a wall were performed consecutively starting with the zero vertical load. A racking preload test was conducted prior to every new test, i.e. change in panels or change in vertical load. Initially all panels were tested individually before being used in combinations giving wall lengths of 2.4, 3.6 and 4.8m. A number of plain wall tests were included but in the majority of these tests the openings were positioned to give practical wall layouts as shown in Figure 6.11. No opening was allowed within 300mm of the ends of a wall and panels were combined in such a way as to create large openings albeit broken up by the end studs of the panels.

In addition to the standard deflection measurements, nail slips and board rotation were measured for a number of wall configurations and also the relative vertical movement between panels.

A number of strength tests were carried out on configurations of 3.6 and 4.8m panels similar to those tested in stiffness. However, it was occasionally necessary to change the panel number in view of the limited use that could be made of panels in strength tests. The tests were carried out either at 2\frac{1}{2} kN/stud vertical load or at both zero and 5 kN/stud. In the latter case the zero load test was carried out first and was followed immediately by that at 5 kN/stud. In all cases the maximum load was predetermined from the stiffness performance and included a safety factor greater than two. Panels were reused in further wall combinations but when they had acted as the leading panel they were subjected to much closer scrutiny and repaired before testing in a less onerous position in the wall.

Test Results

The principal results, the lower racking load producing a 5mm deflection and the strength test load,
together with the factor of safety achieved are given in Table 6.26. The panel combinations may be interpreted using the diagrams included in Figure 6.11. In any strength test the wall did not reach maximum load or show any sign of impending failure; it is considered that much higher factors of safety could have been achieved thus, as previously noted, walls with openings are always likely to be governed by their stiffness.

The plain panel results are plotted in Figure 6.12a and b, to which a linear relationship has been fitted. As no 0.6m long panels were tested, a short panel relationship cannot be determined. In view of the greater number of tests at 2½ kN/stud and the variance in performance, two methods of analysis are considered. The first uses the test results alone. The second, which is based on test experience rather than mathematical justification, uses the mean of each set of results and modifies the zero and 5 kN/stud results to take account of all the results at 2½ kN/stud. In this case, if the mean of all the 2½ kN/stud results is termed M and the means of the results for only the panels tested under the three load conditions are M0, M2½ and M5, then the modified test loads are given as:

(1) at zero vertical load : $M_0 \times \frac{M}{M_{2\frac{1}{2}}}$

(ii) at 2½ kN/stud : $M$

(iii) at 5 kN/stud : $M_5 \times \frac{M}{M_{2\frac{1}{2}}}$

The modifications do not greatly affect the results but because they produce a lower and consequently safer set of values, they have been used in all later comparisons such as in Table 6.27, where the meaned test results are shown and from which design values may be calculated.

The performance of the 1985 panels is very different from previous tests. In general they are much stronger in
weaker situations, i.e. at zero vertical load and as short panels. This means that both the vertical load and panel length relationships are unlike the 1977 and 1979 tests. The 2.4m standard panel results do not match with the vertical load requirements used to find the basic racking resistance and the calculated value of 2.39 kN/m is thus very low in comparison with the test performance at zero vertical load. The results, however, are typical of panels tested using redwood/whitewood studs and it is considered that the differences are due to the behaviour of the studs and not to the changes in sheathing and fixing types. The results may be compared with those of panel P9 in Table 6.1 and panel M7 in Table 6.2, when the high stiffness gained from the use of redwood/whitewood studs may be noted.

The effect of the studs on performance has also reduced the vertical load comparison values at all lengths but the trend for the factor to reduce as panel length increases is still noticeable. The 2.4m length result is remarkable within this trend and further indicates that the zero vertical load results on these panels were much higher than could normally be expected. It is likely that the same effect will be experienced in the results of the walls with openings. Comparison of zero and 5 kN/stud performance for these walls show enhancement factors of 1.47, 1.68, 1.58 and 2.18 at 4.8, 3.6, 2.4 and 1.2m panel lengths respectively. Less significance can be attached to the 2.4 and 1.2m values which were influenced by a small number of unusual results, but the other values are reliable being based on a larger population of readings with a much lower standard deviation. The results are noticeably closer to those of the 1977 and 1979 plain panels and the 1979 walls with openings.

The plain panel tests in this programme include fewer tests for each wall length and therefore the likelihood of an average value being achieved is reduced and consequently the accuracy of the design data.
6.6 WALL DESIGN MODIFICATION FACTORS

6.6.1 Reduction of Results

The results of the three test programmes showed quite marked differences in behaviour. These can be seen by bringing together the vertical load factor and the wall length factors from Tables 6.23, 6.25 and 6.27. They are plotted in Figures 6.13 a to c. For comparison purposes the 1985 test results at 2\(\frac{1}{2}\) kN/stud vertical load have had to be ignored. In graph (a) the variability of the vertical load factor can be seen to be independent of length but the overall reduction with length is clear. Graphs (b) and (c) show the length factor to increase with length and, ignoring the tailing off for short panels, the relationship is approximately linear. The variability in the results increases with length because they have been normalised for the 2.4m wall case. This was done because the length is that of the standard panel on which the majority of tests have been carried out. However, in two of the three wall length programmes very few tests have been carried out on 2.4m panels hence little reliance can be placed on their value; they are not suitable as datum points and may significantly affect all the other readings causing the large variations at 4.8m length.

The three test programmes were carried out on very similar materials; plywood and mediumboard have been shown to be very close in terms of performance, the nails used were identical in diameter and the differences in length and head size would have little significance, and the frame wood although different in each case was of high quality throughout (noting that the SPF used in the 1979 test has been seen in the standard panel tests to give reduced performances compared with the hem-fir and the 1985 SPF). It is therefore possible to consider all the results together and derive all modification factors from the averaged figures. They are plotted in Figure 6.14 using the meaned 5mm deflection loads from Tables 6.23, 6.25 and 6.27 and noting for each data point the number of tests from which

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the result has been averaged. The number of similar tests gives an indication of the reliability of the value and expected variations about a mean for five or less tests can be noted in Figure 6.24 a based on the K300 modification factor (Chapter 5). The behaviour is noted as being fairly uniform and the variability can be seen to be in proportion to the average load. The calculations for the averaged test values are shown in Table 6.28 where the data is weighted according to the number of similar results. The design factors will now be based on these figures which are representative of stiffness performance only. Thus, initially, the factors assume stiffness to be the governing criteria in design and consequently checks will need to be made using the few failure tests carried out on the wall units to determine if this hypothesis is acceptable and what changes may be required if it is not.

6.6.2 **Vertical Load**

An early method of dealing with vertical load was proposed by Robertson and Griffiths (1971). In this method the mid range vertical load was taken as the datum and the modification factors determined for zero and 5 kN/stud referenced to a unit value at 2½ kN/stud. The author found from the test results detailed in Section 6.2 that the most suitable values for the standard test panel were 0.7 and 1.25 at zero and 5 kN/stud respectively. Initially a linear interpolation was allowed between the three results but it was noted that for computer based design an equation linking the points would be more useful. At the same time it was decided that for design purposes a uniformly distributed load should be quoted. This was calculated for the 2.4m panel with studs at 0.6m centres using the formula:

\[
F = \frac{5V}{2.4}
\]

where \(F\) is the equivalent UDL in kN/m and \(V\) is the stud load. Thus 2½ and 5 kN/stud loads became 5.2 and 10.4 kN/m respectively and the vertical load relationship was determined to be
\[ KVL = 0.7 + 0.063F - 0.001F^2 \] - 6.11

where KVL represents the vertical load modification factor. This equation was included in early drafts of the Code of Practice.

Tests on panel lengths other than 2.4m introduced two further problems. Firstly the relationship between stud load and UDL varied with panel length and secondly the results, as shown in Section 6.6.1, indicated the vertical load relationship to be length dependent. The general relationship between stud load and UDL can be shown to be:

\[ F = \frac{V}{\text{studs spacing}} + \frac{V}{L} \] - 6.12

It should be noted that in all the tests in this investigation the stud spacing is nominally 0.6m. Thus a constant stud load produces increased equivalent distributed loads as panel length reduces. In long panels the changes are small but the effect on 0.6 and 1.2m panels is significant. Figure 6.15 shows the modification for UDL and stud loads generated using equations 6.11 and 6.12. The author considered these modification factors to be unsuitable as they were based on datum taken at a mid level of vertical load which could not be directly related to normal test conditions, i.e. to achieve the datum value at 2.4m length the vertical load is 2.5 kN/stud but for the same value modification factor at 4.8m length the vertical load must be 2.81 kN/stud. He therefore proposed that the datum case be changed to zero vertical load for the following reasons.

(i) Zero vertical load is the same in terms of stud load and uniformly distributed load. Therefore test results can be more easily interpreted.

(ii) Modification factors are always positively related to the datum value simplifying the equation by which they are generated.
(iii) All modification factors are greater than unity thus if a designer accidentally omits the modification factor he will achieve a fail safe situation as the design will automatically assume the zero vertical load case which is the least beneficial to racking performance. Previously omission of the vertical load factor assumed a 5.2 kN/m top load which could be an overestimate of that provided.

The proposal was accepted for inclusion in the draft for publication of BS 5268 (BSI, to be published). The alterations necessary were minimal because the vertical load factor was considered independent of all other factors. The change was achieved by dividing equation 6.11 by 0.7 thus the vertical load modification factor used in the Code, denoted as K400, was given by:

$$K_{400} \text{ (BSI)} = 1 + 0.09F - 0.0015 F^2$$  \hfill (6.13)

In addition the vertical load factors used in determining basic racking resistance from tests had to be altered (i.e. the change from K303 to K302 noted in Table 5.1). The effect of this change is shown in Figure 6.16 for both the UDL and stud load cases. The K400 factor is also shown on Figure 6.13a as a comparison with the test results.

As a result of the changes the effect of wall length on vertical load can be more easily analysed. In Table 6.29 the requirements for the vertical load factor at 5 kN/stud for the three test programmes are compared with the Code modification factor adjusted to give the stud load case using equation 6.12. The meaned values for the test requirement have also been plotted on Figure 6.16. It is clear that the vertical load factor is dependent on length and thus if independent length and vertical load factors are used, design
values for some conditions will be either inefficient or unsafe. As a consequence the following design conditions can be analysed.

a) The vertical load factor is kept constant and takes the values already calculated for the 2.4m panel (i.e. uses equation 6.13) and the length factor is based on the zero vertical load case. Then (as shown in Figure 6.16) the design values would be progressively more unsafe as length increased from 2.4m and vertical load increased. Below 2.4m length, design values would all be safe but this would be of little practical benefit.

b) The design approach noted above is retained but the vertical load factors found for the maximum test length are substituted. This has the benefit that only walls longer than 4.8m will be unsafe but, because the vertical load factors will not be the same as those found for standard panels, the basic racking resistance value is the only starting point for panel design.

c) The vertical load factor is kept constant and takes the values already calculated for the 2.4m panel but the length factor is based on the maximum vertical load case (say 5 kN/stud). Then the design loads would be safe for all walls longer than 2.4m but there would be an increasing loss in efficiency for longer panels and reduced vertical loads. The loss of safety in short panels at low vertical load would be inconsequential. This is a satisfactory solution in terms of safety but the choice of an arbitrary vertical load condition at which the length factor is determined is not sensible as previously noted. Furthermore at vertical loads higher than the standard condition the method would be similar to solution 'a' and unsafe for all lengths greater than 2.4m.

d) The vertical load factor reduces with panel length related to the test requirements and the length factor is
based on the zero vertical load results. This gives a safe efficient method for all walls within the range tested although the nature of the modification factor will possibly allow extrapolation of the design equations (which can be checked using the computer analysis). In practice it is adviseable for the vertical load modification factors to be identical at 2.4m length to those already determined from the standard panel tests. The vertical load factor can be directly related to the test results without the need for an additional safety margin because the basic racking resistance is calculated from the weakest vertical load performance (based on the standard factors) and the vertical load condition is not being extrapolated in the calculation of wall strength. This will enable the length modification factor to be determined from all the test data using a normalisation process whereby a zero vertical load condition is found by dividing the test result by the appropriate vertical load modification factor. Calculating the length factor in this way will achieve the most accurate solution minimising errors and inefficiencies.

A decision was taken to adopt solution 'd' for the principal design method and to revert to solution 'b' for the simplified approach where there is an advantage in having independent modification factors and loss of efficiency is accepted as being the necessary consequence of greater simplicity in calculation.

The proposed vertical load modification factor K110 is related to the uniformly distributed load case and is based on the results of the 5 kN/stud tests shown in Table 6.29 b. At 2.4m it must coincide with the K400 factor which is identical to that used in Section A of this Chapter to determine basic racking resistance. It is noticeable that the average of the 14 tests at 2.4m length is identical to that value proving the accuracy of the model.

Two equations were analysed for their ability to match the requirements of the test results, concentrating on the more typical longer walls. They were
\( (i) \quad \text{KVL} = (K_{400}) \times \left(\frac{2.4}{L}\right)^m \quad - 6.14 \)

\( (ii) \quad \text{KVL} = 1 + \left[ (K_{400} - 1) \left(\frac{2.4}{L}\right)^m \right] \)

i.e.

\[ \text{KVL} = 1 + \left[ (0.09F - 0.0015F^2) \left(\frac{2.4}{L}\right)^m \right] \quad - 6.15 \]

Equation 6.15 was found to give a larger variation between 0.6 and 4.8m but it was noted that neither equation could cope with the very large change between the 1.2 and 0.6m panels. Therefore in trials conducted, varying the value of 'm' in practical design increments of 0.05, accuracy was sought in the 1.2 to 4.8m range. The best fitting equation was found to be:

\[ K_{110} = 1 + \left[ (0.09F - 0.0015F^2) \right] \left(\frac{2.4}{L}\right)^{0.4} \quad - 6.16 \]

noting that equation 6.12 must be substituted into equation 6.15 to model the stud load condition of the tests.

Modification factors using this equation are given in Table 6.29 a and plotted in Figure 6.17. They compare very well with the tests for the more important lengths on which the majority of the tests were concentrated viz 1.2, 2.4 and 4.8m. In the analysis it was noted that a slight increase in the value of 'm' would improve the model at 1.2m length and reduce the factor at 4.8m thus increasing safety. In order to determine the better option the vertical load factors for the wall opening tests were examined. The results are shown in Table 6.29 b. It is noticeable that all the factors for openings exceeded those for plain walls in the same test programme. The results are still very much in agreement with the proposed modification factor and consequently there is no need to increase safety for longer panels, hence equation 6.16 is retained.

The \( K_{110} \) modification factor can be tabulated for both UDL and stud loads but its generating equation is more
suited to use in a computer based design package due to the wide range of variables.

The simplified vertical load modification factor $K_{210}$ can be modified from the $K_{400}$ factor using the results of the 4.8m wall tests resulting in the equation:

$$K_{210} = 1 + 0.07F - 0.0013F^2$$  \hspace{1cm} - 6.17

This can be further simplified to

$$K_{210} = 1 + 0.06F$$  \hspace{1cm} - 6.18

with only a 2% loss in efficiency within the normal design range. Equation 6.18 will therefore be used for design purposes and combined with equation 6.12 to generate the modification factors for stud loadings given in Table 6.29 for comparison with the test requirements. The results are also plotted in Figure 6.17. It is clear that the equation is very generous in the safety afforded panels shorter than 4.8m but its use on longer panels may lead to unsafe conditions unless the length factor is reduced to provide the required safety.

The modification factors for both the standard and simplified design methods are tabulated in Table 6.30 for the common vertical load cases used in this thesis.

6.6.3 Wall Length

Early work on the effect of wall length was carried out independently of the vertical load. Methods of analysis were fitted to results for different lengths of panels with a constant stud load. It was immediately clear, as demonstrated in Figures 6.6, 6.9 and 6.12 that no single relationship could be used to cover all the results. A linear analysis was a reasonable interpretation of results for panels 1.2m and longer but required a second line to cover short panels which should, in theory, have been related to the main analysis. The quadratic solution provided a
single relationship but did not achieve good correlation, furthermore the nature of the quadratic solution eliminated extrapolation of the results because the racking resistance was forced to increase with length. Early tests on 4.8m wall panels had shown that, for a given deflection limit the racking load from a single jack acting at the front of the panel was substantially less than the sum of the loads applied at two points along the panel length. This result together with the reducing factors of safety suggested that after a certain length had been reached, the racking resistance would start to decrease as it would become increasingly difficult to distribute the load through a longer wall. The load versus length response would then form a reverse curve. These initial ideas are outlined in Figure 6.18. In practice the design factor for length will tend to the reverse curve behaviour because there is no data to cover walls longer than 4.8m and predictions will have to be very conservative. The results of the computer analysis will be useful in making such predictions.

A major finding from the early test programmes was that, assuming adequate continuity and fixings, panel length could be considered independently of panel joints. The results also showed that the replacement of 1.2m wide boards with two 0.6m boards had little effect on performance, although it was considered that this result could not be extrapolated to cover lesser widths nor should 0.6m wide boards be predominant in a wall layout.

Few tests have been carried out on panels where the board width exceeded 1.2m. Such panels are possible in practice, if either particle board or tape jointed plasterboard are used, and would show an improvement in performance, however the design method need only cover the standard cases and these panels may be studied separately. It is suggested that the computer analysis discussed in Chapter 7 can be used to estimate their behaviour.

The aim of this section is to determine a modification factor for wall length which can be used with the basic
racking resistance (BRR) and vertical load modification factors already detailed to calculate the design load for plain walls. The modification factors will be independent of the basic racking resistance of the panel and therefore they can be analysed directly from the test results which represent the racking loads to produce a 5mm deflection. These are termed the test racking loads (TRL) and relate to the test racking resistance (TRR) calculated for a standard panel at zero vertical load. These factors are unimportant in design and are used here in order to avoid safety factors which are identical for design load and BRR. Clearly TRR is based solely on stiffness and the factors of safety required for use with the test data to protect against premature failure will have to be examined later in considering the design loads for plain walls. The modification factor is determined so as to give a mean prediction of test data since the results show no sign of greater variability with increasing length. TRR is calculated to be 2.05 kN/m for the meaned test results (noting the value, at zero and 5 kN/stud to be 2.06 and 2.05 kN/m respectively).

In order to satisfy the design relationship proposed in Chapter 4 the plain wall design must take the form:

\[ TRL = TRR \times L \times KVL \times KL \quad - 6.19 \]

where KL is the length modification factor.

The linear relationships shown in Figures 6.6, 6.9 and 6.12 are clearly unsuitable because they are unique to each test and therefore dependent on the value of BRR. The quadratic solution would fail for the same reason and is also of unsuitable format as the racking load is accelerating with length. A linear relationship such as that shown in Figure 6.19 would be suitable but would impose the following constraints, that:

1. the vertical load modification factor is independent of length,
(ii) $KL = \frac{(a - b)}{L}$ for long panels - 6.20

and

$KL = \frac{(a - b)}{SL}$ for short panels - 6.21

where 'a' and 'b' are constants and 'SL' is the limiting length for short panels.

The first constraint means that the analysis cannot accurately model the test results, but it would be suitable for use in the simplified design method when the vertical load modification factor is given by equation 6.17. If this equation is used then the following assumptions may be made:

(i) the zero vertical load test results will be accurately modelled by the design values for 'a' and 'b' for wall lengths 1.2m and longer,

(ii) the 4.8m length results at any vertical load will also be accurately modelled,

(iii) test results at vertical loads other than zero on walls shorter than 4.8m will exceed the design values and should not be used to determine the values of 'a' and 'b'.

Analysis of the test results (Figures 6.6, 6.9 and 6.12) have shown that the 0.6m panels fall in the short wall length and so their results must be omitted from the analysis to determine values for 'a' and 'b'. The linear relationship is determined by plotting:

$$\frac{TRL}{KL} \propto L$$
for the cases noted above and fitting a straight line relationship to the data as shown in Figure 6.20. The TRL values were taken from Table 6.28 and K210 factors from Table 6.30. A linear regression showed the best fit to be:

\[
\frac{\text{TRL}}{K_{210}} = 3.245L - 2.736
\]

Taking TRR as 2.05 kN/m and substituting in equation 6.19 gives:

\[
KL = (1.58 - \frac{1.33}{L})
\]

The design equation may then be rewritten as:

\[
\text{TRL} = 1.58 \times \text{TRR} \times K_{210} \times (L - 0.84)
\]

where the 1.58 value replaces the load factor and \((L - 0.84)\) is an equivalent length. This equation may be conservatively approximated to:

\[
\text{WRL} = 1.6 \times \text{BRR} \times K_{210} \times (L - 0.9)
\]

- 6.22

Using this simplified equation the K211 factor is given as:

\[
K_{211} = (1.6 - \frac{1.44}{L})
\]

- 6.23

Test data at zero vertical load (Table 6.28) indicate that the racking resistance of 0.6m panels should be approximately one quarter that of 1.2m panels for safety thus the value for K211 for short panels will have to be one half that of K211 for the 1.2m panel. Substituting into equation 6.23 and 6.20 gives 'SL', the maximum short panel length, to be 1.03m. This value is extremely sensitive; reducing it to a more practical value of 1.0m reduces the performance of the 0.6m panel by 20%. As a compromise the value of K210 can be based on the 1.03m short panel length but for practical purposes the boundary is taken as 1.0m. Then the
The design equation can be written as:

\[ WRL(L<1.0\text{m}) = BRR \times L \times (1.6 - \frac{1.44}{1.03}) \times K_{210} \]

Thus the modification factor for short panel length will be:

\[ K_{211} = 0.2 \quad (L < 1.0\text{m}) \quad -6.24 \]

For use with equation 6.22, the short panel design can be rewritten as

\[ WRL(L<1.0\text{m}) = 1.6 \times BRR \times (0.125L) \times K_{210} \quad -6.25 \]

Wall lengths outside the tested range (i.e. >4.8m) will be considered later because they will require an extrapolation of the design process.

The simplified wall design method has been used to predict test values for the plain panel tests. The results are shown in Table 6.31 and may be compared with the measured results in Table 6.28. They can be seen to model very accurately the test results at zero vertical load showing the efficiency of the panel length factor. The 5 kN/stud loads are all underestimated except for 4.8m length because the vertical load factor is independent of wall length. This is exactly as expected and as illustrated in Figure 6.17. A possible problem with a linear solution is that, although it is determined from the basic racking resistance, the analytical model may not predict that value for a 2.4m wall at zero vertical load. This is not the case with these results due to the accuracy with which the model represents the test results.

In preparing design information for the Code of Practice the author ensured direct use of the basic racking resistance by dividing the analysis into two parts for walls either shorter or longer than the standard length. Power curves fixed to the 2.4m panel results were fitted to
the rest of the data but, because a fixed vertical load factor was being used, the length factor was unable to model accurately the behaviour of the wall at all three vertical loads tested. For the short panels this did not present a major problem, as the loads were very small; the proposed equation:

\[ K_{100} = (L/2.4)^{0.75} \quad (0 < L < 2.4\text{m}) - 6.26 \]

was designed to give an average performance. However for longer panels more care was necessary and the proposed equation:

\[ K_{100} = (L/2.4)^{0.25} \quad (2.4 < L < 6.0\text{m}) - 6.27 \]

was designed to give safe performance figures at 10.4 kN/m and consequently underestimated performance at zero vertical load.

The Code allowed the extrapolation of equation 6.27 to 6.0m walls as the power function was small. For longer panels \( K_{100} \) was restricted to \((6.0/2.4)^{0.25}\) i.e. the value at 6.0m. Thus the following relationships between racking load (WRL) and length were established:

1. up to 2.4m \( WRL \propto L^{1.75} \)
2. between 2.4 and 6.0m \( WRL \propto L^{1.25} \)
3. over 6.0m \( WRL \propto L \)

Even though the length and vertical load factors used in the Code are independent they must be considered linked in the assessment of test data. Table 6.32 tabulates the plain wall test loads based on the Code method. The results show efficient modelling of the 5 kN/stud tests but the zero vertical load case is underestimated. This indicates firstly that the length factor is effective and secondly that the design was based on high vertical load
results to achieve safety when using the independent vertical load factor (see note C in 6.6.2). The power solution will now be investigated together with a linear based solution to determine the length modification factor for the standard design method. In the former case if \( KL \) is to have a value of one at 2.4m then a suitable equation for the modification factor will be:

\[
KL = \left( \frac{L}{2.4} \right)^x
\]  

- 6.28

Early work has shown that \( x \) will lie between 0 and 1 (such that \( WRL \) is proportional to \( L \) and \( L^2 \) respectively). The values of \( x \) used in the Code of Practice are 0.75 and 0.25 (equations 6.26 and 6.27); here the main advantage of the proposal is seen, in that very different equations can be applied for the two wall types so long as they give coincident values at the changeover point. The factor \( KL \) may now be calculated from all the results, in the knowledge that the vertical load factor is an accurate model of performance. A normalising process is used whereby the plain wall test results, the test racking resistance and the vertical load factor are inserted into the equation:

\[
TRL_{PW} = TRR \times L \times K110_{PW} \times KL
\]  

- 6.29

where subscript 'PW' refers to the plain wall. Assuming the value of \( KL \) to be unity for a standard panel then:

\[
TRL_{SP} = TRR \times 2.4 \times K110_{SP}
\]  

- 6.30

where subscript 'SP' refers to the standard panel. Dividing equation 6.29 by 6.30 and substituting equation 6.28 gives:

\[
\frac{TRL_{PW} \times K110_{SP}}{TRL_{SP} \times K110_{PW}} = \left( \frac{L}{2.4} \right)^{1+x}
\]  

- 6.31

Suitable values of \( x \) can be found by plotting the left hand side of the equation against \( (L/2.4) \) on a logarithm scale.
The design data is shown in Table 6.33 and indicates the accuracy of the K110 factor in the similarity between the TRL/K110 results for identical wall lengths. The results are plotted in Figures 6.21 a and b for the short and long wall length cases. The slope of the linear solution passing through the origin gives values of \((1 + x)\) from which it is found that the most suitable equations for K111 are:

\[
K_{111} = \left(\frac{L}{2.4}\right) \quad (L < 2.4\text{m}) \quad - 6.32
\]

\[
K_{111} = \left(\frac{L}{2.4}\right)^{0.38} \quad (2.4 < L < 4.8\text{m}) \quad - 6.33
\]

Table 6.34 shows calculated test loads for the plain walls which in comparison with the test results shown in Table 6.28 are very accurate throughout the range.

The same procedure can be used to determine a linear solution for walls longer than 1.2m. The solution is linear only in terms of the equation for K111 because the effect of the K110 factor will result in the design being linear only at zero vertical load. The left hand side of equation 6.31 is plotted against 'L' (Figure 6.22) such that the equation for the best fitting straight line represents \((K110 \times L)/2.4\), giving:

\[
K_{111} = 1.61 - \frac{1.39}{L} \quad - 6.34
\]

This is almost identical to equation 6.23 for K211 and it is proposed that the latter is used for design purposes so that the practical equivalent length formula can be used for design whereby for \(L > 1.2\text{m}:\)

\[
WRL = 1.6 \times BRR \times K_{110} \times (L - 0.9) \quad - 6.35
\]

The design values using this equation are shown in Table 6.35 and again correlate very well with the test results.
Comparing the linear method with that of the power curve the advantages of the former are:

(i) it is more simple to use,

(ii) for typical wall lengths only one equation is needed up to 4.8m after which it is likely that both methods will require a further equation for safety in extrapolation,

(iii) the same equation is appropriate to both the standard and simplified design methods,

(iv) extrapolation is safer because the racking resistance is constant in relation to length,

(v) it is more accurate for longer walls.

However, it is less accurate for shorter walls particularly at high vertical loads.

Consequently the linear solution for Klll is proposed for design. It is considered that in practical terms a solution for short panels is not necessary. If one is necessary then the factor derived for the simplified analysis (equation 6.24) could be used but a more accurate model is obtained if the linear solution is combined with the power curve for short panels. Figure 6.23 shows how the two equations (6.32 and 6.35) relate, the first intersection point can be shown to occur at 1.44m irrespective of vertical load such that the final design solution for Klll could be:

\[
K_{lll} = \frac{L}{2.4} \quad (L < 1.45m)
\]

\[
K_{lll} = 1.6 - \frac{1.44}{L} \quad (1.45 < L < 4.8m)
\]

- 6.36
6.6.4 **Plain Walls**

The remaining work on the length modification factor is considered in conjunction with the vertical load factor and therefore can be classified as affecting the design of plain walls. Four functions will be considered.

(i) The extrapolation of results to walls longer than 4.8m.

(ii) The safety factors incorporated into plain wall design.

(iii) Differences between test panel and practical wall behaviour.

(iv) A comparison with design values used in other countries.

If a linear solution is used for KL and it can be shown that the racking load can be applied continuously along the wall due to the aspect ratio of the horizontal diaphragm then, in theory, there should be no need to change the design method for longer walls. However, a requirement for increased safety when test results are extrapolated means that the design equation should be adjusted. Thus the upper bound solution is to use the same equation viz:

\[ WRL = BRR \times L \times (1.60 - \frac{1.44}{L}) \times K_{110} \]

The lower bound is to allow no increase in load after 4.8m which is unnecessarily restrictive and need only be the case if no information was available regarding longer walls. Some work on 7.2m walls has been carried out at Princes Risborough and, although it has not been published, the results have been used to confirm to the Code of Practice drafting committee that a linear load/length relationship for longer panels is acceptable. It is proposed therefore
that the \( K_{1110} \) factor for longer walls is directly related to the resistance at 4.8m. Hence:

\[
K_{111} = 1.60 - \frac{1.44}{4.8}
\]

giving:

\[
K_{111} = 1.3 \quad (L > 4.8m)
\]  

It is seen that at vertical loads other than zero the \( K_{1110} \) factor will make the racking load response non-linear. The \((2.4/L)^{0.4}\) factor (equation 6.16) reduces the vertical load behaviour in longer panels improving safety and therefore 'L' must refer to the full length of the wall. The long panel equation (6.37) may also be applied to the simplified wall design factor \( K_{211} \).

The application of safety factors in the design of timber frame walls must be checked for two separate conditions viz:

(1) that there is an overall factor of safety against failure, this is of major importance in wall design because the parameters have been based on stiffness results only,

(21) the maintenance of safety standards in the use of modification factors. This is essential as design will always be related to test behaviour measured on the comparatively short 2.4m panels.

In the first case the Code requires a factor of safety of 1.6 between the design load (representing a deflection of 0.003 times the panel height) and the failure load. The factor increases to 2.0 if the 5mm deflection load (representing a deflection of 0.002 times the panel height) is used.

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In testing, partial safety factors (K300) are included to cover reduced test information. Figures 6.24 a and b show the effect of the K300 factor on stiffness and strength performance. The graphs are most useful in determining the factors of safety required in standard panel tests where the stiffness and failure information is based on different numbers of identical tests, however, they may be used here to identify the requirements for factors of safety based on the test results. The design values for stiffness have been based on all the test results, regardless of the number of tests for each wall length, and therefore represent the mean test condition. For stiffness to govern design, a factor of safety of 2 must apply between the mean stiffness and the lowest failure load. Thus the difference between the design load and an average individual failure will be 2/0.75, i.e. 2.67. This means that the failure loads should exhibit an average factor of safety, when compared with the design load, of greater than 2.67 and that no factor of safety should be less than 2.00. The failure results of all the plain wall tests are brought together in Table 6.36. It must be noted that all the 1979 programme failure tests were conducted on walls with openings, when high factors of safety were recorded (i.e. > 2.67 on average), and that the 1985 tests were often stopped after a safety factor of two had been obtained in order to check more failure conditions. The results show that:

(i) the average safety factor is 2.48,

(ii) no safety factor is less than 2.00,

(iii) the factor of safety is independent of length.

Points (i) and (ii) indicate that to maintain an adequate factor of safety the test racking resistance for the programme should be reduced to 1.89 kN/m (i.e. 2.05 x 2.48/2.67).
This compares very well with the test racking resistance averaged for all the plywood and mediumboard sheathed standard panels which had a value of 1.86 kN/m. Thus the wall test programmes are an excellent representation of the standard panel results. The design proposal for basic racking resistance for plywood and mediumboard sheathings fixed with 3.25mm diameter nails is 1.82 kN/m whereas the tested values (1.25 x TRR) are 2.33 kN/m for the standard panels and 2.36 kN/m for the walls. The safety margin of 23% in design is to allow for variations in type of sheathing and in the quality of all the materials. It is notable that the individual results of the wall tests varied from the mean by less than 13% therefore it can be seen that the wall tests do not represent the overall variability that can be expected in the sheathing types.

Point (iii) above, is crucial to the analysis of the modification factors and shows that it is not necessary to make adjustments to either the length or vertical load factors as a result of failure performance. Thus in terms of overall strength, safety standards are being maintained. It can be argued that basing the modification factors on mean performance levels also indicates a maintenance of standards. However this could be a chance occurrence as a result of meaning the data from the three test programmes. The variation in the test results can be attributed to three factors:

(i) differences in basic racking resistance in the three sets of materials,

(ii) differences in vertical load and length response in the three programmes,

(iii) the reduced reliability in test information when fewer than five identical tests have been conducted.
It can be shown, based on the information contained in Figure 6.24 a, that the variability in reliability is within accepted tolerances. Clearly the K300 factor explains why the single and double test averages are often furthest from the mean line. The mean performance lines for the three programmes then indicate that the variation in results is due to point (i) and not point (ii) because the lines fan away from the mean such that the percentage difference is approximately constant. An analysis has been carried out on the data to show the variation in each programme from the mean, using Tables 6.28 and 6.35 but ignoring the shorter panels which do not fit so accurately the design equation. It indicates that the 1977, 1979 and 1985 tests vary by +5%, -11% and +12% about the mean suggesting their individual TRR values to be 1.98, 1.68 and 2.12 kN/m respectively.

The test results show that safety factors are maintained in the plain panel tests and therefore indicate the acceptability of the modification factors. However the standard panel tests showed that with stronger combinations of materials the improvement factors are likely to be reduced (Section 6.3.6). This could affect the safety of wall designs if the mean values are used for the modification factors particularly in the case of length where the 2.4m standard panel results may be greatly enhanced. The problem is illustrated by the 1985 test results which initially indicated a weaker vertical load and length response for a very strong material combination although these results were later shown to be more likely a consequence of the very limited test information for the shorter wall lengths.

The problem of using enhancement factors for very strong materials is examined for the three possible design cases, namely the use of:

1. datum racking resistance values,
2. test values for basic racking resistance,
3. test values for full scale walls.
In the first case the safety factor built into DRR ensures that there will be no problem. To achieve a very strong combination the materials must be above average and thus there is a 23% safety margin to counter any reduction in enhancement by vertical load or length. The same behaviour pattern shows an advantage to weak panels because they are likely to show greater improvement with length and will therefore increase their factor of safety in practical conditions.

Where the basic racking resistance is tested the design value can, either by chance or by conducting a full programme of tests, achieve the mean performance level for a 2.4m panel. If then, it cannot achieve the mean enhancement predicted by the modification factor there is no safety margin to fall back on and the wall design value could be compromised. It will therefore be necessary to overcome the problem by applying an additional reduction factor to the test results. In this way the safety margins will be brought more in line with those of the datum racking resistance which is a logical requirement because they are all extrapolated in typical design and the standard panel test result does not give any additional accuracy to the extrapolation procedure. A 0.87 safety factor is recommended based on the reduction of the test results when determining datum racking resistance, hence:

\[
BRR_{\text{design}} = 0.87 \times BRR_{\text{test}}
\]

This reduction factor is a very small penalty to pay for the use of design factors based on meaned test performance.

Full scale wall test results will not be extrapolated and therefore they can be used directly in design noting that they will include the normal partial safety factors.

One practical problem of wall design that can be investigated at this stage is the difference between the
test situation and the additional restraints found in a building. It is often argued that because the leading stud of a test wall is not tied back to another wall it is only partially restrained and thus the test load will be less than the practical capacity. Considering the design equation at zero vertical load based on equation 6.35, i.e.:

\[ W_{RL} = 1.6 \times BRR \times (L - 0.9) \]

it is possible that the lack of restraint causes the loss of 0.9m in the effective length and that a fully restrained wall would have a racking load given by:

\[ W_{RL,\text{restrained}} = 1.6 \times BRR \times L \]  \hspace{1cm} - 6.39

Thus the loss in load for the test case is 2.95 kN (Figure 6.25a). For a given length of wall it is possible to calculate what vertical load is required uniformly distributed to give this increase in load, i.e. use:

\[ 2.95 = 1.6 \times BRR \times (L - 0.9) \times (0.09F - 0.0015F^2) \left( \frac{2.4}{L} \right)^{0.4} \]

to find the unknown 'F'. The UDL can then be used to find the equivalent load that must be applied to the leading stud using the formula detailed in Section 6.8.5 where:

\[ F = \frac{2aF_p}{L^2} \]

In this case 'F_p' is the holding down force on the leading stud that has to be motivated to achieve full restraint and 'a' is the wall length, hence:

\[ F_{\text{restraint}} = \frac{FL}{2} \]  \hspace{1cm} - 6.40

The value of 'F_{\text{restraint}}' is length dependent but for typical walls of 4.8m or more it will need to be about 10 kN.
This force must be capable of being applied instantly to the panel; if it relies on nail slip in a connection, then, in the displacement during which the vertical load is being taken up, the wall may already have racked by the allowable amount. It is difficult to justify this type of restraint in most practical design cases and it is therefore advisable to use the design load based on the test result.

It is an interesting function of the linear solution that if identical panels are coupled into a wall (see Figure 6.25 b) the load contributed by the last panel will always be the same, i.e.:

\[
WRL_{nth\, panel} = WRL_{n \, panels} - WRL_{(n-1)\, panels} = 1.6 \times BRR \times L - 6.41
\]

Thus the restraint to the leading panel is the equivalent of placing a similar panel in front of it, noting that it is the rotation of the preceding panel that provides holding down to the subsequent panel and that a return wall does not offer a similar restraint. It is noticeable that the situation is different if vertical load is applied and that the contribution of successive panels decreases. This affects the leading stud restraint such that its performance varies with panel length and vertical load. Figure 6.25 c shows an hypothesis for the effect of the leading stud restraint on a vertically loaded panel whereby the racking resistance considered to apply to the restrained wall is based on the racking resistance calculated for the wall length and vertical load as given by:

\[
RR_{L,VL} = \frac{dy}{dL} (1.6 \times BRR \times (L - 0.9)[1 + (0.09F - 0.0015F^2) \left(\frac{2.4}{L}\right)^{0.4}]) - 6.42
\]

The possible increase in performance is then:

\[
RR_{L,VL} \times L - WRL_{L,VL}
\]
which is seen to decrease with length and vertical load and may reach zero. The hypothesis although not checked quantitatively is in agreement with practice in that the effectiveness of any restraint must decrease if an alternative, in the form of vertical load, is applied to the panel. Furthermore the longer the panel the stronger it becomes and thus the more difficult it is to increase its capacity.

The conditions analysed in this section are the limiting ones of either full restraint or no additional restraint (the test case). In practice return wall restraint will lie between these values and will require very careful consideration.

One further short programme of tests investigating the effect of wall length was carried out as part of the work on plasterboard panels. In these tests, reported by Randall (1981), the stiffness of two 2.4m long panels joined together was investigated. Because this test was an addition to the standard panel procedure, no failure tests could be carried out on the 4.8m wall. Thus the effect of panel length can only be determined by comparing the stiffness of the 4.8m wall with the average value for the same test on the 2.4m panels. Four types of panel were tested, 12.5mm standard plasterboard, 9.5mm and 12.5mm MR plasterboard and 12.5mm MR plasterboard with 9.0mm mediumboard. The standard panel results are given in Tables 6.2 and 6.5 and the 4.8m wall enhancement results are shown in Table 6.37.

The single plasterboard sheathed walls are reasonably uniform in their behaviour with the exception of the zero vertical load results for the 12.5mm MR board. On average their performance is more evenly matched by the proposed design than by the Code method. The design values for the 4.8m plasterboard panel at zero vertical load, based on the proposal for DRR given in Section A, is 5.62 kN which in all cases allows for a very low factor of safety against failure in the tests. This safety margin is significant because for internal walls plasterboard will be used in long plain panels, and often without vertical load.
The results for the combined sheathings show a particularly low improvement at zero vertical load and the 4.8m result represents a basic racking resistance of 1.82 kN/m compared with a proposed value, relating to the tested nail sizes, of 1.88 kN/m. There is also a strong possibility of a low failure load further reducing the margin of safety and casting doubt on the suitability of the proposed additional value of DRR for plasterboard. However, there is no such problem at higher vertical loads and little significance should be placed on the single test result.

In general there is little test evidence to assist the designer and thus the following conclusions are drawn.

(i) More test data is required concerning long plasterboard walls, particularly in the conditions in which they are used, i.e. as a lining, to a main sheathing, as a separating wall or as a double sheathing representing an internal wall.

(ii) Tests should concentrate on zero vertical load failure performance which is likely to be critical to design.

(iii) Unless the computer analysis can show a clear indication that the walls should, in theory, be stronger, the design value for the combination of the sheathing and lining may need to be reduced.

The problem will therefore be re-examined in Chapter 8 in the light of the theoretical solution.

In concluding the work on plain panels it is useful to compare the proposed design method with the design values
used in other countries. In all cases the design values are quoted as a load per metre run of panel for a given vertical load condition, although the current investigation shows racking resistance to vary quite considerably with length. The design methods are compared by noting the length of wall required using the current proposed method to match the design resistance quoted by the other sources. Thus if $RR_V$ is the racking resistance for a combination of materials (for which the calculated basic racking resistance is BRR) at a vertical load $V'$, and where 'Lc' is the comparative length, then:

$$RR_V \times Lc = BRR \times Lc \times K_{100} \times K_{111}$$

from which 'Lc' can be calculated.

The racking resistances noted in Chapter 3 are analysed below.

a) American Tests (Adams, 1983)
9.5mm Plywood : use BRR value of 1.82 kN/m
ASTM E-72 test on 2.4m panel
Design resistance = 2.9 kN/m (but it is uncertain whether the load applies to the test panel or a typical wall)

For an ASTM test it is necessary to fix the length and find the equivalent vertical load.
Hence if 'Lc' is 2.4m a vertical load of 7.5 kN/m is required, if 'Lc' is extended to 4.8m the vertical load reduces to 5 kN/m.

The results are easily justified in view of the high vertical load induced by the restraints used in the ASTM test but for practical vertical loads 2.9 kN/m is very high.

b) Britain (TRADA 1980)
9.5mm Plywood : use BRR value of 1.82 kN/m
Design value based on ASTM tests
Design resistance = 3.5 kN/m
If 'Lc' is 4.8m the equivalent vertical load required is approximately 10 kN/m. This value is too high for design purposes.

c) Australia (PAA, 1982, CSR, 1982)
9.5mm Plywood: use BRR value of 1.82 kN/m
Zero vertical load design value
Design resistance = 2.25 kN/m
'Lc' required = 4.8m
The result is a good match with the proposed design values.
12.5mm Plasterboard: use BRR value of 0.90 kN/m
Holding down bolts create an unknown vertical load
Design resistance = 2.5 kN/m
If 'Lc' is taken as 4.8m the vertical load requirement would be in excess of 15 kN/m.
The design value is very high and cannot be justified for British use. However, the BRR value includes a higher safety factor than that used with plywood.

d) New Zealand (SANZ 1984)
9.5mm Plywood: use BRR value of 1.82 kN/m
Holding down method equivalent of say 2.0 kN/m
Design values = 3.35 kN/m under 1.8m long and 4.15 kN/m over 1.8m.
3.35 kN/m at 1.8m represents a vertical load in excess of 15 kN/m and 4.15 kN/m at 4.8m represents a vertical load in excess of 12 kN/m. The racking resistances are very high and cannot be justified for British use. It is thought that the results are based very much on failure requirements and not a stiffness limit. Values based on the shorter panel are almost twice those proposed in the current investigation.

It can be seen that there is a wide range of values proposed throughout the world for racking resistance. Those proposed for use in Britain are comparatively low but have been based on a much wider test experience.
6.6.5 Openings

The effect of openings on wall performance is complex and will be influenced by a number of factors, such as:

(i) the size, shape and location of the openings,

(ii) the size, shape and location of the remaining sheathed panels,

(iii) the extra framing around the windows and the consequent additional fixings.

The most accurate assessment of the effect of openings must consider all these factors by taking account of every individual sheathing, framing and fixing element. This is done in the computer analysis (see Chapter 7) but it is far too complicated for an empirical design procedure. The current investigation requires a design method that is both simple in application and based on a property of the wall which is easy to assess.

One method would be to sum the parts of the wall once the sheathed area has been broken down into rectangles, as shown in Figure 6.26. Two assumptions are included in this method, they are as follows.

(i) Boards are applied to the walls in rectangular shapes rather than the openings being cut out from boards which is likely to be a much stronger arrangement.

(ii) Board joints within these rectangles do not reduce the performance. This assumption has been shown to be true in the plain wall tests for 0.6m boards but is possibly optimistic in the case of panels with openings where the boards are often very much narrower.
Figure 6.26 shows that the effect of the openings is to break down the wall into very small units which have been shown in the wall length tests to be very weak in comparison with larger walls because, approximately (because the function varies with vertical load), the relationship between racking load and length for short panels is given by:

\[ \text{WRL} \sim L^2 \]

Thus the sum of performances of the individual parts will be low even if the area of openings is small. The factor having greatest influence on performance will be the number of openings in a wall rather than the total size. Early tests on 2.4m panels positively indicated that the wall performance was much greater than the sum of the parts. Comparison between door and window openings further suggested that the enhancement achieved by the small area of panel under a window was far in excess of its proportionate size, thus performance was influenced by the shape of the opening as well as its size. It can be concluded that, by summing the individual parts, the analysis of the wall is an inefficient lower bound method which is only suitable if all the openings form total discontinuities within the wall as shown in Figure 6.26 b. A full height opening is an obvious discontinuity, however, further tests have shown that the longer the opening, the greater the depth of lintol that can be accepted whilst still creating the effect of a total discontinuity. This method of analysis would be simple, although relatively laborious, to apply to wall design and would also be independent of the position of the opening so that the racking resistance would be the same for loads applied at either end of the panel.

A second method of design is noted at this stage although full calculations have not been included. This method is most suited to walls built from modules. It determines a performance value for each module based on the preceding panel. The minimum value for the module is applied when it is at the windward end of the wall and
there is nothing ahead of it to prevent it uplifting under
racking load, the maximum value is obtained when it follows a
plain panel. Intermediate values are obtained dependent
on the depth of opening in the preceding panel when a
linear variation between the maximum and minimum values can
be applied. The behaviour of the plain panels is best
represented by a linear relationship such as that shown in
Figure 6.19 which has already been used in the simplified
plain panel design method. In this instance it will be
advantageous to use equations 6.22 and 6.25 instead of the
standard approach. Then the maximum and minimum design
values for plain panels will be as noted in Figure 6.27.
Thus the general equation for a plain panel is given by:

\[ P_{RL_{\text{plain}}} = 1.6 \times BRR \times K_{210} (L - 0.9 \left( \frac{h_o}{h} \right)_p ) - 6.45 \]

where \( P_{RL} \) is the panel racking load and \([h_o/h]_p\) refers to
the height of opening ratio of the previous panel.

The racking resistance of panels with door openings
is zero rated but window modules may be awarded a value
based on the height of sheathing under the window such that:

\[ P_{RL_{\text{window}}} = 1.6 \times BRR \times K_{210} (\frac{hp}{h})(L - 0.9 \left( \frac{h_o}{h} \right)_p ) - 6.46 \]

Full details are shown in Figure 6.28. The method of
determining the wall racking load from the sum of the panel
performance figures is shown in Figure 6.29. It can be
seen that the vertical load modification factor is common
to all parts of the summation and can be taken outside the
summation. This would have no effect on a UDL design case
where the load, as well as the modification factor, is
independent of length. However, the stud load condition
is length dependent (equation 6.12) and for design purposes
the \( K_{210} \) factor must relate to the wall and not the panel.
Thus the general design case for the wall is given by:
\[ \text{WRL} = \left( \leq \sum_{n=1}^{l} \text{PRL}_{n} \right) \times K210_{\text{wall}} \times \text{other applicable modification factors} - 6.47 \]

where \( \left( \leq \sum_{n=1}^{l} \text{PRL}_{n} \right) \) is the summation of the individual racking loads, under zero vertical load, of the 'n' panels in the wall; the term replaces \((\text{BRR} \times \text{KL} \times \text{Ko})\) in standard panel design.

The method of analysis is used to check the results of the 1979 test programme which used door and window modules. The zero vertical load design values for the four types of panel, noting the door panel to be zero rated, are given in Table 6.38 based on the test racking resistance (considering stiffness results only) for the 1979 combination of materials shown in Section 6.6.4 to be 1.83 kN/m (2.05 x 0.89). The theory has been used to predict all the 4.8m panel combination test performances. The quality of prediction is very good with analytical results averaging 95% of test values at zero vertical load and 98% at 5 kN/stud. There is one significant anomaly shown in the prediction of the plain panel results. Here the plain walls including two 0.6m long modules are 9% stronger than those using only 1.2m long panels. This trend is not substantiated either in practice or by standard theory. The error is in the prediction of the maximum value from the 0.6m long panel; two 0.6m panels tested together should, approximately, equal a single 1.2m panel, thus at zero vertical load the combination should total 0.89 kN making the value of the second panel 0.67 kN not the 1.69 kN shown in Table 6.38. The overestimation is due to the assumption that the maximum value for any plain panel must lie totally on the principal design line shown in Figure 6.27. This assumption is clearly wrong and has resulted in an overestimation of performance by up to 9% depending on the panel preceding the 0.6m module.

An alternative proposal has been investigated whereby for plain panels the maximum load capacity of a module is determined from the results of tests on two similar modules.
in combination. This may be shown as:

\[
WRL_{\text{second panel}} = WRL_{\text{combination}} - WRL_{\text{first panel}} - 6.48
\]

This alteration will have no effect if 'L' is greater than the short panel length, i.e. 1.2m modules will retain the value given in Table 6.38. However for short panels (assuming \( L > \text{short panel length}/2 \)) then:

\[
WRL_{\text{max}} = 1.6 \times BRR \times (2L - 0.9 - 0.125L^2)
\]

The results of this alteration are shown in Tables 6.38 and 6.39. The change reduces the overall performance at both vertical loads by 6% and the plain panel performance by 4%. With the correction applied, the model gives remarkably good estimates of the test values; the maximum differences are in the order of 20% which (Figure 6.24 a) is within the statistical limits for variation in a single test result. Overall, the results show an increased safety margin of 9.5% reducing slightly with vertical load.

The method of dealing with short panels will still provide some unusual results, as shown in Figure 6.30, but in the second design method they will always err on the side of safety when using combinations of short panels.

Initially it was stated that this method was suitable for modules only, however, by breaking the walls into units as shown in Figure 6.26 and ignoring discontinuities in sheathings, it could be applied to any wall unit. Thus the analysis could be checked with the 1985 test programme. This has not been done because the process is very laborious and not suitable for computer analysis. Since it can only be used with the simplified design method for panel length and vertical load the procedure does not meet the overall requirements of the analysis. Thus at this stage the approach has been abandoned in favour of an alternative solution that will be easier to use in practice. To emphasise the problem the 'module' method of design will normally give different loads for the wall depending on the
direction of loading; this is likely to be more accurate but will create further complications in the design process. Should a more rigorous approach to openings be required this method of analysis has many factors to commend it as a basis for design.

The third method of analysis considers the wall as a whole and relates a property of the openings to the loss in performance of the wall compared with the fully sheathed case. The assumptions for behaviour are similar to those made for the plain, or imperforate, walls. The property of the opening can be of varying complexity covering size, shape and position; the suitability of the property for use in design must balance ease of calculation with accuracy.

When work within the Sub-committee preparing the Code of Practice for Timber Frame Walls was directed to finding a suitable method for dealing with openings, it was decided that a reduction modification factor related to area of openings would be most suitable and that it should be independent of all other variables. Test information was extremely limited and, in the main, had been carried out on 2.4m panels which allowed either a door or a moderate size window to be included. Robertson and Griffiths (1981) related the percentage strength of the imperforate panel to the percentage of the opening using results from early British tests and from overseas work based on the ASTM test. They were unable to quote these values independent of vertical load and gave two sets of figures, one relating to a zero vertical load and the other to a 5 kN/m vertical load. Their findings are plotted in Figure 6.31 where it is seen that the results do not follow a pattern that could be identified by an equation. No guidance was given on how to interpret vertical load conditions between the two stated values. However, it is noticeable that the differences were small and for practical purposes the lower curve could be used for all vertical loads reducing the complexity of the overall wall design.
The draft for the Code of Practice circulated within the British Standards Sub-committee in 1984 included an independent opening modification factor, the values of which are also plotted in Figure 6.31. They had been determined with the benefit of the 1979 panel test results. It is therefore surprising to note that for small percentage areas of opening, racking performance was predicted to be higher than that previously quoted. A rapid fall off was also predicted so that walls with 60% openings were to be considered non structural. The values, however, cannot be considered in total isolation and it is probable that the higher factors were acceptable because other modification factors such as those for length and vertical load had been reduced from previous proposals.

In 1985 the author was invited to reassess the design method. In considering the panel opening modification factor in isolation, it was clear that the previous curves, none of which could be defined by an equation, could be replaced by a linear solution. Figure 6.31 shows a suitable modification factor (Ko) to be given by:

\[ Ko = 1 - 1.67p \]

where \( p = \frac{\text{Area of opening}}{\text{Total area of wall}} \)

This equation although fitting closely the previous curves did not correlate with the 1979 test results. The other modification factors had been altered with regard to this test data, thus it was necessary to achieve a closer correlation, particularly as the 1979 results gave a better indication of practical wall performance because they were based on tests on 4.8m long walls. Using both previous work and the 1979 results, suitable values for the modification factor, shown in Figure 6.31, were calculated using the equation:

\[ Ko = (1 - p)^2 \]

noting 'p' to have a maximum value of 0.75.
This equation was included in the draft Code of Practice (BSI, 1986) together with a number of restrictions on its use and guidance on its application, all of which are noted in Appendix 1. Essentially, these rules stated that:

(i) openings should be properly framed and walls should be capable of transferring horizontal load above and below the opening,

(ii) design equations must be modified if windows are within 300mm of the end of the wall,

(iii) openings separated by less than 300mm should be considered as single openings,

(iv) small openings can be neglected,

(v) wall performance can be calculated either in total or as the sum of parts, whichever is more beneficial to design.

Figure 6.32 gives examples of the application of some of these rules. The modification factor was given the notation K300.

It is notable that, except in the case of the power relationship \( (K_o = (1 - p)^2 \) ), walls with greater than 80% openings cannot carry racking load. In practice this is unlikely to be the case and design should revert to summing the component parts of the wall based on the principal sheathed areas. By considering only one property of the openings (viz their area) the method is unable to predict accurately for all types of openings. Two problem conditions are noted.

(i) Very large openings; if the majority of the area of openings is lost in one opening it is probable that there will be large
areas of plain wall which will be relatively strong. Thus the wall performance will be underestimated and hence the need for note (v) above.

(ii) Where openings are close together the plain panel linking them may be too small to make a contribution and then the panel performance may be overestimated hence notes (ii) and (iii) above.

The former problem relates to large area losses, whilst the latter is more significant with smaller areas of opening. Thus the rules will be of greater importance if designs are based on the linear solution rather than the power curve.

A method of analysis which considers the loss in performance of a wall, compared with the fully sheathed wall, to be related to a property of the openings is the most suitable for use with the design method proposed for plain walls. However, the relationship to the area of opening, although simple to apply, does not consider either the shape or position of the opening. Two further relationships are considered in the following investigation; they are as follows.

(i) Loss of full height wall which relates to the remaining length of full height wall. This case is similar to the loss of area approach; it is likely to be less accurate as it does not distinguish between windows and doors but it is a very simple parameter to calculate and apply to the design.

(ii) Loss of area moment. Here the first moment of area of the openings is calculated about the leeward end of the wall and noted
as a proportion of the area moment of the wall in total. Area moments are calculated about the vertical axis so as to simplify the calculations. The difference compared with taking area moment about the leeward lower corner of the wall is not significant. This approach should allow the importance of the position of the opening to be judged and, in practice, would mean that a wall's racking resistance would depend on the direction of loading.

It is notable that none of these approaches covers the shape of the opening. However, if the factors are based on test data covering both door and window openings, this should not be significant. It would be possible to cover shape by weighting the areas in terms of the depth of opening in both the loss of area and loss of moment area investigations. This would increase the difficulty in calculating the modification factor and would need to be considerably more accurate in prediction than alternative methods to justify its inclusion.

The required value for the opening modification factor (Ko) can be found using the general equation for wall racking load:

\[ WRL = BRR \times L \times KL \times KVL \times Ko \]

where KL and KVL, the modification factors for length and vertical load respectively, have already been assigned. For an imperforate wall Ko will be unity, thus for a practical wall the required modification factor will be given by:

\[ Ko \text{ reqd.} = \frac{\text{Wall test load}}{\text{Load for the identical test on an imperforate wall}} \]

where BRR, L, KL and KVL are the same for the two panels.
These results compare two different tests and the variability in testing will be reflected in the values obtained for Ko.

In calculating the required value for the modification factor the test data for all the 1979 and 1985 wall tests were compared with the meaned results for the imperforate walls such that the wall length, vertical load and combination of panel materials was identical for both the perforate and imperforate panels, i.e.:

\[
Ko \text{ reqd.} = \frac{\text{Wall Unit Test Result}}{\text{Meaned Plain Wall Performance}} - 6.51
\]

(Required Wall Modification Factor)

The required value of Ko is compared with a predicted value (Ko pred.) based on one of the three geometrical properties of wall noted earlier. To determine Ko pred. it is necessary to examine these properties in terms of the loss caused by the opening compared with the plain panels, such that:

\[
Ko \text{ pred.} = \frac{\text{Property of Sheathed Panels in Test Wall}}{\text{Property of Sheathed Panels in a Plain Wall}} - 6.52
\]

Figure 6.33 shows how these factors are determined for all three approaches and the calculated values from all the test walls are given in Table 6.40.

The accuracy of the equation for the prediction of Ko can be tested by comparing the value predicted by the geometric property with the value required by the test results as shown in Figure 6.34. The theory for the stiffness test (Figure 6.24a) suggests that the variability of a single test could be ±25% so for perfect modelling no results should fall outside these limits and the mean test response should fall on the design requirement line. For safety the test response must be above the design requirement line.
Initially, in the determination of the most suitable equation for the modification factor, the three functions shown in Figure 6.33 were plotted in their most simple form against $K_{o \text{ reqd}}$, i.e.

(i) $K_{o \text{ pred}} = q$ for full height panel effect

(ii) $K_{o \text{ pred}} = (1-p)$ for area effect

(iii) $K_{o \text{ pred}} = (1-m)$ for area moment effect.

In none of the cases were the results acceptable but the following points were of note.

(i) The correlation factors for the three cases (in the order noted above) were 0.92, 0.94 and 0.87 thus the area moment function, which was the most complicated to calculate, was the least accurate method of linking together all the results.

(ii) There was little variation in performance between tests on different panel lengths, vertical load or basic racking resistance which indicated that the values determined for those factors in the plain panel tests also applied to the wall opening tests and thus the modification factor for wall openings can be considered independent of all other factors.

It was decided that the loss of wall area approach was most suited to the standard modification factor and that the full height panel length so much simplified the calculation of the factor that it would be best for the simplified design approach.

The standard modification factor was determined using the formula:
The favoured value for 'b' was two and then 'a' was varied in increments of 0.05 to obtain a best fit solution when the mean test line just exceeded the safe interaction line. The trials showed a = 1.3 to be most suitable, for which, the results are plotted in Figure 6.35.

Secondary checks were carried out and the following observations made.

(i) If the meaned plain wall performance was substituted by the design plain wall performance calculated using the test racking resistance value for the test material and the design modification factors for vertical load and length, the required value for 'a' was not affected.

(ii) If the 1.2m panel results were omitted from the comparison (because they represent unrealistic walls in practice), the change in value of 'a' was insignificant, the correlation factor, however, was improved.

(iii) In the main analysis the area of opening was based on the sum of the areas of the individual panels. If the opening area was recalculated so that the frame members common to adjoining window openings (as shown in Figure 6.36) were included, the value for 'a' could be reduced to 1.25. This modification would be acceptable based on the rules set out in the draft Code but, in view of the specialist nature of the test panels (i.e. that they donot have a continuous lintel over openings extending into a second panel) it was thought that the more conservative value for 'a' of 1.3 should be retained in the design.
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(iv) The walls which included a long window opening flanked by 0.3m wide full height members were consistently weak in their response to loading due to the increased bending deflection in the thin pillar section. This weakness cannot be overcome unless the window corner zones are strengthened by continuing the sheathing over and under the windows (see Figure 6.37).

The proposed equation for the window opening modification factor is given as:

\[ K_{112} = (1 - 1.3p)^2 \]  \hspace{1cm} - 6.54

where \( p = \frac{\text{Area of openings in wall}}{\text{Total area of the wall}} \)  \hspace{1cm} - 6.55

Design values are compared with those of the draft Code in Table 6.40. The proposals represent a significant reduction in the present values. This is a result of the previous lack of data on openings on longer wall units.

The same approach was adopted for reviewing the full height wall length factor, but here more emphasis was placed on achieving a simple relationship. The equation finally adopted was:

\[ K_{212} = q^2 \]  \hspace{1cm} - 6.56

where \( q = \frac{\text{Length of full height wall}}{\text{Total length of wall}} \)  \hspace{1cm} - 6.57

for which the results are shown in Figure 6.38.

An examination of overall safety is necessary because the modification factors have been based on stiffness results alone. The failure loads shown in Tables 6.24 and 6.26 indicate higher factors of safety for imperforate walls than
for the plain walls previously considered. Therefore no
difficulties are foreseen in the use of the datum and basic
racking resistances discussed in Section 6.6.4, typically,
imperforate walls should exhibit greater factors of safety
and thus can accept higher basic racking resistance values
without worry of premature failure. Because stiffness is
the design criteria it is acceptable for the modification
factor to be based on meaned data, noting the high safety
margin built into the datum racking resistance values.
The correlation of the mean line is good (coefficient =
0.966 for K112) and nearly all the data for normal length
walls falls within 10% of the mean line.

The design values using K112 are low in comparison
to those previously based on K300 (see Table 6.40). This
is a major worry to designers as few external walls are
without openings. The results clearly indicate the weaknesses
caused by openings particularly where full height panels are
narrow. It is important that this more accurate assessment
of wall behaviour is retained to take account of the weaker
construction forms, such as single storey hall type structures
with few internal walls and no external cladding. However,
it may only be viable if a better assessment is made of the
path of the wind load to determine more accurately the forces
to which the imperforate wall will be subjected.

Walls with large areas of opening are calculated to
be particularly weak and in cases where the area is lost in
large openings the design method may be unnecessarily
restrictive; this is the penalty paid for employing a single
equation to cover a very complex condition. The option
remains to design the wall as the summation of parts if this
achieves a higher racking performance. Clearly, as the K112
factor is more restrictive more walls will be designed in
this way.

The factor for wall openings is a lower bound solution
based on the use of rectangular sheathings, which prevent the
strengthening of zones of weakness around window corners, and the use of modules which create further discontinuities in full height sheathings. It is anticipated that:

(i) walls with boards encircling openings (Figure 6.37) will improve strength,

(ii) walls built as single units from a minimum number of full height boards will possibly improve strength.

These hypotheses may best be checked using the computer based analysis. An introductory programme has already been carried out to examine point (i) although the full scale tests were primarily to prove the capabilities of the analytical design method detailed in Chapter 7. The work has been reported by Baughurst (1985) and examined different methods of sheathing around a 1.1 x 1.12m window opening in a 2.4m square panel. Two configurations were tested and comparison made with the computer analysis which also covered a third layout. The tests used a 9.5mm Canadian CSP plywood and a redwood/whitewood frame with the normal 63mm long, 3.25mm diameter gun driven wire nails fixed at standard centres. Details of the frame and the layout of the sheathing board are given in Figure 6.39.

The design values obtained in the tests are noted in Table 6.41 along with the predicted design values based on both the present method included in the Code and the proposed method. In both calculations BRR is taken from the P9 test result in Table 6.1 a and has a value of 2.34. The area of opening is 1.232m² giving values of 0.62 for K300 and 0.52 for K112. It is noticeable that the Code method greatly overestimates the results and that the proposed K112 factor is unsafe at higher vertical loads. This can be explained by two factors; firstly the basic racking resistance value is very high for plywood and reduces the safety margin built into the design method and secondly the modification factor is less suited to use with short walls.
The use of the C shaped sheathings increased performance by more than 45% with greater improvement the higher the vertical load. The window condition tested provides an optimum situation in having the sheathing continuous at the corners because its size does not require the use of additional small boards to complete the cladding under the window. The larger windows tested in America (Rose, undated) shown in Figure 3.15, with results included in Table 3.1, required supplementary sheathings and the percentage increase in the ASTM test was then only 31%. A further reason why the Surrey tests may be considered to have covered an optimum condition is that the length of full height sheathing preceding the panel was not reduced in order to achieve the continuous corner. If a 1.2m plain wall preceded the opening, it is unlikely that the enhancement would be so great if a 1.2m rectangular sheet followed by separate window sheathings was replaced by a 0.6m rectangular sheet followed by a 1.2m sheet continuous around the window corners as shown in Figure 6.41. Taking note of this, a factor to cover improvements in sheathing around windows is hypothesised (see Figure 6.40) whereby the encircling sheathing is considered to reduce the effective size of the opening. Then, if 'c' is the width of full height board and 'd' the width of the board continuous around the opening and the window height 'h_w' is less than half the total panel height (h/2) the length of window opening may be reduced by:

(i) \( d/4 \) if \( c \geq 300mm \)

(ii) \( d/2 \) if \( c > 600mm \)

Thus in Baughurst's panel the effective opening would be 0.55 x 1.12 giving \( K_{12} \) as 0.74 and the design values as 3.46, 4.95 and 6.12 kN at the three vertical loads which very accurately models the test results. This method clearly has promise but the factors used in equations 6.58 will require further verification before they can be adopted for use in design.
6.6.6 **Summary of Wall Design Factors**

Table 6.42 summarises the wall design modification factors determined for both the standard and simplified design methods in Section B. One factor that has not been covered in detail by testing is that dealing with variation in vertical load along the length of the wall. The Code of Practice notes the formula

\[ F = \frac{2a F_p}{L^2} \]

- 6.59

where 'F' is the equivalent uniformly distributed load for a concentrated load 'F_p' at a distance 'a' from the leeward end of the wall. The same equation can be adapted to cover variations in uniformly distributed load if 'F_p' is taken as the total load and 'a' relates to its centre of action. The use of equation 6.59 is shown in Figure 6.42.

No tests in the current work have investigated the effect of panel height. The modification factor K113 has been taken directly from the Code. The equation is theoretically based and has not been checked by test because of the very limited use made of non standard height panels.
<table>
<thead>
<tr>
<th>Wall Length</th>
<th>Individual Panels</th>
<th>Combination of Panels</th>
<th>5kN/stud Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5mm Defl.</td>
<td>Panel</td>
<td>5mm Defl.</td>
</tr>
<tr>
<td>0.6</td>
<td>0.42</td>
<td>A600</td>
<td>1.11</td>
</tr>
<tr>
<td></td>
<td>0.38</td>
<td>B600</td>
<td>1.08</td>
</tr>
<tr>
<td></td>
<td>0.26</td>
<td>C600</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>0.22</td>
<td>D600</td>
<td>1.19</td>
</tr>
<tr>
<td>1.2m</td>
<td>1.60</td>
<td>E1200</td>
<td>1.19</td>
</tr>
<tr>
<td></td>
<td>1.53</td>
<td>F1200</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>0.92</td>
<td>G1200</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>0.78</td>
<td>H1200</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>1.18</td>
<td>M1200</td>
<td>1.18</td>
</tr>
<tr>
<td>1.8m</td>
<td>4.46</td>
<td>K1800</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>2.36</td>
<td>L1800</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>4.46</td>
<td>A600+B600+C600</td>
<td>3.33</td>
</tr>
<tr>
<td></td>
<td>2.36</td>
<td>C600+D600</td>
<td>3.33</td>
</tr>
<tr>
<td>2.4m</td>
<td>5.53</td>
<td>I2400</td>
<td>9.09</td>
</tr>
<tr>
<td></td>
<td>4.89</td>
<td>J2400</td>
<td>9.09</td>
</tr>
<tr>
<td></td>
<td>4.58</td>
<td>N2400</td>
<td>9.09</td>
</tr>
<tr>
<td></td>
<td>4.25</td>
<td>O2400</td>
<td>9.09</td>
</tr>
<tr>
<td></td>
<td>5.53</td>
<td>E1200</td>
<td>9.09</td>
</tr>
<tr>
<td></td>
<td>4.14</td>
<td>C600+L1800</td>
<td>6.06</td>
</tr>
<tr>
<td></td>
<td>5.09</td>
<td>G1200+H1200</td>
<td>6.06</td>
</tr>
<tr>
<td></td>
<td>6.06</td>
<td>K1800+D600</td>
<td>6.06</td>
</tr>
<tr>
<td></td>
<td>4.61</td>
<td>L1800+D600</td>
<td>6.06</td>
</tr>
<tr>
<td>3.0m</td>
<td>7.57</td>
<td>C600+J2400</td>
<td>13.30</td>
</tr>
<tr>
<td></td>
<td>8.03</td>
<td>L2400+D600</td>
<td>13.30</td>
</tr>
<tr>
<td>3.6m</td>
<td>8.54</td>
<td>E1200</td>
<td>13.70</td>
</tr>
<tr>
<td></td>
<td>8.06</td>
<td>F1200</td>
<td>13.70</td>
</tr>
<tr>
<td></td>
<td>8.06</td>
<td>G1200</td>
<td>13.70</td>
</tr>
<tr>
<td></td>
<td>10.28</td>
<td>K1800</td>
<td>13.70</td>
</tr>
<tr>
<td></td>
<td>9.60</td>
<td>L1800</td>
<td>13.70</td>
</tr>
<tr>
<td></td>
<td>10.14</td>
<td>I2400</td>
<td>13.70</td>
</tr>
<tr>
<td>4.8m</td>
<td>17.44</td>
<td>E1200+J2400</td>
<td>24.66</td>
</tr>
<tr>
<td></td>
<td>13.00</td>
<td>M1200+P3600</td>
<td>24.66</td>
</tr>
<tr>
<td></td>
<td>12.80</td>
<td>I2400+J2400</td>
<td>24.66</td>
</tr>
<tr>
<td></td>
<td>13.36</td>
<td>N2400+O2400</td>
<td>24.66</td>
</tr>
<tr>
<td></td>
<td>12.64</td>
<td>Q2400+H2400</td>
<td>24.66</td>
</tr>
<tr>
<td></td>
<td>14.82</td>
<td>P3600+M1200</td>
<td>24.66</td>
</tr>
</tbody>
</table>

* Same panels and combinations used for 5kN/stud vertical load as zero vertical load.

Table 6.21 1977 Panel Combination Stiffness Test Results
<table>
<thead>
<tr>
<th>Wall Length m</th>
<th>Zero Vertical Load</th>
<th>5 KN/stud Vertical Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Panel(s)</td>
<td>Load at Failure KN</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>D600</td>
<td>1.46</td>
</tr>
<tr>
<td>1.2</td>
<td>H1200, A600 + B600</td>
<td>5.02</td>
</tr>
<tr>
<td>1.8</td>
<td>K1800, L1800</td>
<td></td>
</tr>
<tr>
<td>2.4</td>
<td>I2400, E1200 + F1200</td>
<td>12.46</td>
</tr>
<tr>
<td>3.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.8</td>
<td>Q4800, O2400 + N2400</td>
<td>33.98</td>
</tr>
</tbody>
</table>

Table 6.22 1977 Panel Combination Failure Test Results
Best Fitting Linear Solution
Normal Walls: \( y = 5.87x - 3.31 \)
Correlation Coefficient = 0.979
Short Panels: \( y = 0.53x \)
Best Fitting Quadratic Solutions
\( y = 0.39x + 1.10x \)

Panel Length (mm)

(a) Zero Vertical Load Case

Key
1 = Joint of boards on a single stud
+ = Joint between panels

(b) 5kN/Stud Vertical Load Case

Figure 6.6 1977 Panel Combination Test Results
<table>
<thead>
<tr>
<th>Wall Length (m)</th>
<th>Similar Tests</th>
<th>Zero Vertical Load</th>
<th>5kN/Stud Vertical Load</th>
<th>Vertical Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Meaned 5mm Defl. Load (kN)</td>
<td>Failure Load (kN)</td>
<td>Factor of Safety</td>
</tr>
<tr>
<td>0.6</td>
<td>4</td>
<td>a 0.32 b 1.46(s)</td>
<td>b/a 4.56</td>
<td>a/a₂ 0.064</td>
</tr>
<tr>
<td>1.2</td>
<td>7</td>
<td>a 1.19 5.20(s)</td>
<td>b 4.21</td>
<td>a/a₂ 0.239</td>
</tr>
<tr>
<td>1.8</td>
<td>5</td>
<td>a 3.19</td>
<td>b/c 0.641</td>
<td>c 5.53 d 14.40(s)</td>
</tr>
<tr>
<td>2.4</td>
<td>11</td>
<td>a 4.98 12.46(s)</td>
<td>b/c 2.50 2.26</td>
<td>a/a₂ 1.000</td>
</tr>
<tr>
<td>3.0</td>
<td>2</td>
<td>a 7.80</td>
<td>b/c 1.566</td>
<td>c 13.22</td>
</tr>
<tr>
<td>3.6</td>
<td>6</td>
<td>a 9.42</td>
<td>b/c 1.892</td>
<td>c 14.10 d 33.70(s)</td>
</tr>
<tr>
<td>4.8</td>
<td>7</td>
<td>a 13.87 33.98(s)</td>
<td>b/c 2.45 2.23</td>
<td>a/a₂ 2.785</td>
</tr>
</tbody>
</table>

Notes: (s) single panel  (c) combined panel
Basic Racking Resistance for 2.4mm Panel using Partial Safety Factor of 0.87 = 2.25kN/m.

Table 6.23 1977 Plain Panels in Combination - Reduced Results.
Figure 6.7 Panel Details: 1979 Test Programme
<table>
<thead>
<tr>
<th>Panel</th>
<th>Racking Load (kN)</th>
<th>5kN/stud Vertical Load</th>
<th>5kN/stud Vertical Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>First Cycle</td>
<td>Worst Cycle</td>
<td>First Cycle</td>
</tr>
<tr>
<td></td>
<td>Zero Vertical Load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plain</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A600</td>
<td>0.34</td>
<td>0.32</td>
<td>0.83</td>
</tr>
<tr>
<td>B600</td>
<td>0.19</td>
<td>0.15</td>
<td>0.82</td>
</tr>
<tr>
<td>C600</td>
<td>0.31</td>
<td>0.30</td>
<td>0.69</td>
</tr>
<tr>
<td>D600</td>
<td>0.23</td>
<td>0.22</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>E1200</td>
<td>1.08</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td>F1200</td>
<td>0.83</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td>G1200</td>
<td>1.01</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>H1200</td>
<td>0.90</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td>I1200</td>
<td>1.01</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>J1200</td>
<td>0.88</td>
<td>0.83</td>
</tr>
<tr>
<td></td>
<td>Door</td>
<td>NOT MEASURED</td>
<td>NOT MEASURED</td>
</tr>
<tr>
<td></td>
<td>K1200</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>L1200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Window</td>
<td>M1200</td>
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<td>0.14</td>
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<tr>
<td></td>
<td>N1200</td>
<td>0.16</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>O1200</td>
<td>0.16</td>
<td>0.16</td>
</tr>
<tr>
<td></td>
<td>P1200</td>
<td>0.18</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Table 6.24 1979 Panel Combination and Opening Test Results.

a: Individual Panels.
<table>
<thead>
<tr>
<th>Panel Combination</th>
<th>Type of Combination</th>
<th>Racking Load (kN)</th>
<th>Zero Vertical Load</th>
<th>5kN/Stud Vertical Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>5mm Defl. Load</td>
<td>Failure Load</td>
<td>Factor of Safety</td>
</tr>
<tr>
<td>EF</td>
<td>P+P</td>
<td>3.38</td>
<td>4.45</td>
<td>0.63</td>
</tr>
<tr>
<td>G1</td>
<td>P+P</td>
<td>12.35</td>
<td>11.52</td>
<td>11.20</td>
</tr>
<tr>
<td>NH</td>
<td>P+P</td>
<td>11.54</td>
<td>11.54</td>
<td>12.08</td>
</tr>
<tr>
<td>AEF1B</td>
<td>H+P+P+P+P+P</td>
<td>11.32</td>
<td>11.32</td>
<td>13.56</td>
</tr>
<tr>
<td>CEF1D</td>
<td>H+P+P+P+P+P</td>
<td>5.67</td>
<td>5.67</td>
<td>5.67</td>
</tr>
<tr>
<td>G1JIJD</td>
<td>H+P+P+P+P+P</td>
<td>8.20</td>
<td>8.20</td>
<td>8.20</td>
</tr>
<tr>
<td>CXEF0D</td>
<td>H+P+P+P+P+P</td>
<td>10.93</td>
<td>10.93</td>
<td>10.93</td>
</tr>
<tr>
<td>G1NH</td>
<td>P+P+P+P+P+P</td>
<td>5.66</td>
<td>5.66</td>
<td>5.66</td>
</tr>
<tr>
<td>GP1H</td>
<td>P+P+P+P+P+P</td>
<td>20.43</td>
<td>3.61</td>
<td>8.84</td>
</tr>
<tr>
<td>GI0H</td>
<td>P+P+P+P+P+P</td>
<td>20.43</td>
<td>3.61</td>
<td>8.84</td>
</tr>
<tr>
<td>G1JH</td>
<td>P+P+P+P+P+P</td>
<td>5.08</td>
<td>5.08</td>
<td>5.08</td>
</tr>
<tr>
<td>GKLH</td>
<td>P+P+P+P+P+P</td>
<td>2.99</td>
<td>2.82</td>
<td>2.82</td>
</tr>
<tr>
<td>GJLH</td>
<td>P+P+P+P+P+P</td>
<td>7.53</td>
<td>7.53</td>
<td>7.53</td>
</tr>
<tr>
<td>CPKOD</td>
<td>H+P+P+P+P+P+P+P+P+P</td>
<td>3.43</td>
<td>3.43</td>
<td>3.43</td>
</tr>
<tr>
<td>CPKJD</td>
<td>H+P+P+P+P+P+P+P+P+P</td>
<td>5.01</td>
<td>5.01</td>
<td>5.01</td>
</tr>
<tr>
<td>CPKPD</td>
<td>H+P+P+P+P+P+P+P+P+P</td>
<td>5.70</td>
<td>5.70</td>
<td>5.70</td>
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<td>4.94</td>
<td>4.94</td>
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<tr>
<td>CHKND</td>
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<td>2.40</td>
<td>2.40</td>
</tr>
<tr>
<td>CJJKP</td>
<td>H+P+P+P+P+P+P+P+P+P</td>
<td>2.32</td>
<td>2.32</td>
<td>2.32</td>
</tr>
<tr>
<td>CJKPD</td>
<td>H+P+P+P+P+P+P+P+P+P</td>
<td>4.60</td>
<td>13.42</td>
<td>2.92</td>
</tr>
<tr>
<td>CGLH</td>
<td>P+P+P+P+P+P+P+P+P+P</td>
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<td>3.65</td>
<td>3.65</td>
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<td>GOLH</td>
<td>P+P+P+P+P+P+P+P+P+P</td>
<td>4.70</td>
<td>4.70</td>
<td>4.70</td>
</tr>
</tbody>
</table>

* Key to type of combinations: H = 0.6m plain panel  D = 1.2m door panel  P = 1.2m plain panel  W = 1.2m window panel.

Tables 6.24 1979 Panel Combination and Opening Test Results.

b: Panel Combinations.
Figure 6.8 Panel Combination Details - 1979 Test Programme
Figure 6.9 1979 Panel Combination Tests: Stiffness Results

(a) Zero Vertical Load Case

(b) 5kN/Stud Vertical Load Case
<table>
<thead>
<tr>
<th>Wall Length (m)</th>
<th>Similar Tests</th>
<th>Zero Vertical Load</th>
<th>5kN/Stud Load</th>
<th>Vertical Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Measured 5mm Defl. Load (kN)</td>
<td>Length Factor</td>
<td>Measured 5mm Defl. Load (kN)</td>
</tr>
<tr>
<td>0.6</td>
<td>4</td>
<td>a</td>
<td>a/a₂</td>
<td>b</td>
</tr>
<tr>
<td>1.2</td>
<td>6</td>
<td>0.25</td>
<td>0.063</td>
<td>0.78</td>
</tr>
<tr>
<td>2.4</td>
<td>2</td>
<td>0.86</td>
<td>0.219</td>
<td>1.98</td>
</tr>
<tr>
<td>4.8</td>
<td>8</td>
<td>3.92</td>
<td>1.000</td>
<td>6.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11.86</td>
<td>3.026</td>
<td>17.77</td>
</tr>
</tbody>
</table>

Notes: Basic Racking Resistance for 2.4m Panel Using Partial Safety Factor of 0.87 = 1.78kN/m.

Table 6.25 1979 Plain Panels in Combination - Reduced Results.
Figure 6.10 Panel Details - 1985 Test Programme

- A1 = A8
- B1 = B2
- C1 = C2
- D1 = D2 (handed)
- E9 and E11
- F9 and F11 (handed)
- G1
- H1 (handed)
- G2
- H2 (handed)

two rectangular boards

single 'L' shaped board
Figure 6.11 Panel Combination Details - 1985 Test Programme
<table>
<thead>
<tr>
<th>Panels</th>
<th>Length (m)</th>
<th>Zero Vertical Load</th>
<th>2.5kN/stud Vertical</th>
<th>5.0kN/stud Vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>5mm Defl. Load(kN)</td>
<td>Maximum Load(kN)¹</td>
<td>Factor of Safety</td>
</tr>
<tr>
<td>A1</td>
<td>1.2</td>
<td>-</td>
<td>-</td>
<td>1.95</td>
</tr>
<tr>
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<td>1.2</td>
<td>-</td>
<td>-</td>
<td>2.65</td>
</tr>
<tr>
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<td>1.2</td>
<td>1.95</td>
<td>-</td>
<td>2.95</td>
</tr>
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<td>1.55</td>
<td>-</td>
<td>2.32</td>
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<td>1.2</td>
<td>-</td>
<td>-</td>
<td>2.53</td>
</tr>
<tr>
<td>A6</td>
<td>1.2</td>
<td>-</td>
<td>-</td>
<td>2.56</td>
</tr>
<tr>
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<td>1.2</td>
<td>-</td>
<td>-</td>
<td>2.46</td>
</tr>
<tr>
<td>A8</td>
<td>1.2</td>
<td>-</td>
<td>-</td>
<td>2.50</td>
</tr>
<tr>
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<td>6.77</td>
<td>-</td>
<td>8.79</td>
</tr>
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<td>-</td>
<td>8.20</td>
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<td>A5+A6</td>
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<td>-</td>
<td>-</td>
<td>8.50</td>
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<td>A7+A5</td>
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<td>-</td>
<td>-</td>
<td>8.62</td>
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<tr>
<td>A5+A6+A7</td>
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<td>21.41²</td>
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<tr>
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<td>14.16</td>
<td>29.18</td>
<td>20.33</td>
</tr>
<tr>
<td>A5+A6+A7+A8</td>
<td>4.8</td>
<td>-</td>
<td>-</td>
<td>18.07</td>
</tr>
</tbody>
</table>

Note 1. Test stopped after required factor of safety had been achieved unless failure noted.
Note 2. Panel combination loaded to failure.

Tables 6.26 1985 Panel Combination and Opening Test Results
a: Plain Walls.
<table>
<thead>
<tr>
<th>Panels</th>
<th>Length (m)</th>
<th>Zero Vertical Load</th>
<th>2.5kN/stud Vertical</th>
<th>5.0kN/stud Vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Smn Def. Load(kN)</td>
<td>Maximum Load(kN)</td>
<td>Factor of Safety</td>
</tr>
<tr>
<td>E9</td>
<td>1.2</td>
<td>-</td>
<td>0.85</td>
<td>-</td>
</tr>
<tr>
<td>E11</td>
<td>1.2</td>
<td>0.66</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>F9</td>
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</tr>
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<td>-</td>
<td>0.82</td>
<td>-</td>
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<td>-</td>
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<td>1.96</td>
<td>-</td>
</tr>
<tr>
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<td>1.2</td>
<td>-</td>
<td>1.91</td>
<td>-</td>
</tr>
<tr>
<td>G2</td>
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<td>1.00</td>
<td>1.49</td>
<td>-</td>
</tr>
<tr>
<td>G1+H1</td>
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<td>4.05</td>
<td>5.63</td>
<td>-</td>
</tr>
<tr>
<td>G2+H2</td>
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<td>-</td>
<td>4.54</td>
<td>-</td>
</tr>
<tr>
<td>A5+F11</td>
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<td>-</td>
<td>4.86</td>
<td>-</td>
</tr>
<tr>
<td>E11+A8</td>
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<td>-</td>
<td>4.79</td>
<td>-</td>
</tr>
<tr>
<td>F9+D2</td>
<td>2.4</td>
<td>-</td>
<td>1.53</td>
<td>-</td>
</tr>
<tr>
<td>F9+C1</td>
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<td>-</td>
<td>1.42</td>
<td>-</td>
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<tr>
<td>D1+E9</td>
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<td>-</td>
<td>1.28</td>
<td>-</td>
</tr>
<tr>
<td>A5+G1+C2 +A6</td>
<td>3.6</td>
<td>3.04</td>
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<td>9.66</td>
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<td>3.17</td>
<td>4.55</td>
<td>5.18</td>
</tr>
<tr>
<td>D2+E8+A8</td>
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<td>3.03</td>
<td>4.40</td>
<td>5.02</td>
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<td>F11+B1+E11</td>
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<td>1.65</td>
<td>2.75</td>
<td>2.45</td>
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<td>-</td>
<td>4.76</td>
<td>3.31</td>
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<td>1.96</td>
<td>-</td>
<td>3.77</td>
</tr>
<tr>
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<td>-</td>
<td>7.22</td>
<td>14.72</td>
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<td>9.38</td>
<td>-</td>
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<tr>
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<td>-</td>
<td>7.76</td>
<td>-</td>
</tr>
<tr>
<td>A7+D2+E11+AB</td>
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<td>16.93</td>
<td>11.21</td>
<td>-</td>
</tr>
<tr>
<td>A5+F1+C2+A6</td>
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<td>7.93</td>
<td>7.96</td>
<td>12.15</td>
</tr>
<tr>
<td>D2+C2+F9+E9</td>
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<td>-</td>
<td>7.97</td>
<td>-</td>
</tr>
<tr>
<td>D1+E3+B2+A4</td>
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<td>-</td>
<td>9.22</td>
<td>-</td>
</tr>
<tr>
<td>A1+D1+B2+C1</td>
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<td>-</td>
<td>8.56</td>
<td>-</td>
</tr>
<tr>
<td>G2+N2+D1+C1</td>
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<td>5.10</td>
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<td>-</td>
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<tr>
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<td>7.55</td>
<td>9.97</td>
<td>-</td>
</tr>
<tr>
<td>F11+B1+E11+C1</td>
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<td>16.02</td>
<td>2.12</td>
<td>10.19</td>
</tr>
<tr>
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<td>4.47</td>
<td>2.11</td>
<td>2.91</td>
</tr>
<tr>
<td>F9+D2+G2+H2</td>
<td>4.8</td>
<td>-</td>
<td>6.50</td>
<td>-</td>
</tr>
<tr>
<td>G2+N2+C2+E9</td>
<td>4.8</td>
<td>-</td>
<td>6.15</td>
<td>-</td>
</tr>
<tr>
<td>F9+D1+D1+E9</td>
<td>4.8</td>
<td>5.32</td>
<td>2.35</td>
<td>3.19</td>
</tr>
<tr>
<td>D1+E9+D9+C1</td>
<td>4.8</td>
<td>-</td>
<td>2.84</td>
<td>-</td>
</tr>
</tbody>
</table>

Note 1. Test stopped after required factor of Safety had been achieved.

Tables 6.26 1985 Panel Combination and Opening Test Results
b: Walls with Openings
Best Fitting Linear Solution
Zero Vertical Load
as tested $y = 3.41x - 2.14$
modified $y = 3.30x - 1.61$
5kN/stud vertical load
as tested $y = 4.75x - 2.30$
modified $y = 4.63x - 1.69$

Key
- - - as tested
- - - - modified

N.B. See text for modification details

(a) Zero and 5kN/Stud Vertical Load Cases

Panel Length (m)

Best Fitting Linear Solution
2.5kN/Stud Vertical Load
as tested $y = 4.49x - 2.63$
modified $y = 4.44x - 2.24$

Key
- - - as tested
- - - - modified

N.D. See test for modification details

(b) 2.5kN/Stud Vertical Load Case

Figure 6.12 1985 Panel Combination Tests: Stiffness Results
<table>
<thead>
<tr>
<th>Wall Length (m)</th>
<th>Zero Vertical Load 2.5kW/Std Load</th>
<th>5kW/Std Load</th>
<th>Similar Tests Mean 5mm Def. Load (kN)</th>
<th>Mean Length Factor</th>
<th>Length Factor 5mm Def. Load (kN)</th>
<th>5kW/Std Load</th>
<th>2 kW/Std Load</th>
<th>Vertical Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2</td>
<td>2-2-2</td>
<td>1.66</td>
<td>a</td>
<td>a/\sqrt{2}</td>
<td>b</td>
<td>b/2</td>
<td>c</td>
<td>c/c_2</td>
</tr>
<tr>
<td>2-4-1</td>
<td>1.66</td>
<td>2.46</td>
<td>b</td>
<td>b/2</td>
<td>c/c_2</td>
<td>c/c_2</td>
<td>c</td>
<td>c/c_2</td>
</tr>
<tr>
<td>2.4</td>
<td>1-4-1</td>
<td>6.57</td>
<td>0.252</td>
<td>0.292</td>
<td>3.05</td>
<td>0.327</td>
<td>1.49</td>
<td>1.84</td>
</tr>
<tr>
<td>2.4</td>
<td>1-4-1</td>
<td>6.57</td>
<td>0.252</td>
<td>0.292</td>
<td>3.05</td>
<td>0.327</td>
<td>1.49</td>
<td>1.84</td>
</tr>
<tr>
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<td>1-4-1</td>
<td>9.83</td>
<td>1.496</td>
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<td>8.53</td>
<td>9.34</td>
<td>1.42</td>
<td>1.50</td>
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<td>1-4-1</td>
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<td>2.081</td>
<td>2.197</td>
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<td>1-4-1</td>
<td>13.67</td>
<td>2.081</td>
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<td>2.081</td>
<td>2.197</td>
<td>1.30</td>
<td>1.576</td>
</tr>
</tbody>
</table>

Notes: Basic Racking Resistance for 2.4m Panel Using Partial Safety Factor of 0.87 = 2.39kW/m.

Table 6.27 1985 Plain Panels in Combination - Reduced Results.
b) Length Factor At Zero Vertical Load

Key
- 1977 tests
- 1979 tests
- 1985 tests
--- K100 factor x wall length

--- 2.4m

Figure 6.13 Comparison of Test Results for the Three Wall Programmes
Figure 6.14 Combined Plain Wall Test Results
### Table 6.28 Determination of Mean Performance Values for Plain Wall Tests

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>0.6</th>
<th>1.2</th>
<th>1.8</th>
<th>2.4</th>
<th>3.0</th>
<th>3.6</th>
<th>4.8</th>
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<tbody>
<tr>
<td>0.6</td>
<td>0.32</td>
<td>0.25</td>
<td>-</td>
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<td>1.12</td>
<td>0.78</td>
<td>-</td>
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<tr>
<td></td>
<td>(4)</td>
<td>(4)</td>
<td></td>
<td>(8)</td>
<td>(4)</td>
<td>(4)</td>
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</tr>
<tr>
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<td>1.19</td>
<td>0.86</td>
<td>1.66</td>
<td>1.08</td>
<td>2.67</td>
<td>1.98</td>
<td>3.05</td>
</tr>
<tr>
<td></td>
<td>(7)</td>
<td>(6)</td>
<td>(1)</td>
<td>(14)</td>
<td>(7)</td>
<td>(6)</td>
<td>(1)</td>
</tr>
<tr>
<td>1.8</td>
<td>3.19</td>
<td>3.19</td>
<td>3.19</td>
<td>3.19</td>
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<td>(5)</td>
<td></td>
<td>(5)</td>
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<td>(5)</td>
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<td>4.98</td>
<td>3.92</td>
<td>6.57</td>
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<td>(2)</td>
<td>(1)</td>
<td>(14)</td>
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<td>(2)</td>
<td>(2)</td>
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<td>(2)</td>
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<tr>
<td></td>
<td>(6)</td>
<td>(1)</td>
<td>(7)</td>
<td>(7)</td>
<td>(6)</td>
<td>(1)</td>
<td>(1)</td>
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<tr>
<td>4.8</td>
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<td>11.86</td>
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<td>19.63</td>
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<td>(7)</td>
<td>(8)</td>
<td>(1)</td>
<td>(16)</td>
<td>(7)</td>
<td>(8)</td>
<td>(1)</td>
</tr>
</tbody>
</table>

Note: Figures in brackets indicate number of tests in average result.

Table 6.28 Determination of Mean Performance Values for Plain Wall Tests
Figure 6.15 The Effect of Length on the Original Vertical Load Modification Factor

Figure 6.16 The Effect of Length on the $K_{V/L}$ Code of Practice Vertical Load Modification Factor
<table>
<thead>
<tr>
<th>Wall Length (m)</th>
<th>Factor Required by Test</th>
<th>Code Factor K400</th>
<th>Standard Design Factor K110</th>
<th>Simplified Design Factor K210</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>3.39</td>
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<td>2.88</td>
<td>1.86</td>
</tr>
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<td>1.2</td>
<td>2.22</td>
<td>1.89</td>
<td>2.18</td>
<td>1.70</td>
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<td>1.73</td>
<td>1.82</td>
<td>1.92</td>
<td>1.64</td>
</tr>
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<td>2.4</td>
<td>1.77</td>
<td>1.77</td>
<td>1.77</td>
<td>1.61</td>
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<td>3.0</td>
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<td>1.75</td>
<td>1.68</td>
<td>1.59</td>
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<td>1.62</td>
<td>1.58</td>
</tr>
<tr>
<td>4.8</td>
<td>1.54</td>
<td>1.71</td>
<td>1.54</td>
<td>1.56</td>
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</table>

(a) Plain walls

<table>
<thead>
<tr>
<th>Length</th>
<th>Vertical Load Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1979 Tests</td>
</tr>
<tr>
<td></td>
<td>Walls with Openings</td>
</tr>
<tr>
<td>1.2</td>
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</tr>
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<td>2.4</td>
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<td>3.6</td>
<td>1.57</td>
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<tr>
<td>4.8</td>
<td>1.50</td>
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</table>

(b) Tested values for walls with openings

Table 6.29 Vertical Load Modification Factors for a 5kN/Stud Loading
Figure 6.17 The Effect of Length on the K_{110} Standard and K_{210} Simplified Vertical Load Modification Factors
<table>
<thead>
<tr>
<th>Length</th>
<th>Zero Vertical Load K110 and K210</th>
<th>2.5kN/Stud</th>
<th>5.2kN/m UDL</th>
<th>5kN/Stud</th>
<th>10.4kN/m UDL</th>
</tr>
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<tbody>
<tr>
<td>0.6</td>
<td>1.0</td>
<td>2.13</td>
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<td>1.74</td>
<td>1.31</td>
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<td>1.0</td>
<td>1.75</td>
<td>1.39</td>
<td>1.56</td>
<td>1.31</td>
</tr>
<tr>
<td>1.8</td>
<td>1.0</td>
<td>1.50</td>
<td>1.35</td>
<td>1.47</td>
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<td>1.33</td>
<td>1.43</td>
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<td>1.32</td>
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<td>1.36</td>
<td>1.31</td>
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<td>1.30</td>
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<td>1.31</td>
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<td>1.30</td>
<td>1.31</td>
</tr>
</tbody>
</table>

Table 6.30 Vertical Load Modification Factors for Common Vertical Load Cases
Racking Load at 5mm Deflection

Wall length

- constantly increasing resistance accelerating increase in load
- consistent resistance
- resistance decreases in longer panels, eventually a maximum load is reached

Figure 6.18 Prediction of the Effect of Length on Racking Performance for a Constant Stud Load Condition
(i) 2.4m panel results fixed by basic racking resistance and vertical load modification factor.

(ii) Long wall equation = \( WRL = BRR \times (aL-b) \times K_{VL} \)

(iii) Short wall equation = \( WRL = BRR \times (aSL-b) \times \frac{L}{SL} \times K_{VL} \)

(iv) Out of test range wall stiffness is reduced to allow safe extrapolation. The slope of the lines should be based either on test information or computer analysis. If no information is available, the slope of the lines may need to be reduced to zero. (See text for further details).

(v) The lines refer to three vertical load conditions.

Figure 6.19 A Linear Solution To The Effect of Wall Length
Mean Line (ignoring 0.6 results)
\( Y = 3.245x - 2.736 \)
Correlation Coefficient = 0.999

Figure 6.20 Analysis of Test Results to Determine a Linear Solution for Wall Length Modification Factors
Table 6.31 Plain Panel Test Loads using the Simplified Design Approach

<table>
<thead>
<tr>
<th>L (m)</th>
<th>Eff L (m)</th>
<th>KL</th>
<th>TRR (kN/m)</th>
<th>K210 (kN/stud)</th>
<th>TRL (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>0.075</td>
<td>1.60</td>
<td>2.05</td>
<td>1.0</td>
<td>0.25</td>
</tr>
<tr>
<td>1.2</td>
<td>0.3</td>
<td>1.60</td>
<td>2.05</td>
<td>1.0</td>
<td>0.98</td>
</tr>
<tr>
<td>1.8</td>
<td>0.9</td>
<td>1.60</td>
<td>2.05</td>
<td>1.0</td>
<td>2.95</td>
</tr>
<tr>
<td>2.4</td>
<td>1.5</td>
<td>1.60</td>
<td>2.05</td>
<td>1.0</td>
<td>4.92</td>
</tr>
<tr>
<td>3.0</td>
<td>2.1</td>
<td>1.60</td>
<td>2.05</td>
<td>1.0</td>
<td>6.89</td>
</tr>
<tr>
<td>3.6</td>
<td>2.7</td>
<td>1.60</td>
<td>2.05</td>
<td>1.0</td>
<td>8.86</td>
</tr>
<tr>
<td>4.8</td>
<td>3.9</td>
<td>1.60</td>
<td>2.05</td>
<td>1.0</td>
<td>12.79</td>
</tr>
</tbody>
</table>

TRL = TRR x K210 x K212 x L
= KL x TRR x K210 x Eff L
KL = 1.6, Eff L = (L - 0.9)
K212 = 1 + 0.06F, where F = \( \frac{V}{0.6} + \frac{V}{L} \)

Table 6.32 Plain Panel Test Loads using the Original Draft Code of Practice Design Approach

<table>
<thead>
<tr>
<th>L (m)</th>
<th>K100</th>
<th>K400</th>
<th>TRR (kN/m)</th>
<th>TRL (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>0.35</td>
<td>1.0</td>
<td>2.08</td>
<td>2.05</td>
</tr>
<tr>
<td>1.2</td>
<td>0.59</td>
<td>1.0</td>
<td>1.89</td>
<td>2.05</td>
</tr>
<tr>
<td>1.8</td>
<td>0.81</td>
<td>1.0</td>
<td>1.82</td>
<td>2.05</td>
</tr>
<tr>
<td>2.4</td>
<td>1.00</td>
<td>1.0</td>
<td>1.77</td>
<td>2.05</td>
</tr>
<tr>
<td>3.0</td>
<td>1.06</td>
<td>1.0</td>
<td>1.75</td>
<td>2.05</td>
</tr>
<tr>
<td>3.6</td>
<td>1.11</td>
<td>1.0</td>
<td>1.73</td>
<td>2.05</td>
</tr>
<tr>
<td>4.8</td>
<td>1.19</td>
<td>1.0</td>
<td>1.71</td>
<td>2.05</td>
</tr>
<tr>
<td>6.0</td>
<td>1.26</td>
<td>1.0</td>
<td>1.70</td>
<td>2.05</td>
</tr>
</tbody>
</table>

TRL = TRR x K400 x K100 x L
K400 = 1 + 0.09F - 0.0015F^2
F = \( \frac{V}{0.6} + \frac{V}{L} \)
K100 = \( \left( \frac{L}{2.4} \right)^{0.75} \) (L < 2.4m)
= \( \left( \frac{L}{2.4} \right)^{0.25} \) (L > 2.4m)
<table>
<thead>
<tr>
<th>Length</th>
<th>TRL/K110</th>
<th>Number of Tests</th>
<th>TRLpw x K110sp (a)</th>
<th>Log a</th>
<th>Log ( \frac{L}{2.4} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>0.28</td>
<td>0.33</td>
<td>0.305</td>
<td>16</td>
<td>0.062</td>
</tr>
<tr>
<td>1.2</td>
<td>1.08</td>
<td>1.10</td>
<td>1.09</td>
<td>28</td>
<td>0.221</td>
</tr>
<tr>
<td>1.8</td>
<td>3.19</td>
<td>2.88</td>
<td>3.035</td>
<td>4</td>
<td>0.706</td>
</tr>
<tr>
<td>2.4</td>
<td>4.94</td>
<td>4.93</td>
<td>4.935</td>
<td>28</td>
<td>1.000</td>
</tr>
<tr>
<td>3.0</td>
<td>7.80</td>
<td>7.87</td>
<td>7.835</td>
<td>4</td>
<td>1.588</td>
</tr>
<tr>
<td>3.6</td>
<td>9.48</td>
<td>8.76</td>
<td>9.12</td>
<td>14</td>
<td>1.848</td>
</tr>
<tr>
<td>4.8</td>
<td>12.85</td>
<td>12.84</td>
<td>12.845</td>
<td>32</td>
<td>2.603</td>
</tr>
</tbody>
</table>

Note

\[
K_{110} = 1 + ((0.09F - 0.0015F^2) (2.4)^{0.4}), \quad F = \frac{V}{L} + \frac{V}{0.6L}
\]

Table 6.33 Design Data for the Analysis of K111
Figure 6.21 Power Solution for $K_{111}$ Factor for Wall Length ($KL$)

Note: Number of tests shown in brackets
<table>
<thead>
<tr>
<th>L (m)</th>
<th>TRR (kN/m)</th>
<th>K11</th>
<th>K110</th>
<th>TRL (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Overt</td>
<td>5kN/stud</td>
</tr>
<tr>
<td>0.6</td>
<td>2.05</td>
<td>0.25</td>
<td>1.0</td>
<td>2.88</td>
</tr>
<tr>
<td>1.2</td>
<td>2.05</td>
<td>0.50</td>
<td>1.0</td>
<td>2.17</td>
</tr>
<tr>
<td>1.8</td>
<td>2.05</td>
<td>0.75</td>
<td>1.0</td>
<td>1.92</td>
</tr>
<tr>
<td>2.4</td>
<td>2.05</td>
<td>1.00</td>
<td>1.0</td>
<td>1.77</td>
</tr>
<tr>
<td>3.0</td>
<td>2.05</td>
<td>1.09</td>
<td>1.0</td>
<td>1.69</td>
</tr>
<tr>
<td>3.6</td>
<td>2.05</td>
<td>1.17</td>
<td>1.0</td>
<td>1.62</td>
</tr>
<tr>
<td>4.8</td>
<td>2.05</td>
<td>1.30</td>
<td>1.0</td>
<td>1.54</td>
</tr>
</tbody>
</table>

\[
TRL = TRR \times K110 \times K111 \times L
\]

\[
K110 = 1 + (0.09F - 0.0015F^2) \left(\frac{2.4}{L}\right)^0.4
\]

\[
F = \frac{V}{0.6} + \frac{V}{L}
\]

\[
K111 = \left(\frac{L}{2.4}\right)^0.38 (L > 2.4m)
\]

\[
K111 = \left(\frac{L}{2.4}\right) (L < 2.4m)
\]

Table 6.34 Plain Panel Test Loads Using the Power Solution Approach for the K111 Factor

<table>
<thead>
<tr>
<th>L (m)</th>
<th>Eff L (m)</th>
<th>KL</th>
<th>TRR (kN/m)</th>
<th>K110</th>
<th>TRL (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>-</td>
<td>1.6</td>
<td>2.05</td>
<td>1.0</td>
<td>2.88</td>
</tr>
<tr>
<td>1.2</td>
<td>0.3</td>
<td>1.6</td>
<td>2.05</td>
<td>1.0</td>
<td>2.17</td>
</tr>
<tr>
<td>1.8</td>
<td>0.9</td>
<td>1.6</td>
<td>2.05</td>
<td>1.0</td>
<td>1.92</td>
</tr>
<tr>
<td>2.4</td>
<td>1.5</td>
<td>1.6</td>
<td>2.05</td>
<td>1.0</td>
<td>1.77</td>
</tr>
<tr>
<td>3.0</td>
<td>2.1</td>
<td>1.6</td>
<td>2.05</td>
<td>1.0</td>
<td>1.69</td>
</tr>
<tr>
<td>3.6</td>
<td>2.7</td>
<td>1.6</td>
<td>2.05</td>
<td>1.0</td>
<td>1.62</td>
</tr>
<tr>
<td>4.8</td>
<td>3.9</td>
<td>1.6</td>
<td>2.05</td>
<td>1.0</td>
<td>1.54</td>
</tr>
</tbody>
</table>

\[
TRL = TRR \times K110 \times K111 \times L
\]

\[
KL = \frac{V}{0.6} + \frac{V}{L}
\]

\[
K110 = 1 + (0.09F - 0.0015F^2) \left(\frac{2.4}{L}\right)^0.4
\]

Table 6.35 Plain Panel Test Loads Using the Linear Solution Approach for the K111 Factor

-325-
Best Fit Solution

\[ Y = 0.67x - 0.58 \]

Gives \[ K_{111} = 1.6 - \frac{1.39}{L} \]

Figure 6.22 Linear Solution for \( K_{111} \) for Walls Longer than 0.6m
Intersection by Calculation
Gives \( L = 1.44 \)

For Short Walls
\[ K_{111} = \frac{L}{2.4} \]

For Longer Walls
\[ K_{111} = 1.6 + \frac{1.44}{L} \]

Key
- \( \times \) Test Data
- + Power Curve Solution
- ○ Linear Solution

Figure 6.23 The Interaction of the Linear and Power Curve Solutions for \( K_{111} \) for Short Walls
Figure 6.24 The Effect of the \( K_{300} \) Modification Factor
<table>
<thead>
<tr>
<th>Test Programme</th>
<th>Length</th>
<th>Vertical Load</th>
<th>Design Load</th>
<th>Failure Load</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>1977</td>
<td>4.8</td>
<td>0</td>
<td>12.79</td>
<td>33.98</td>
<td>2.66</td>
</tr>
<tr>
<td>1985</td>
<td>4.8</td>
<td>0</td>
<td>19.70</td>
<td>47.34</td>
<td>2.40</td>
</tr>
<tr>
<td>1977</td>
<td>4.8</td>
<td>5</td>
<td>13.99</td>
<td>33.70</td>
<td>2.41</td>
</tr>
<tr>
<td>1985</td>
<td>4.8</td>
<td>5</td>
<td>19.70</td>
<td>40.79\textsuperscript{1}</td>
<td>2.07\textsuperscript{1}</td>
</tr>
<tr>
<td>1985</td>
<td>3.6</td>
<td>0</td>
<td>8.63</td>
<td>21.41\textsuperscript{1}</td>
<td>2.48\textsuperscript{1}</td>
</tr>
<tr>
<td>1977</td>
<td>2.4</td>
<td>0</td>
<td>4.92</td>
<td>12.46</td>
<td>2.53</td>
</tr>
<tr>
<td>1977</td>
<td>2.4</td>
<td>5</td>
<td>8.71</td>
<td>21.40</td>
<td>2.46</td>
</tr>
</tbody>
</table>

Notes

1. Not the maximum value because test stopped after wall had reached two times its stiffness load

2. Maximum value but same wall had been tested close to failure at a lower vertical load

Table 6.36 Factors of Safety for Plain Wall Tests
<table>
<thead>
<tr>
<th>Panel Type</th>
<th>Vertical Load (KN/stud)</th>
<th>Average 5mm Deflection of 2.4m Panel (KN)</th>
<th>Average 5mm Deflection of 4.8m Panel (KN)</th>
<th>Percentage Improvement</th>
<th>Percentage Improvement of Design Load for Same Change in Length Proposed</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.5mm standard</td>
<td>0</td>
<td>3.01</td>
<td>8.90</td>
<td>296%</td>
<td></td>
</tr>
<tr>
<td>plasterboard</td>
<td>2½</td>
<td>4.47</td>
<td>11.20</td>
<td>251%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>5.56</td>
<td>12.48</td>
<td>224%</td>
<td></td>
</tr>
<tr>
<td>12.5mm MR plasterboard</td>
<td>0</td>
<td>3.25</td>
<td>6.95</td>
<td>214%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2½</td>
<td>4.21</td>
<td>10.14</td>
<td>241%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>4.70</td>
<td>11.11</td>
<td>227%</td>
<td></td>
</tr>
<tr>
<td>9.5mm MR plasterboard</td>
<td>0</td>
<td>2.99</td>
<td>7.84</td>
<td>262%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2½</td>
<td>4.29</td>
<td>10.96</td>
<td>255%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>5.13</td>
<td>11.70</td>
<td>228%</td>
<td></td>
</tr>
<tr>
<td>Average plasterboard</td>
<td>0</td>
<td>2.57</td>
<td>7.67</td>
<td>257%</td>
<td>260%</td>
</tr>
<tr>
<td></td>
<td>2½</td>
<td>4.01</td>
<td>10.41</td>
<td>249%</td>
<td>236%</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>5.06</td>
<td>11.41</td>
<td>226%</td>
<td>226%</td>
</tr>
<tr>
<td>12.5mm MR plasterboard plus 12.0mm MDF</td>
<td>0</td>
<td>5.77</td>
<td>11.36</td>
<td>197%</td>
<td>260%</td>
</tr>
<tr>
<td></td>
<td>2½</td>
<td>8.36</td>
<td>19.46</td>
<td>233%</td>
<td>236%</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>10.45</td>
<td>24.14</td>
<td>231%</td>
<td>226%</td>
</tr>
</tbody>
</table>

Table 6.37 4.8m Long Plasterboard Wall Tests
Lintel areas can be ignored

(a) Walls broken into rectangles regardless of panel and board joints. Here the design will be inefficient as panels under windows provide continuity.

(b) A wall with full height discontinuities which will be well suited to the design method.

(c) The slenderness of the lintol will determined the amount of discontinuity caused by the door height opening.

Figure 6.26 Design Based on the Summation of Sheathing Rectangles
Vert Load F KN/m defines $K_{210}$

Standard Panel Equations

- $R_{L1,\text{min}} = 1.6 \times BRR \times (L_1 - 0.9) \times K_{210}$
- $R_{L1,\text{max}} = 1.6 \times BRR \times L_1 \times K_{210}$

Short Panel Equations

- $R_{L2,\text{min}} = 1.6 \times BRR \times (0.125 \times L_2) \times K_{210}$
- $R_{L2,\text{max}} = 1.6 \times BRR \times L_2 \times K_{210}$

Figure 6.27 Module Method for Openings: Plain Panel Design Values
Plain Panel

$$RL = 1.6 \, BRR \times K_{210} \times (L - 0.9 \left[ \frac{h_o}{h} p \right]$$

Door Panel

$$RL = 0$$

Window Panel

$$RL = 1.6 \, BRR \times K_{210} \times \frac{h_p}{h} \left[ (L - 0.9 \left[ \frac{h_o}{h} p \right] \right]$$

where $$\left[ \frac{h_o}{h} p \right]$$ is the opening ratio of the preceding panel.

Figure 6.28 Module Method For Openings: Panel Design Values Using Standard Equation
**WRL** = \( \sum_{A}^{F} \text{RL} \)

**RL_A** = minimum load for panel of length \( L_1 \)
\[ = 1.6 \times BRR \times K_{210} \times (L_1 - 0.9) \]

**RL_B** = maximum load for panel of length \( L_1 \)
\[ = 1.6 \times BRR \times K_{210} \times L_1 \]

**RL_C** = maximum load for window panel of length \( L_1 \)
\[ = 1.6 \times BRR \times K_{210} \times \frac{h_{01}}{h} \times L_1 \]

**RL_D** = part load for panel of length \( L_1 \)
\[ = 1.6 \times BRR \times K_{210} \times (L_1 - 0.9) \times \frac{h_{01}}{h} \]

**RL_E** = 0, no load for door openings

**RL_F** = part load for short panel of length \( L_2 \)
\[ = 1.6 \times BRR \times K_{210} \times \left( \frac{h_{02}}{h} \times 0.875 \times L_3 \right) \]

**Figure 6.29 Module Method for Openings:**
Typical Wall Design Method

<table>
<thead>
<tr>
<th>Preceding Panel</th>
<th>( \frac{h_{0}}{h} ) P</th>
<th>1.2m Plain Panel Load (kN)</th>
<th>0.6m Plain Panel Racking Load (kN)</th>
<th>1.2m Window Panel Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain Panel</td>
<td>0.00</td>
<td>3.19</td>
<td>1.56 (0.60) (i)</td>
<td>1.33</td>
</tr>
<tr>
<td>Window Panel</td>
<td>0.49</td>
<td>2.02</td>
<td>0.91 (0.40) (i)</td>
<td>0.84</td>
</tr>
<tr>
<td>Door Panel</td>
<td>0.90</td>
<td>1.04</td>
<td>0.34 (0.24) (i)</td>
<td>0.43</td>
</tr>
<tr>
<td>No Panel</td>
<td>1.00</td>
<td>0.80</td>
<td>0.20 (0.20) (i)</td>
<td>0.33</td>
</tr>
</tbody>
</table>

**Notes:**
(i) Alternative method for short panels, see text for details
(ii) BRR for panels = 1.77kN/m

**Table 6.38 Individual Panel Design Values At Zero Vertical Load For The 1979 Test Programme**

-335-
<table>
<thead>
<tr>
<th>Panel Combination</th>
<th>Zero Vertical Load Design Values (kN)</th>
<th>5kN/Stud Vertical Load Design Values (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
<td>Module Design Method</td>
</tr>
<tr>
<td>PPPP (4)</td>
<td>11.65</td>
<td>11.41</td>
</tr>
<tr>
<td>HPPP (4)</td>
<td>12.07</td>
<td>12.46</td>
</tr>
<tr>
<td>HDPH</td>
<td>6.37</td>
<td>6.59</td>
</tr>
<tr>
<td>HDPDH</td>
<td>3.21</td>
<td>1.74</td>
</tr>
<tr>
<td>CPPPH</td>
<td>5.54</td>
<td>6.59</td>
</tr>
<tr>
<td>CPPDPH</td>
<td>5.08</td>
<td>5.53</td>
</tr>
<tr>
<td>CPPDP</td>
<td>2.99</td>
<td>2.02</td>
</tr>
<tr>
<td>PPPD</td>
<td>7.53</td>
<td>5.53</td>
</tr>
<tr>
<td>HWWWP</td>
<td>3.85</td>
<td>3.15</td>
</tr>
<tr>
<td>HWWPH</td>
<td>7.06</td>
<td>6.55</td>
</tr>
<tr>
<td>HPWHPH</td>
<td>6.87</td>
<td>7.12</td>
</tr>
<tr>
<td>HWPPPH</td>
<td>8.88</td>
<td>9.13</td>
</tr>
<tr>
<td>HWPWH</td>
<td>7.56</td>
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<tr>
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<td>4.70</td>
<td>3.49</td>
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Note: Bracketed values refer to alternative method for dealing with short panels.

Table 6.39 Comparison of Test Results With Design Models for the 1979 Test Programme
Zero Vertical Load Results Only

**RL**

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**Alternative Proposal**

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**Original Proposal**

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**0.6m Walls**

**RL**

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**Alt**

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**Orig**

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**1.2m Walls**

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**Orig**

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<tr>
<td>5.50</td>
<td>0.22</td>
<td>3.59</td>
<td>1.69</td>
</tr>
</tbody>
</table>

**2.4m Walls**

NOTES

- Wall panels not to scale
- Racking loads (RL) are conservative when underlined and unsafe when boxed

Figure 6.30 Module Method For Openings: Comparison of Predictions for Alternative Approaches to Short Panel Design
Figure 6.31 The Effect on Design of Proposals For an Openings' Modification Factor
Example A

Small openings may be ignored if \( c < 250 \)

Example B.

Figure 6.32 Examples of Code Guidance in Designing Walls with Openings
Table 6.40 Geometric Properties and Test Results
For Use in Determining the Wall Opening Modification Factor
(a) $K_0$ based on full height length.

$$K_0 = \frac{f(\text{full height length})}{\text{total length}} = f\left(\frac{\ell_1 + \ell_3 + \ell_5}{L}\right).$$

Note: this is identical to using the length lost due to openings when:

$$K_0 = f\left(1 - \frac{\ell_2 + \ell_4}{L}\right).$$

Let $q = \frac{\ell_1 + \ell_3 + \ell_5}{L}$

Then $K_0 = f(q)$

(b) $K_0$ based on sheathed area.

$$K_0 = f\left(\frac{\text{area of sheathed wall}}{\text{total area of wall}}\right) = f\left(1 - \frac{A_1 + A_2}{h \times L}\right)$$

Let $p = \frac{\text{area of openings}}{\text{total wall area}} = \frac{A_1 + A_2}{h \times L}$

Then $K_0 = f(1-p)$

(c) $K_0$ based on area moment of sheathed area about the leeward edge

$$K_0 = f\left(\frac{\text{area moment of sheathed wall}}{\text{total area moment of wall}}\right)$$

$$= f\left(1 - \frac{A_2 (\ell_5 + \ell_4)/2 + A_1 (\ell_5 + \ell_4 + \ell_3 + \ell_2)/2}{h \times L \times L/2}\right)$$

Let $m = \frac{\text{area moment of openings}}{\text{total area moment of wall}}$

Then $K_0 = f(1 - m)$

Figure 6.33 Calculations For The Predicted Value of The Wall Opening Modification Factor
From test results and plain panel design

Figure 6.34 The Interaction Diagram Used to Determine the Opening Modification Factor
Figure 6.35 Test Result Interaction Using $K_{112} = (1 - 1.3p)^2$. 

Key
1979 tests
- 4.8m walls
- 3.6m walls
- 2.4m walls
- 1.2m walls

1985 tests
- 4.8m walls
- 3.6m walls
- 2.4m walls
- 1.2m walls

a = Mean data line (correlation coefficient = 0.966)
b = 10% exclusion line
c = 5% exclusion line
d = $p = \frac{\text{Area of opening}}{\text{Total wall area}}$
The proportion of opening 'p' is given by:

(i) \[ p = \frac{A_W + A_D}{Lh} \]
in the principal design method

(ii) \[ p = \frac{A_W + A_D + A_S}{Lh} \]
in the alternative approach that is correct for the Code interpretation of area.

Figure 6.36 Calculation of Area of Opening
Figure 6.37 The Weakness of Walls with Long Window Openings
Figure 6.38 Test Results Interaction Using $k_{212} = q^2$

Key

1979 tests
- 4.8m walls
1985 tests
  - 4.8m walls
  - 3.6m walls
  + 2.4m walls
  X 1.2m walls

a = Mean data line (correlation coefficient = 0.924)
b = 10% exclusion line
c = 5% exclusion line
q = $\frac{\text{Length of full height wall}}{\text{Total length of wall}}$

-346-
(a) Framing around window opening

(b1) 'C' shaped sheathings  (b2) Three sheathing boards
(b) Layout of sheathing and nail lines

NB Computer design for 'L' shaped sheathing similar to 'C' shaped but omitting board over lintol

Figure 6.39 Test Panels for Sheathing Around Openings
<table>
<thead>
<tr>
<th>Proportion of Window Opening $P$</th>
<th>Modification Factor</th>
<th>Draft (i)</th>
<th>Proposed (ii)</th>
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</tr>
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<tr>
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<td></td>
</tr>
<tr>
<td>0.75</td>
<td>-</td>
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</table>

(i) $K_{300} = (1 - p)^2$

(ii) $K_{112} = (1 - 1.3p)^2$

\[ p = \frac{\text{Area of openings in wall}}{\text{Total area of the wall}} \]

Table 6.41 Design Values for the Window Opening Modification Factor

![Diagram](image)

Actual Opening = \( h_w (d_1 + e + d_2) \)

Effective Opening

if \( c_1 \) and \( c_2 \geq 300 \) = \( h_w (0.75 d_1 + e + 0.75d_2) \)

if \( c_1 \) and \( c_2 \geq 600 \) = \( h_w (0.5 d_1 + e + 0.5d_2) \)

Figure 6.40 Effective Window Openings
(a) Standard layout  
(b) Continuous corner

Layout (b) is an example of a continuous corner sheathing arrangement which is unlikely to show the full enhancement expected when compared with layout (a).

Figure 6.41 A Weakness in the Continuous Corner Sheathing Arrangement
<table>
<thead>
<tr>
<th>Modification Factor</th>
<th>Value of Factor or Change from Generating Equation</th>
<th>Change from Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_{110}$ Vertical Load</td>
<td>$1 + (0.09F - 0.0015F^2)(\frac{2.4}{L})^{0.4}$</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>$F = \frac{V}{\text{stud spacing}} + \frac{V}{L}$</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>$F = \frac{2aF_p}{L^2}$</td>
<td></td>
</tr>
<tr>
<td>$K_{111}$ Length</td>
<td>$\frac{L}{2.4}$ if $L &lt; 1.45$ m</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>$(1.6 - \frac{1.44}{L})$ if $1.45 &lt; L &lt; 4.8$ m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$1.30$ if $L &gt; 4.8$ m</td>
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</tr>
<tr>
<td>$K_{112}$ Openings</td>
<td>$(1 - 1.3p)^2$</td>
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<tr>
<td>$K_{113}$ Height</td>
<td>$\frac{2.4}{h}$</td>
<td>NO</td>
</tr>
<tr>
<td>$K_{210}$ Vertical Load Simplified Design</td>
<td>$1 + 0.07F - 0.0011F^2$</td>
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<tr>
<td>$K_{211}$ Length Simplified Design</td>
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<td></td>
<td>$1.30$ if $L &gt; 4.8$ m</td>
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<tr>
<td>$K_{212}$ Openings Simplified Design</td>
<td>$q^2$</td>
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</table>

Notes:  
$F$ = UDL load (kN/m)  
$F_p$ = equivalent UDL load (kN/m)  
$V$ = stud load (kN/stud)  
$L$ = wall length (m)  
$p$ = area of openings in wall  
(total area of wall)  
$h$ = wall height (m)  
$q$ = total length of full height panel  
(length of wall)  

Table 6.42 Summary of Modification Factors for Use in Wall Design
Equivalent UDL = \frac{2[(F_{p1} \times a_1) + (F_{p2} \times a_1)]}{L^2}

from which \( K_{110} \) can be calculated

Equivalent UDL = \frac{2[(F_{p1} \times L_1 \times a_1) + (F_{p2} \times L_2 \times a_2)]}{L^2}

Figure 6.42 Variations in Vertical Load Along Wall Length
SECTION C

FACTORS AFFECTING THE USE OF THE WALL

6.7 EXTERNAL PANEL FASTENINGS

6.7.1 Introduction

The design proposals made so far in this Chapter relate to panels using specialist fastenings to simplify and standardise the test work. It is essential that these results be related to the practical use of timber frame walls on site. The important internal variations in a panel have been covered by the modification factors affecting basic racking resistance. However the external fixings to the wall are not included in this way and yet they will have a significant effect on the performance of the wall unit. The fixings that must therefore be considered are:

(i) joints between panels,

(ii) base fixing of panels,

(iii) the effect of the return wall.

Limited series of tests have been undertaken to enable proposals to be made covering standard practical cases. It is not possible to investigate all the specialist connections used by different manufacturers, and thus it is important that proposals for modification factors should be based on standard structural practice so that any specialist fittings can be independently assessed and, if necessary, checked by test.

Two approaches to the determination of modification factors are indicated which will relate to the significance of the fixing to the overall strength of the structure. In the first case a minimum standard is noted, below which
the fixing may become critical to design. Improvement on the standard will not have a significant effect on wall behaviour. Here a minimum quality of fixing is set which will allow the standard design values to be used. This fixing may be either independent of racking load, as in the case of the vertical panel joint, or directly dependent, as in the case of shear along the base. No enhancement in performance is allowed for improvement to the minimum acceptable standard but a reduction in fixing requires a stringent modification factor to be included to reduce the overall capacity of the wall. In the second case the fixing is an enhancement to the standard panel case and a modification factor is required to cover its use, the factor should always be greater than unity. This will cover the return wall effect and base fixings which resist uplift of the panel.

Two further factors that should be considered in the use of timber frame walls are external attachments which can improve performance, such as brick walls, and acceptable levels of damage that may have to be sustained by the wall, particularly during construction, and which may require the inclusion of a reduction factor.

Throughout this section the factors are mainly of a secondary nature in the calculation of wall performance and few tests have been carried out, thus they are kept simple in concept and may err heavily on the side of safety.

6.7.2 Joints Between Panels

The effect of joints between panels was examined by testing pairs of 1.2m long panels in combination and varying the fixings linking the abutting studs. It was not practical to test the different joints on separate pairs of panels nor would this method necessarily be accurate due to the variability in performance of similar panels. Instead, all variations were tested on the same pair of panels using double headed nails and bolts which could easily be removed and reused.
Two pairs of panels were used in the tests both sheathed with 9.5mm Canadian CSP plywood, fixed on Redwood/Whitewood frames using 63mm long 3.25mm diameter gun driven wire nails at standard centres. Panels were bolted through the sole plate to the test rig in the normal manner and were linked by a top plate continuous over both panels and fixed to them with coach screws or bolts, at 600mm intervals, centrally between studs. These fittings are representative of standard conditions, although in practice the joint would be nailed, and they contribute greatly to the panel joint. The nails used to join the studs were 75mm long 3.75mm diameter, evenly spaced along the stud, and the bolts were the M12 black bolts used in all the previous tests. The vertical joints tested were, in chronological order:

(i) no nails,
(ii) 4 nails,
(iii) 8 nails,
(iv) 16 nails,
(v) 4 nails,
(vi) 4 nails and 3 bolts.

For both pairs of panels three deflection limits were used, 5, 10 and 15mm. In the first two cases, tests were conducted on each joint at both zero and 5 kN/stud vertical load. In the final case only the 5 kN/stud load case was used. The procedure in each set of six tests on a pair of panels was to carry out a single load cycle for each type of joint without resetting the deflection gauges between cycles. A typical set of data is shown in Figure 6.43.

No design loads were required from the tests which were used solely to compare the performance of different fixings. Comparison was difficult due to the set in the wall between tests and greater accuracy was established using the two identical tests on the 4 nail configuration. The most accurate comparisons were obtained when the loads required to deflect the panels 5mm in the 10 and 15mm
deflection tests were measured, taking the start of each cycle to be a separate datum. Results were adjusted, based on the change in value between the two 4 nail tests, and then the percentage differences were calculated. The results, shown in Table 6.43, vary for the six sets of tests applicable to the analysis but there is no trend linking the variation to a specific type of test. The results have therefore been averaged before being used to predict the effect of different jointing methods.

If the effect of the nails is considered additive, then, compared with using three M12 bolts, twelve 3.75mm nails will reduce performance by 3.5% and four nails will cause a 7% reduction. Typically eight nails are recommended for practical use in such a joint when their spacing will be 300mm.

It is proposed that the design factors quoted in Sections A and B are applicable to walls made up of panels joined vertically with either:

(1) 3.75mm diameter nails at 300mm centres along the full stud length, or

(11) 3 bolts of at least 10mm diameter at intervals of not less than 1.0m.

Additionally panels should be linked by top and bottom plates, or equivalents, overlapping the join by at least 600mm and fixed with 3.75mm diameter nails also at 300mm centres.

This requirement may be considered inadequate in terms of the test results but it must be noted that in all domestic buildings a wall contributing to racking resistance will include an internal lining. This lining will, either by position of the board over the joint or by the taping of its joint, provide extra continuity between the panels.
regardless of the linings contribution to racking resistance.

The small differences experienced in the test results means that there should be little risk in extending the rules to cover all wall lengths. Furthermore the difference between four and eight nails is not significant and so the design method will remain safe if some of the vertical nails are accidentally omitted. The guideline is a practical one and should already be accepted as a standard requirement. It is for these reasons that no reduction factor is necessary to deal with weaker jointing methods.

The tests indicate, from the no nail performance values, that the continuity of the top and bottom plates is essential to the effectiveness of the joint. It is therefore important that this provision is closely examined when checking the wall structure both on site and through the calculations. If a designer does not use a top plate it will be necessary for him to prove adequate continuity between panels by other means.

6.7.3 Base Fixings

Base fixings are normally required to transfer shear loads from the wall into the foundations. They may then be treated separately from the panel design and the joint capacity analysed using standard timber engineering practice based on BS 5268 part 2 (BSI, 1984).

If the wall is not vertically loaded it will be necessary for the base fixings to provide some resistance to overturning to allow the full racking design load to be resisted in the wall. Joint types such as vertical nailing do not always achieve an adequate resistance to overturning and will require to be designed independently, based on the results of racking tests. Tie down straps linking the panel studs direct to the foundations are often used to reduce panel rotation. In theory they do not increase
the shear resistance of the base joint, however, by reducing overturning they improve the performance of vertically nailed joints and they also improve overall wall performance, when the force motivated in the straps due to overturning forces acts as an equivalent vertical load on the panel. In the latter case this then increases the requirement for the horizontal joint.

Information on base fixings has been taken from two separate test programmes carried out at Surrey. The first formed part of the current investigation but the experimental work was performed in a separate investigation supervised by the author and has been fully detailed (Dillon, 1980). The tests were conducted on 2.4 and 4.8m panels clad with 9.0mm mediumboard fixed to hem-fir frames with 50mm long 3.25mm diameter clout head nails at standard centres. The materials were similar to those used in the 1979 wall tests reported in Section 6.5. The following situations were covered.

(i) Variations in bolting down the bottom rail, increasing the spacing of the bolts from the standard 600mm (approximate) spacing used in all the major test programmes.

(ii) Vertical nailing with 75mm long 3.75mm diameter nails at 300mm or 150mm or 75mm spacing.

(iii) The use of holding down straps with both the above mentioned base fixings. Details of the straps are shown in Figure 5.9, they were placed at either 600mm centres over studs or 1200mm centres at every board junction.

The second programme of tests was undertaken as part of a research contract for the Department of the
Environment (see Appendix B). The tests were carried out on 2.4m panels clad with 9.5mm douglas fir plywood fixed to SPF frames with 63mm long 3.25mm diameter gunned wire nails at standard centres. The materials were similar to test P8 in Section 6.2. Standard panel tests were carried out, replacing the normal bolted base connections with the following joints.

(1) 75mm long 3.75mm diameter nails at 200mm centres driven vertically through the bottom rail of the panel into the base plate which had been bolted to the test rig.

(ii) Similar nails at 100mm centres.

(iii) Holding down straps at 1.2m centres at board edges used in conjunction with standard holding down bolts. The straps were similar to those shown in Figure 5.9.

(iv) Horizontally driven 63mm long, 3.25mm diameter cover nails fixing the overlapping edge of the plywood sheathing directly to the base plate at 150mm centres.

All the test results in both programmes have been adequately documented and are not detailed here. Instead, the requirements for the design of base joints are considered and compared with the test work. At present the specification for base fixings is entirely independent of the shear wall in both North America (Anderson, 1975 and CMHC, 1979) and Britain (TRADA, 1980 a). The shear resistances achieved by these specifications based on short term loads and SC3 quality timber is as follows:

(1) American practice; either
4.1mm nails 80mm long at 400mm centres = 1.27 kN/m
or
12.0mm bolts at 2400mm (maximum) centres = 1.15 kN/m
(ii) British practice;
3.75mm nails 75mm long at 300mm centres = 1.48 kN/m

The shear resistances are very low in comparison with the 1.68 kN/m datum racking resistance for plywood when it is considered that all the wall modification factors, except that for openings, will enhance this value. The problem will be compounded when fixings are omitted at door openings.

Clearly this method of specifying base fixings is wrong. It is essential that such fixings be designed to resist the racking load carried in the wall when, for complete safety, the resistance should be the maximum capacity of the wall. The design of these fixings is relatively simple but the widespread use of the standard specification undermines its importance and makes its introduction in design and checking procedures difficult. The spacing of fixings can be determined for the maximum racking capacity of the wall using the formula:

\[
\text{Spacing} = \frac{\text{Load per fixing} \times L}{\text{BRR} \times L \times K_{110} \times K_{111} \times K_{112}}
\] - 6.60

if the less common modification factors for height etc. are omitted. If the applied wall load is known the spacing may be increased and calculated using the formula:

\[
\text{Spacing} = \frac{\text{Load per fixing} \times L}{\text{Racking Load Applied}}
\] - 6.61

In both cases the fixing load is calculated for a short term duration.

To give an example; if a 4.8m plywood sheathed wall carries a 10.4 kN/m vertical load and is required to perform to its full capacity the nails spacing using 3.75mm diameter nails in SC3 timber will be:
Spacing = \((0.355 \times 1.25) \times 4.8\)
\[
1.68 \times 4.8 \times 1.58 \times 1.30 \times 1.0
\]
= 0.129m, say 125mm centres.

If the wall included 20% openings the spacing could be increased to 225mm. It is clear that base fixing design is of greatest importance in plain walls and thus in load bearing separating and internal partition walls which previously may have been given minimal consideration by the erector.

Equations 6.60 and 6.61, above, may be applied to both holding down bolts and vertical nailing, although in the latter case there is a secondary problem in that the resistance of the wall to overturning may be reduced as the nails are acting in withdrawal. This mainly affects short panels (because racking load is approximately proportional to \(L^{1.4}\) for long panels whereas the overturning resistance of a fixing is proportional to \(L^2\)) and at low vertical loads (because vertical load provides a restoring moment). The effect of overturning will affect both panel stiffness and strength and is demonstrated in the results of tests on 2.4 and 4.8m panels, although only stiffness results are available in the latter case. Table 6.44 examines the 2.4m panel results and compares the stiffness and strength design values for the vertical nail fixings with the following:

(i) the basic racking resistances of the materials used in the panel calculated from standard tests using holding down bolts,

(ii) the basic racking resistances of the materials as calculated using the proposed design values,
(iii) the shear resistance of the base fixings in SC3 timber.

Table 6.45 covers the 4.8m panel results. In both cases the design value for the wall should be the lower of the wall racking load and the base nail shear load.

The 4.8m panel results shows the tested stiffness values to exceed the design requirements. In the region where panel failure could be a problem, i.e. at zero vertical load with weak base fixings, the factor of safety achieved by the stiffness result (11.3/7.10) is so high that strength is unlikely to be a critical factor. Thus panel design will initially be governed by base shear when nail spacing is wide, but as the spacing decreases the racking resistance will become critical, first at zero vertical load and then at progressively higher vertical loads. The very high stiffness values recorded in the 5 kN/stud vertical load tests are unlikely to be representative of the overall panel performance because it has been seen that the tested safety factor reduces with vertical load and panel length. For this reason no improvement in racking performance should be allowed in cases where the base fixing exceeds that required by the standard test results.

The 2.4m panel results clearly indicate that panel strength in short panels is affected to a much greater degree, in comparison with panel stiffness, by the inadequate resistance to overturning. The panel performance has been so greatly reduced that in many cases the test value for design does not attain the calculated value. In practical design this could lead to unsafe conditions because short panels, although less common, are more likely to contain a lower percentage of openings. In practice the building structure will include secondary features that will resist the overturning, such as return walls, but to maintain a consistent standard of design safety it is
necessary to either reduce the likelihood of overturning or reduce the allowable design load. Three alternatives are considered.

(i) To require walls of, say, 3.0m in length or less, which are vertically nailed to the base plate but are not vertically loaded, to be additionally fixed with holding down straps at either end. Assuming the straps to tie the studs to the foundations using the equivalent of six 3.25mm nails sufficient vertical load will be motivated to enable the full shear resistance of the base fixing to be attained.

(ii) To require the same walls noted in (i) above to be fixed with nails at very close centres so that racking resistance will govern design.

(iii) To reduce the capacity of such walls by the inclusion of a short panel modification factor \( K_{116} \) in the base shear design such that:

\[
K_{116} = \frac{K_{VL}}{2} \quad (L < 3.0m) \quad - 6.62
\]

where \( K_{VL} \) is the appropriate vertical load modification factor and:

\[
K_{116} = 1.0 \quad (L \geq 3.0m) \quad - 6.63
\]

The third alternative is at present adopted because it allows a unified approach to holding down straps to be maintained, it avoids an increase in secondary rules regarding fixings and it can easily be incorporated into a
computer based design program. The results of using the modification factor are noted in Table 6.44. It slightly overestimates the reduced panel performance at zero vertical load, but this can be accepted in view of the secondary restraints available in practice, and is increasingly conservative at higher vertical loads and as the quality of the base fixing reduces. This reduction in efficiency, however, is insignificant in comparison with the standard nail shear design case; it is probable that the nail shear underestimates test performance because no account is taken of friction motivated by the vertical load.

The bolted base fixings used throughout the test work do not present the same problems as vertical nailing for two reasons. Firstly if the initial bolt is close to the leading edge of the wall and a large washer is used, rotation of the bottom rail of the wall will be insignificant. Secondly the short term shear resistance of 12mm bolts at 600mm centres (4.6 kN/m) is very high in comparison with the applied load and thus bolts can be removed from the base fixing without significant reduction in performance. Tests (Dillon, 1980) indicated small losses which could be attributed to the reduction in stiffness of base resistance. It was noted that the loss in performance increased with panel length. This was predictable because the base fixing resistance is independent of length whereas the equivalent racking resistance has been shown to increase with length. (equation 6.36)

Thus for design purposes holding down bolts may be calculated using the guidelines given in BS 5268 part 2 (BSI, 1984) without the need for any additional clause except for the size of washer.

Holding down straps fixing the studs of the panels direct to the foundations were noted in both test programmes to improve panel racking resistance particularly at zero vertical load. The improvement was seen to decrease with panel length, vertical load and the quality of the main base
fixings. A safe method for calculating the effect of the straps, which is consistent with the test results, is to consider their resistance to uplift as an equivalent vertical load. This will normally be calculated from the shear resistance of the nails attaching the strap to the stud although it will also be necessary to check its tensile capacity and fixing to the foundations. As with applied vertical load the resistance to uplift is only motivated in regions where the studs are lifted due to the rotational action of the racking force. In both cases the vertical load is calculated from the sum of the individual values when related to their location along the wall. If the holding down straps are identical and set at regular intervals along the wall, starting at the leading edge, an equation similar to 6.12 may be used to calculate the equivalent vertical load whereby

\[ F = \frac{V}{ \text{strap spacing}} + \frac{V}{L} \]  

- 6.64

otherwise equation 6.59 should be applied to individual straps.

Two examples of 'holding down strap' design are shown in Figure 6.44 and are related to test results. In the 2.4m panel tests sufficient data were collected to compare design loads which, throughout, were governed by stiffness. At zero vertical load the test racking resistance is much higher than that predicted and represents a vertical load approximately twice that calculated. This is quite acceptable in view of the safety factor included in the shear resistance of the nails fixing the holding down strap. At higher vertical loads the predicted racking performance is significantly nearer the test results. This occurs because the vertical load precompresses the panel and the uplift deflection caused by the racking load does not allow the maximum holding down force in the strap nails to be motivated. However, it is clear that the force motivated is in excess of the calculated value as the
predictions produce safe design values throughout the tests. In the 4.8m test the same behaviour pattern is noted although comparison can only be made of stiffness results. At zero vertical load, improvements in racking resistance are greater when the straps are used in combination with nails, due to the nails limited resistance to overturning. At 5 kN/stud the effect is reversed but it must be noted that the failure load is much more likely to govern in the case of the nail tests and thus stiffness will not be indicative of design performance. The bolted tests indicated that the improvement gained by the holding down straps reduces with length, particularly at zero vertical load. This is to be expected in part as the vertical load modification factors have been shown to reduce with length. However it is also possible that the straps will provide less uplift resistance because, in general, the vertical movements in longer walls are less significant than in short units.

It is clear that more tests are required to investigate the effect of failure on these longer walls. The vertical load behaviour of the holding down straps is different in comparison with normal gravity loads because load is only motivated if the panel lifts at the strap position. Uplift is more likely to occur at failure so that on panels where strength is the governing criterion the straps are likely to provide a significant contribution. Thus as the factor of safety reduces for longer panels at higher vertical loads the overall percentage improvement gained by the straps should be higher than that calculated for stiffness. The design method can then be seen to be applicable to all lengths of wall.

It is possible that the effect of holding down straps will be reduced on short window panels where the rotation of the panel is small in comparison with the shear. This is indicated by the results of the short window panel tests where the vertical load factors were reduced and small uplifts were measured. Fortunately, in longer walls with openings the vertical load factor was similar to that of
plain panels and thus, in the general case, the design method for the holding down straps should be acceptable. Clearly this hypothesis should be checked by further tests using holding down straps on long walls with openings.

The final method of holding down covered in tests (DOE tests - Appendix B) used horizontally driven nails to fix the overlapping edge of the sheathing directly to the baseplate. This system is common in practice and may also be used in addition to other fixings between the panel and the base plate. If the horizontal nails are the only form of connection then the design load may be limited by their shear resistance in the direction of load calculated using equations 6.60 and 6.61. The nails will provide a better resistance to uplift than vertically driven nails; therefore design need not incorporate the K116 modification factor. The overlap of the boards must be sufficient to achieve a reasonable edge distance for fixings, particularly if the vertical load is low, to prevent premature failure of the panel due to breakout of the sheathing around the nails in uplift conditions. Test results are shown in Table 6.46. It can be seen that the panel behaved in similar fashion to the standard panel except at zero vertical load where the strength result was very low. This was a consequence of the sheathing overlap being only 30mm which caused a 20% loss of edge distance. Apart from this case design is always governed by nail shear.

It is clear from these tests, and is also seen in the other holding down tests, that using BS 5268 part 6 values for nail shear will restrict the racking performance because the factor of safety built into the Code values is much higher than required by the full scale tests but in this particular instance the base fixing is identical to the standard method. In normal circumstances if the base nailing of the panel is inadequate the factor of safety will approach this higher factor. If more nails are included in the base the factor of safety will reduce to that of the test panel, which is approximately 2.0 for test values of BRR and slightly higher if based on standard design values, and design is
governed by panel behaviour. The nail analysis has not been extended to consider resistance to uplift and thus in situations where uplift is critical to failure, i.e. at low vertical loads, particularly on short panels racking design results will again be critical and the overall safety factor will reduce to that of the panel.

If the horizontal nails were combined with an adequate base fixing, such as holding down bolts, they could be considered to provide an equivalent vertical load in a similar manner to the holding down straps. However no tests are available to check this hypothesis.

In summarising the work on base fixings, the connection shown in Figure 2.6 is used to illustrate the complexity of the practical situation; here the following cases will require to be checked:

1. shear load transferred directly from the bottom rail to the steel channel via the base plate,

2. shear load transferred from the bottom rail to the steel channel via the base plate,

3. shear load transferred to the foundations from the steel channel via the shot nails.

If the design is based on allowable shear values for the nail it is likely that at high vertical load approximately twice as many fixings will be required than would be found necessary if full scale tests were carried out on the wall and the base fixing. This is a result of:

1. friction motivated along the shear surface,

2. the higher factor of safety required of Code values,
(iii) the higher resistance to overturning of a long vertically loaded wall.

As the panel is reduced in length and/or the vertical load is decreased it is probable that the test and design requirements for base fixings will become more compatible because of the increasing effect of overturning which reduces the factor of safety in design towards that used in the test. Hence the following conclusions may be drawn.

(1) The design of base fixings using BS 5268 part 2 values is often conservative in relation to panel design by BS 5268 part 6 hence the acceptance of much wider spaced base fixings in present day practice.

(ii) Short or low vertically loaded walls require a higher concentration of fixings to achieve a specific racking resistance due to the problem of overturning.

(iii) The design rules proposed in this section err towards increased safety by being based on standard design practice. The rules may be relaxed at a later date but at present insufficient tests on longer walls have been carried out to allow a design method to be based on test evidence.

6.7.4 Return Walls

The return wall effect has not been tested in any detail at Surrey; some work was carried out at Princes Risborough in the 1970s but changes in test method since then partly invalidate the results and they are not included in this investigation.
In theory the return wall will resist the uplift of the leading edge of the racking wall by an amount that is either governed by the nails connecting it to the racking wall or its own resistance to uplift in terms of base fixing and vertical load (Figure 6.45). In practice the return wall restraint will lie between the zero condition of the test panel and the maximum restraint discussed in Sections 6.6.4 and 6.6.5 when a preceding panel in the line of the wall applies a rotational force which actively restrains the leading stud of the original wall. The return wall restraint is passive and relies on the uplift of the racking wall for its motivation. Thus its effect will be comparatively small.

If it is assumed that the return wall is tied down firmly to the foundations then the wall will behave in a similar manner to a holding down strap and the effect on vertical load could be calculated using equation 6.59 as discussed in Section 6.6.5. It is clear that for normal fastenings, 3.75mm nails at 300mm centres, on a typical length of wall this will be small and will give little increase in racking load. Furthermore it is probable that the return wall will be prone to uplift in the same way as the racking wall due to the weakness of the connection between the sheathing and the bottom rail. Therefore it is recommended that no improvement is allowed for return walls in standard conditions. However it is noted that the return wall effect could be used to reduce design problems in short walls with poor base fixings when under low vertical loads which might then eliminate the need for the K116 modification factor.

6.8 **SECONDARY FACTORS**

6.8.1 **The External Brick Skin**

The design method for the contribution of masonry included in BS 5268 part 6 (BSI, to be published) which is detailed in Appendix A has not been developed by the author of this thesis. The rules allow only a minimal contribution from the brick wall based on the density of use of a
minimum specification wall tie.

A small number of tests on brick skin walls used in conjunction with timber frame walls have been carried out at Surrey and the results may be compared with the Code guidelines. The test panels have been described in Chapter 5 and full details of the test are given by Griffiths (1978 b). The results are summarised in Table 6.47 together with results for similar tests carried out without the brick cladding. The results show the brickwall contribution to be approximately 3.0 kN for 2.4m panels, regardless of sheathing and vertical load, rising to 6.0 kN for 4.8m panels. These results are based on stiffness but the failure results indicate an adequate factor of safety in all the brick wall tests. The brick wall contribution can be calculated to be 1.25 kN/m for a wall tie density of 2.8 ties/m². This value is notably higher than that of the Code and indicates that the additional factor of safety included in the Code relating to the use of brickwork lies between 3 and 4.

The tests carried out as part of the current investigation are not sufficient to produce an alternative to the Code design method. Thus when discussing the racking resistance of the building in Chapter 8 the design values for brickwork are taken directly from BS 5268 part 6.

6.8.2 The Wetted Panel Effect

Early in the programme of racking work a mediumboard sheathed timber frame panel was tested to simulate a wall which had been wetted by driving rain whilst in the upright position. The effect of the rain had been to cause a differential moisture content through the sheathing such that the swelling of the board made it buckle between the studs. The buckling had made the construction of the brick outer skin difficult as a standard cavity could not be maintained. The test on the trial panel indicated that the
buckling had not caused a reduction in panel performance.

As a result of this work further tests were carried out to examine the effect of more onerous wetting conditions on panels using a range of sheathing materials. The moisture content of the board was increased by immersing the panel in a water bath which limited its length to 1.2m. Initially two new conditions were tested. Firstly, immediately after the panel had been removed from the water bath after immersion for 16 hours, it was tested in stiffness and strength under a 2.5 kN/stud vertical load. Secondly, a similar panel was immersed for the same period of time and then allowed to dry out, it was tested once the moisture content of the frame timber had returned to the value noted prior to immersion. Later two further tests were introduced, they were similar to those described above but including a 90 hour wetting period. All panels were tested for stiffness prior to immersion so that the change in performance could be measured directly.

The panels tested consisted of hem-fir frames using the following sheathings, fixed with 50mm long 3.25mm clout head nails at standard centres:

(1) 8.0mm spruce plywood,
(ii) 9.0mm mediumboard,
(iii) 12.5mm BIIB,
(iv) 6.4mm tempered hardboard.

The results are recorded in Table 6.48. The problems encountered in testing are illustrated in the results of the plywood tests, where the panel immersed in a water bath for 90 hours seemed to have gained in stiffness. This anomaly has arisen because:

(1) preload tests were not carried out on the panels to establish a settled datum,
(ii) the panel had to be removed from the test rig for immersion and it is likely that the results were sensitive to the installation procedure following wetting.

The effect of immersion of the panels is to allow water to penetrate more easily the edges of the sheathing so that the board will swell and damage the boundary fixings. It was noticeable that the boards which offered least resistance to edge penetration of water, BIIB and mediumboard showed the greatest loss in performance and that failure loads in particular were greatly reduced. The more resistive boards like plywood and tempered hardboard showed little sign of swelling and exhibited little loss in performance. It is possible that increases in performance of the panels wetted for a short period were due to a very slight swelling of the board tightening the grip of the nail. It was noticeable that the weaker sheathings such as the insulation boards lost a considerably greater proportion of load during the cyclic testing, thus indicating the importance of the cyclic test to all racking performance work because the first cycle may not identify the true weakness of the materials.

In general the tests showed that:

(i) loss in performance was very much greater after 90 hours immersion when the water had thoroughly penetrated the board to the nail line,

(ii) once the panel had been wetted it would not regain its full performance on drying out,

(iii) losses in the strong category of sheathings were relatively small and could be covered by a reduction factor for wetted panels,
(iv) losses in the weak category of sheathing were substantial and preclude the structural use of such panels once they have been damaged.

It is proposed that a reduction factor $K_{114}$ be included for the strong category boards alone, with a value of 0.75 to cover any panel that has been damaged by water to an extent over and above the normal exposure to rainfall whilst erected but during construction, but does not show visible signs of major damage. For weak category boards $K_{114}$ takes a zero value. It is possible that some sheathings such as type 1 chipboard, which could be classified as category 1 on strength grounds, would not have an adequate resistance to wetting necessitating a $K_{114}$ value of zero and, therefore, a special design clause.

It is clear that the value of $K_{114}$ is very dependent on the type of material and the amount of damage which could be very difficult to estimate. Great care will therefore be necessary in its application. The problems of assessment also require that the number of different cases be limited. The suggested three values (taking $K_{114}$ to be one for perfect panels) are quite adequate.

6.8.3 Load Duration Effect

The racking load determined in all the tests has been taken to be a result of wind loading and thus all design values represent allowable short term loads. Should it be necessary to consider other load duration periods a modification factor $K_{115}$ could be incorporated.

Design values are normally governed by stiffness limitations but with standard timber the modulus of elasticity does not change with load duration. Thus it could be argued that $K_{115}$ is unnecessary. However, even when failure
load does not govern design the strength criterion is often very similar in value to that of stiffness such that for a longer term duration the strength would become increasingly critical. Then, based on the structural use of timber, \( K_{115} \) would take the value:

\[
K_{115} = \frac{K_3}{1.5} \quad - 6.65
\]

where \( K_3 \) is the load duration factor from BS 5268 part 2. The performance of the timber frame panel however is, in the main, related to fixing performance i.e. in attaching the sheathing to the frame, in joining panels and in base fixings; an argument can therefore be made to replace \( K_3 \) by \( K_{48} \) which would increase racking load capacity. Furthermore, it is noted that the design values are based on tests which take longer to perform than the short term modelled by design.

In view of this conflicting information and because no test data is available it is proposed that the safest condition is adopted, hence equation 6.65 is suitable for design.
Figure 6.43 Typical Results for the Panel Joint Tests
<table>
<thead>
<tr>
<th>Joint Details</th>
<th>0-10mm Deflection Tests</th>
<th>0-15mm Tests</th>
<th>Average Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Zero Vertical Load</td>
<td>5kN/stud Vertical Load</td>
<td>5kN/stud Vertical Load</td>
</tr>
<tr>
<td></td>
<td>Panels 1 &amp; 2</td>
<td>Panels 3 &amp; 4</td>
<td>Panels 1 &amp; 2</td>
</tr>
<tr>
<td>Top rail and bottom rail continuous in all tests</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Nails</td>
<td>-19%</td>
<td>-22%</td>
<td>-5%</td>
</tr>
<tr>
<td>4 Nails</td>
<td>D</td>
<td>A</td>
<td>T</td>
</tr>
<tr>
<td>8 Nails</td>
<td>+7%</td>
<td>+7%</td>
<td>+3%</td>
</tr>
<tr>
<td>16 Nails</td>
<td>+11%</td>
<td>+9%</td>
<td>+3%</td>
</tr>
<tr>
<td>4 Nails 3 Bolts</td>
<td>+19%</td>
<td>+17%</td>
<td>+5%</td>
</tr>
</tbody>
</table>

Table 6.43 Relative Performance Values for Vertical Joints Between 1.2m Long Panels
<table>
<thead>
<tr>
<th>Vertical Load (kN/stud)</th>
<th>Test Results (kN) (1)</th>
<th>Design Values (kN)</th>
<th>Proposed Fixing (3) Design Values (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>200mm spacing</td>
<td>100mm spacing</td>
<td>Racking load</td>
</tr>
<tr>
<td>Base Nail 75mm long, 3.75mm dia.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>stiffness design load</td>
<td>strength (2) design load</td>
<td>stiffness design load</td>
</tr>
<tr>
<td>0</td>
<td>3.53</td>
<td>1.57</td>
<td>5.08</td>
</tr>
<tr>
<td>2.5</td>
<td>7.82</td>
<td>5.12*</td>
<td>8.36</td>
</tr>
<tr>
<td>5</td>
<td>9.84</td>
<td>8.66</td>
<td>10.12</td>
</tr>
</tbody>
</table>

**Notes**

1. Panel Materials: BRR code = 1.82kN/m, BRR test = 2.30kN/m.
2. Design value governed by strength in all cases.
3. Includes modification factor $K_{116}$.
   * Interpolated value.

Table 6.44 The Racking Performance of 2.4m Panels using a Vertical Nailing Type of Base Fixing
| Vertical Load kN/stud | Tests Results (kN) (1) | Design Values (kN) |  |  |  |
|----------------------|----------------------|-------------------|----------------|------------------|-----------------|------------------|
|                      | Stiffness Design Load Only | Racking Load | Nail Shear (2) | Racking Load | Nail Shear | Proposed Design Value (kN) |
|                      | Base Nails 75mm long 3.75mm dia. | 300mm spacing | 150mm spacing | 75mm spacing | BRR=2.35 | BRR=1.82 | 150mm spacing | BRR=2.30 | BRR=1.82 | 150mm spacing | 0 | 2.5 | 5 |
| 0                    | 11.3                 | 13.0             | 18.5           | 13.26          | 10.74           | 7.10             | 14.20           | 28.40           |
| 5                    | 23.0                 | 31.0             | 31.0           | 20.46          | 16.55           | 7.10             | 14.20           | 28.40           |

Notes:

(1) Panel Materials: BRR code = 1.82 kN/m, BRR test = 2.35 kN/m

(2) No changes to nail shear design values are proposed

Table 6.45 The Racking Performance of 4.8m Panels Using a Vertical Nailing Type of Base Fixing

<table>
<thead>
<tr>
<th>Vertical Load kN/stud</th>
<th>Tests Results (kN) (1)</th>
<th>Design Values (kN)</th>
<th>Proposed Design Value (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Horizontal Nails 63mmx3.25mm spacing 150mm</td>
<td>Racking Load</td>
<td>Nail Shear</td>
</tr>
<tr>
<td></td>
<td>stiffness design load</td>
<td>strength design load</td>
<td>BRR=2.30</td>
</tr>
<tr>
<td>0</td>
<td>4.77</td>
<td>3.60</td>
<td>5.52</td>
</tr>
<tr>
<td>2.5</td>
<td>7.91</td>
<td>7.00*</td>
<td>7.89</td>
</tr>
<tr>
<td>5</td>
<td>10.10</td>
<td>10.20</td>
<td>9.77</td>
</tr>
</tbody>
</table>

(1) Panel Materials = BRR code = 1.82 kN/m, BRR test = 2.30 kN/m.

Table 6.46 The Racking Performance of 2.4m Panels Using a Horizontal Nailing Type of Base Fixing

* Interpolated value
Figure 6.44 Holding Down Strap Performance

\[ BRR = 2.30 \text{ kN/m} \]

Shear in strap = 10×285×125×1.25 = 4.45 kN

Equivalent = 2.67 kN/stud

Equivalent UDL = 5.56 kN/m

<table>
<thead>
<tr>
<th>Applied Vertical Load (kN/stud)</th>
<th>Tested Design Load (kN)</th>
<th>Predicted Design Load Without Straps (kN)</th>
<th>Predicted Design Load With Straps (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>10.22</td>
<td>5.52</td>
<td>8.00</td>
</tr>
<tr>
<td>2.5</td>
<td>12.36</td>
<td>7.89</td>
<td>9.94</td>
</tr>
<tr>
<td>5</td>
<td>13.01</td>
<td>9.77</td>
<td>11.37</td>
</tr>
</tbody>
</table>

(i) Stiffness governed all test results

(a) 2.4 panels

---

\[ BRR = 2.35 \text{ kN/m} \]

Shear in straps = 6×105×125×1.25 = 3.60 kN

Equivalent stud load = 2.11 kN/stud

Equivalent UDL = 3.96 kN/m

<table>
<thead>
<tr>
<th>Test Details</th>
<th>Applied Vertical Load (kN/stud)</th>
<th>Main Type of Base Fixing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bolts</td>
<td>Nails at 300 ccs</td>
</tr>
<tr>
<td>Test stiffness load</td>
<td>0</td>
<td>14.0</td>
</tr>
<tr>
<td>No straps (kN)</td>
<td>5</td>
<td>21.7</td>
</tr>
<tr>
<td>Test stiffness load with straps (kN)</td>
<td>0</td>
<td>19.5 (39%)</td>
</tr>
<tr>
<td>Improvement on no straps</td>
<td>5</td>
<td>27.0 (24%)</td>
</tr>
</tbody>
</table>

The design improvement due to the additional load created by the straps is:

(i) at zero vertical load 25%

(ii) at 5kN/stud load 11%

(b) 4.8m panels
Figure 6.45 The Return Wall Fixings Affecting the Design Value Applied to the Racking Wall
<table>
<thead>
<tr>
<th>Test Details</th>
<th>Vertical Load</th>
<th>First cycle 5mm Deflection Load</th>
<th>Failure Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>With Brickwall</td>
<td>Without Brickwall (ii)</td>
</tr>
<tr>
<td>2.4m, mediumboard sheathing</td>
<td>0</td>
<td>8.66 (E)</td>
<td>5.00</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>11.73 (E)</td>
<td>9.00</td>
</tr>
<tr>
<td>2.4m, plasterboard sheathing</td>
<td>0</td>
<td>6.62 (i)</td>
<td>3.50</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>9.42 (i)</td>
<td>6.00</td>
</tr>
<tr>
<td>4.8m, mediumboard sheathing</td>
<td>0</td>
<td>20.22</td>
<td>14.00</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>27.30</td>
<td>22.00</td>
</tr>
</tbody>
</table>

Notes:

E = External sheathing
I = Internal Sheathing
(i) = Non standard nails for plasterboard
(ii) = Averaged values

Table 6.47 Brick Wall Test Results
<table>
<thead>
<tr>
<th>Panel Type</th>
<th>Moisture Content When Wet (Increase in Dry Moisture Content)</th>
<th>Percentage Loss in Stiffness of Wet Panel (i)</th>
<th>Percentage Loss in Stiffness of Wet/Dry Panel (i)</th>
<th>Factor of Safety Wet Failure Dry Stiffness</th>
<th>Factor of Safety Wet/Dry Failure Dry Stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plywood 8mm Spruce</td>
<td>25.7% (+15%) 70.9% (+60%)</td>
<td>-8%</td>
<td>-9%</td>
<td>2.67</td>
<td>3.05</td>
</tr>
<tr>
<td>Mediumboard 9mm</td>
<td>17.8% (+7%) 42.7% (+32%)</td>
<td>+1%</td>
<td>-6%</td>
<td>2.96</td>
<td>2.30</td>
</tr>
<tr>
<td>Tempered Hardboard 6.4mm</td>
<td>9.0% (+2%) 30.0% (+23%)</td>
<td>+11%</td>
<td>-2%</td>
<td>2.62</td>
<td>2.35</td>
</tr>
<tr>
<td>B11B 12.5mm Board Type A</td>
<td>47.0% (+38%)</td>
<td>-45% (ii)</td>
<td>-57% (ii)</td>
<td>1.10</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>16 hr test only</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B11B 12.5mm Board Type B</td>
<td>25.0% (+17%)</td>
<td>-65% (ii)</td>
<td>-39% (ii)</td>
<td>0.95</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td>16 hr test only</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B11B 12.5mm Dense Board Type C</td>
<td>21.9% (+16%) 42.3% (+37%)</td>
<td>-14%</td>
<td>-6%</td>
<td>2.50</td>
<td>2.70</td>
</tr>
</tbody>
</table>

(i) Based on first cycle stiffness results

(ii) Using fourth cycle wet test result due to the very high drop of in load

Table 6.48 Wetted Panel Test Results
SECTION D

PANEL BEHAVIOUR DURING RACKING TESTS

6.9 STANDARD PANELS

6.9.1 General Details

This section is concerned with the behaviour of standard panels under racking load. It investigates the displacements of individual elements of typical panels during the 5mm deflection tests and at failure, for the extreme conditions of vertical load. The four principal sheathing/lining types, i.e. plywood, mediumboard, B11B and plasterboard, are covered in detail and their differences, particularly at failure, are noted.

Tables 6.49 to 6.51 include comparative data for these boards for:

(i) stiffness tests under zero and 5kN/stud vertical load,

(ii) failure under zero vertical load,

(iii) failure under 5kN/stud vertical load.

The tests used for comparison were selected because the types of frame and fixing were consistent; the high quality SPF and the 63mm long by 3.25mm diameter gunned wire nails made the overall performance levels slightly above average for the boards. Figures 6.46 to 6.49 show the failure results for each board type and illustrate their main differences in resistance to loading.

The general behaviour of the panels was noted in Chapter 3 whereby the principal elements have different
primary movements under racking load; the frame tries to lozenge and the sheathing boards rotate. These elements are linked by the fixings and their interaction with the board and frame determines the racking resistance of the panel. The movements of the two elements in a standard panel depends on:

(i) the materials, i.e. the sheathing, the fixing and the frame,

(ii) the vertical load,

(iii) the base fixing of the panel.

In general, points (ii) and (iii) determine the relative significance of either lozenging or rotation and the board/fixing/frame interaction determines the amount of racking load and the method of failure of the panel. At the 5mm stiffness test limit displacements of internal elements are small and need to be measured by special gauges (Chapter 5). However, the relative movements of the boards and the nail slips along the frame members are approximately proportional to the overall panel movements and the pattern of results are similar for the different board types, relating to panel shape and vertical load. The mode of failure, however, is more dependent on the materials and the vertical load. For these reasons the stiffness test results and the failure performances will be examined independently.

6.9.2 Panel Behaviour in Stiffness Tests

The standard method of predicting the behaviour of a wall racking panel has been to assume a pin jointed frame, which will shear in the direction of the racking load and has no intrinsic strength, coupled to a rectangular
sheathing, which will rotate and shear. The development of this theory has been noted in Chapter 3. The sheathing, is connected to the frame by nails and is not directly loaded, thus the displacement of the nails depends on the relative movements of the board and frame and typically will be as shown in Figure 6.50. A comprehensive loading diagram, based on these displacements can be drawn up (Figure 6.51). This can be simplified by removing the horizontal forces from the stud in view of the inability of the hinges to transfer very much horizontal load in practice.

The vertical continuity of the theoretical panel through the hinges means that vertical load will have no effect on the panel and so racking load will be independent of it which is definitely not true of standard panel tests. In practice the frame nails do not behave as hinges in the vertical direction unless the joint is in compression. Tension joints will have to be replaced by springs which represent the withdrawal resistance of the frame nails and the vertical loading. Thus the vertical load will now affect panel behaviour and the mode of resistance of the panel will be considerably changed when no vertical load is present (Figure 6.52). A further practical consideration is necessary for tests covered by this investigation in that the racking load is not applied solely to the top rail of the panel. In practice, because the racking jack is fixed in position and the leading stud lifts, some load will be transferred into the stud and will reach the sheathing via the nails in the top of the stud rather than the nails in the top rail. This effect is also shown in Figure 6.52.

The hypothetical cases are shown for 1.2m long panels but the validity of the predicted behaviour pattern can be checked for standard panels when the additional instrumentation described in Chapter 5 has been fitted. Results from such tests are shown in Figures 6.53 to 6.55.
for standard mediumboard and plywood sheathed panels. The first two figures cover first cycle results for two panels for each sheathing type. They show the internal movements recorded between racking deflections of 2 and 5mm. These values were chosen to enable direct comparison to be made with lowest load cycle test results (Figure 6.55) where the panel set could result in the 2mm racking deflection being the first standard measurement point common to both tests. Figure 6.55 also shows the results between 0 and 5mm of an earlier series of tests on mediumboard panels where the studs were a different material and the internal measurements were not so comprehensive.

Considering first the principal results (Figures 6.53 and 6.54) results are shown at both zero and 5kN/stud vertical load and can be compared with the hypotheses advanced in figure 6.52. The nail slips are shown for the 3mm racking deflection and may be used to determine the transfer of load between frame and sheathing. Later small scale tests (Chapter 7) indicate that the initial stiffness of nails in these materials would be on average 1.2kN/mm for plywood and possibly fractionally higher for mediumboard. These results are very approximate, having been recorded for the first 0.5mm deflection interval in a failure test, and they will also vary quite considerably between frame members. Using the 1.2kN/mm stiffness value it can be shown that a 0.05mm nail slip in the eight nails fixing the short board edge (1.2m) equates to a total load of approximately 1kN. Along the longer board edge sixteen nails are used thus the load to produce the same average nail slip would be 2kN. Assuming, for simplicity, a linear relationship between load and nail slip the loads in each joint between board and frame member can be calculated and compared with the applied racking load. The following results may be noted.
a) The shear load motivated by slip in the bottom rail nails is equivalent to racking load in both the zero and 5kN/stud load cases for all panels confirming the horizontal equilibrium hypothesis.

b) If the racking load is divided by the nail slip in the bottom rail the averaged nail stiffness lies between 1.11 and 1.38kN/mm which is well within the likely limits determined by test. The plywood results are slightly stiffer than those of mediumboard.

c) The nail slips in the top rail are considerably less than those of the bottom rail and there is no correlation with racking load. This confirms that a part of the racking load is being transferred to the sheathing via the leading stud as shown in Figure 6.52. It is not possible to determine accurately how the racking load is proportioned between the stud and the top rail. Three analyses were tried to investigate the proportions, each using a different assumption viz:

(i) nail resistance in the top rail is identical to that in the bottom rail of a given panel hence the load in the top rail can be determined as a proportion of the total load,

(ii) nail stiffness in the top rail is taken to be the average value of 1.25kNmm,

(iii) the load transferred to the stud is independent of vertical load.

Other assumptions could be made but, based on the results, the three above are the more reliable interpretations. No specific trends were noted with regard to the horizontal load entering the stud. The analyses were similar because both (i) and (ii) indicated the stud load
to be approximately the same for both vertical load conditions. From the results of the analyses it is reasonable to assume that:

(i) the nail stiffness in the top rail is in agreement with the nail stiffness calculated in the bottom rail.

(ii) the load transferred direct to the stud is approximately 1.9 ±0.35kN, is independent of vertical load and is likely to be related to the precise location of the stud relative to the bottom rail and the positioning of both the loading plate and the roller assembly used to transfer load at the start of the test.

d) The load transferance from the top rail to the sheathings varies for the two boards, in eight cases out of twelve the slip was higher in the second board but, in general, there was no relationship with the vertical load or the particular panel. The load transferance at the bottom rail, indicated by the nail slip, was always greater for the second board and in general exceeded that of the leading board by more than 50%. Taking account of the load reaching the leading board via the leading stud ('c' above) a large proportion of horizontal load is being transferred between boards at their vertical joint on the central stud either directly in bearing or via the stud and 'perpendicular to grain' nail movements. The higher load transferred out of the second board indicates its greater resistance to rotation due to the rotational clamping of the central stud by the leading board.

e) The nail slip in the leading stud is negligible at zero vertical load showing that there is no resistance to uplift. The very small loads detected by the nail slip may be attributed to the withdrawal resistance of the nails.
fixing the stud to the bottom rail. The results indicate that the resistance of the springs shown in Figure 6.52 may be taken as zero at zero vertical load.

f) At 5kN/stud the nail slips measured in the leading studs of the four panels are variable and random. On average an approximate uplift force of 1kN is imparted on the stud by the rotation of the sheathing board. In view of the 5kN load acting on this stud through the top rail it is surprising that uplifts in the region of 0.95mm are recorded for the 3mm of racking movement. The expected behaviour pattern would be for the board to rotate showing increasing nail slip along the leading stud until a maximum force of 5kN was motivated at which time the stud would start to rise. Initially, the centre of rotation of the panel would be midway along the panel length but once the vertical load had been balanced by the uplift force this location would move back along the board and enable further uplift resistance to be generated from the bottom rail nails. The fact that panels with a 5kN/stud vertical load lift immediately and have been shown over many stiffness tests to generate uplift forces only a fraction of the vertical stud load indicates that part of the vertical load applied over the leading studs is transmitted to the foundations other than through the stud itself.

g) Nail slips on the central stud show the leading sheathing to move down relative to the stud and the trailing sheathing to lift. In general the displacements and therefore the forces generated are less for the leading board. If the nail slips for the two boards are summed and compared with the change in racking load (Table 6.53) some consistency in results is noticeable. Taking average values, it is seen that the nail slip is nearly doubled at 5kN/stud but the racking load has only increased by 50%. This indicates a greater percentage of panel shear in comparison with rotation at higher vertical loads.
The differences in nail slip between the two boards should result in an uplift of the stud at zero vertical load but at 5kN/stud the difference, in absolute terms, should in theory exceed 0.25mm before the stud lifts.

h) The nail slips measured in the trailing stud are very variable and quite random in their nature. The values are on average 25% higher under 5kN/stud load but their range is markedly wide. The forces generated by the slips are in the order of 1.2 to 2.8kN at zero vertical load and 2.2 to 4.5kN at 5kN/stud. They always act down into the foundations.

i) The centres of nail slip have been shown in Figures 6.53 to 6.55. They represent a centroidal position where the nail slip in both axes would be zero if a linear relationship existed between slip and lever arm. If the nail slip modulus is taken to be the nail slip divided by the lever arm it is noticeable that the values in the 'x' and 'y' axes are different. The results of comparing the 'y' axis modulus with that of the 'x' axis are shown in Table 6.54 for both board types at both vertical loads. The difference between board types is marked, but apart from one result the factors linking the vertical load cases are similar. Otherwise no general trends are noted.

The rotational movements of the boards found by measuring 'x' and 'y' direction displacements at two points on a board (Chapter 5) are more consistent than the nail slips. The centres of board rotation calculated from these results give a good indication of the general behaviour of the panel and are similar for all panels. They are used to determine the centre of panel rotation which is the point at which the lines joining the centre of the board to its centre of rotation intersect. The angle of rotation about the 'x' and 'y' axes may also be measured independently in order to determine the horizontal shear of the panel due to
the racking load. The shear deflection is calculated to be:

\[
\frac{\text{angular rotation} - \text{angular rotation}}{(\text{about y axis} \quad \text{about x axis})} \times \text{panel height}
\]

The main findings from these calculations are noted below with typical results shown in Figures 6.53 to 6.56.

a) At zero vertical load the centre of rotation of the leading board is external to the board indicating its uplift at the central stud and, noting the relative nail slip of the board on the central stud, uplift of that stud. The centre of rotation of the second board lies within the leading half of the board. The uplift is greater than that of the stud and is confirmed by the direction of nail slip along the leading edge of the board. However, the downthrow of the board at the trailing edge is remarkable as nail slip here is small indicating that the stud compresses against the bottom rail. The average downward displacement for the 3mm racking deflection interval is approximately 0.65mm which, with nail slips less than 0.1mm, shows a joint closure in excess of 0.5mm.

The centres of rotation may be used with the angles of rotation to determine the board's movements at its edges (Figure 6.56). The uplift of the leading edge of the panel agrees approximately with the measured uplift of the stud but is, in all cases, slightly higher. Considered in conjunction with the movements at the centre and rear studs this result infers that the measurement of rotation about the 'x' axis and the location of the centre of rotation may be slightly inaccurate due to the shear movement of the panel.

(c) At 5kN/stud vertical load the centres of rotation for the first board move forward to lie within the length of
the board. This indicates that the trailing edge of the board is moving down. However, the down throw is less than the nail slip measured between the board and the stud suggesting that the centre stud is lifting. The centre of rotation of the second panel does not confirm this, the rotational uplift averages 0.23mm and is less than the slip of the board relative to the stud which averages 0.30mm.

The results for the fourth cycle stiffness test on Panel B (Figure 6.55) clad with mediumboard indicate the following points:

(i) the overall stiffness of the panel increases in later cycles as a result of its inelastic behaviour and the panel set (both discussed in Chapter 5),

(ii) for similar racking deflections the board rotations are approximately identical which, together with other results, shows that for a given panel and test, board and frame movements are directly related to the overall racking deflection,

(iii) the nail slips measured in the later test cycles show considerable reduction which means that the board's nail resistance is strain hardened.

Finally, the results for Panel J (Figure 6.55), also clad with mediumboard but of different species frame timber, which are based on a 5mm racking deflection are very similar to those of Panels A to D and indicate the elastic behaviour of the panels during the first load cycle. In these earlier tests less data was collected so that board rotations could not be accurately measured. In the same test programme internal measurements were recorded on 0.6m
to 4.8m long walls made up of either individual units or combinations of panels. The general principles discussed for the 2.4m walls were again noted and there were no significant differences between boards joined on one stud and those fixed to separate studs linked by three bolts. In the more complex 4.8m wall cases the following points were noted.

(a) Nail slips in the top rail were in total, low in comparison with the bottom rail. The slip in the leading sheathing was very low, maximum slips were achieved in either the second or third boards, i.e. close to the second load point on these walls, and nail slip reduced in the fourth sheathing.

(b) The nail slips in the bottom rail were variable between sheathings but there was no trend to this variation. In total the slips equated well with the racking load.

(c) Vertical nail slips were small in the leading and trailing studs and tended to peak on the central stud, however the results in general were very variable.

(d) The board rotation points (which could only be defined along the 'x' axis) were measured for the four boards from the leading edge of the wall, results were consistent and were averaged to be:

   (i) at zero vertical load: 1800, 2200, 2600 and 3000mm,

   (ii) at 5kN/stud vertical load: 1000, 2100, 2700 and 3800mm.

These results, in conjunction with the vertical nail slips, show the boards at zero vertical load to behave as if they
form a single sheathing and indicate the increased significance of rotational movement. At 5kN/stud the board movements are totally independent but are more alike; thus shear is the dominant factor in the wall behaviour.

(e) The centres of uplift for the walls are, on average, 1500mm at zero vertical load reducing to 600mm at 5kN/stud. The latter result is very similar to the 2.4m wall whereas the former is further back from the leading edge. The results are consistent with the comment noted in (d) above.

The general conclusions that may be drawn for wall behaviour in stiffness tests are:

(i) that the overall racking resistance of similar walls is variable,

(ii) that internal movements within a wall are more variable even for similar racking resistances,

(iii) the variability cannot be avoided because timber is a natural material and because of the acceptable tolerances in panel construction which, although very small, can have a significant effect on the internal distribution of stress through the wall.

Within the current work there is little advantage to be gained in pursuing this particular investigation because the data have little bearing on the design of timber frame walls. A more comprehensive examination of panel behaviour within the stiffness deflection limits could form the subject of a further investigation which could be linked to a theoretical analysis of wall performance. Much greater care would then be necessary in setting up tests to reduce
the variability in performance to a minimum.

The work on panel failures noted in the following sections is of greater relevance because it indicates any general weakness in the construction form which may need to be considered when defining design values.

6.9.3 Panel Behaviour at Failure Under Zero Vertical Load

Under zero vertical load a single zone of weakness is responsible for the failure of all test panels. This is the connection between the sheathing and the bottom rail of the wall at its windward or leading end. Here the rotational action of the racking force lifts the sheathing relative to the bottom rail which is tied to the foundations and there are no beneficial restoring moments from either vertical load or a preceding sheathing board. The mode of failure is dependent on the construction materials but, in the first instance is governed by the sheathing. The behaviour of the joint determines the uplift that can be sustained by the panel which in turn is linked approximately to the racking deflection because the point of rotation has been shown to be relatively constant for all board types. Thus a brittle board will cause panel failure at very low racking deflection and a ductile connection will allow racking deflections in excess of 50mm at failure with a maximum load plateau being maintained for much of this movement. The different modes of failure will be assessed by considering the types of sheathing.

Clearly plasterboard is a very brittle material which can accept very little nail movement within the board and is weak both in tension and shear close to the board edge. In a nailed connection the board fails with the break out of vee shaped wedges from its edge under each nail as it is progressively lifted away from the bottom rail.
along the length of the panel. Very little movement of the nails is noted or deformation around the nail holes in the frame. Consequently the panels fail at a very low uplift and racking load deflections and the factor of safety related to the 5mm deflection load is very small such that the failure load will govern design. Failure occurs in the plasterboard itself and the cardboard lining which holds the board together has no strength unless the board is laid horizontal when the card wraps round the bottom edge, or the board is glued to the frame. At failure the nail head passes through the paper such that no tear mark is noted below the nail. The following factors can influence the strength of the joint. These are:

(i) the edge distance of the board,
(ii) the orientation of the board,
(iii) overdriving the nail,
(iv) nail spacing.

Referring to item (i) it is necessary to maintain the maximum possible edge distance in a brittle material firstly to increase the shear failure plane and secondly to reduce the damage done when driving the nail. In the latter case, if the edge distance is small the driving action can shear out the edge sections. Considering item (ii) in the vertical case the bottom edge of the board is unprotected whereas if it were tested horizontally the paper wrapped around the board edge would contain the plaster and increase performance. Overdriving the nail in all cases can be a problem because the effect of the hammer hitting the brittle board is to shatter it locally such that all resistance to uplift is lost. When nails are too closely spaced the board will fail along a tensile crack linking the nails which may present a weaker plane. This
is an extreme case but occurs when plasterboard is glued to the bottom rail.

The overall behaviour of the plasterboard panel is detailed in Table 6.50 and shown in Figure 6.49 where very low deflections at failure are noted. The uplifts and racking deflections are generally in similar proportions compared with plywood and mediumboard.

Mediumboard is also rather brittle in its nature but is much stronger than plasterboard and does allow some nail movement within the board without failure. At zero vertical load the typical mode of failure is to have vee notches broken out of the bottom edge of the leading board. However, if good edge distances are maintained the board strength is such that the load capacity will also cause damage to the fixing, particularly if it is of small diameter. This will show in an upward bending of the nail which causes it to turn its head into the board. Only in very weak frames will significant withdrawal of the nail be noticed before the board fails. Thus Table 6.50 and Figure 6.47 show the board to be brittle in its failure mode but to allow much higher racking and uplift deflections than plasterboard. The comparatively high uplifts experienced in the recorded tests are unusual and normally the results would be more like plywood.

Bitumen impregnated insulation board is a very weak board but is more ductile in its behaviour. The board fails by the tearing action of the nail through its thickness which is a bearing failure. Because the loads are so low no damage to nail or frame is likely. The weakness of the board results in the uplift of the panel being reduced in comparison with other board types. The failure curve (Figure 6.48) shows the maximum load to be held for a large proportion of the overall deflection but the total racking deflection is low and the maximum load
plateau is quickly achieved so that factors of safety are low.

Plywood may be classified as a strong sheathing in that the board very rarely fails. The lay up of the plies prevents edge breakout or tearing of the board. However, it is relatively weak in bearing so that large nail movements can be expected within the board. Uplift of the board will cause the nails to bend and pull out of the frame and their heads will turn into the sheathing which tends to pull away from the frame. If the frame material is weak the maximum load will be limited by the pull out resistance of the nail, otherwise it is possible that the head of the nail will turn through more than 60° and will pull through the board. In both cases the mode of failure is plastic (Figure 6.46) such that the maximum load is maintained over a large deflection interval.

In some early tests where the racking resistance was very high and the holding down arrangements of the panel were inadequate the failure occurred in the frame material, in the same location. In some cases a tension crack formed in the bottom rail along the line of the nails whilst in others, when the washers for the holding down bolts were of inadequate size, the bottom rail cupped and failed in tension along its underside in a line passing through the bolt holes. Such failures were indicative of very strong sheathings; typically tempered hardboard and the thick flakeboards.

In general the failure mode is very much dependent on the strength of the sheathing relative to the fixing and frame. Both tempered hardboard and flakeboard are strong in tension and dense, which prevents nail movement, thus failure is more likely in the fixing. However, as the dense board prevents twisting of the nail through the thickness of the board the pull out form of failure is
reduced, shear in the nail is also unlikely and so a frame failure becomes possible. Of the other boards tested, chipboard is rather brittle but behaves like mediumboard if sufficiently thick and carefully fixed, building boards are very brittle and in many instances benefit from being predrilled. In general they behave like plasterboard. Returning to the general behaviour of the panel, as the racking load increases so too does the deflection and uplift of the leading stud. Failure will occur first around the frontmost fixing of the sheathing to the bottom rail. This is unlikely to cause failure of the panel, the racking load may drop for a short interval, due to the hydraulic loading system but the load will be redistributed and as uplift continues further nails will fail progressively along the bottom rail until a maximum condition is reached after which load will drop off. In brittle boards the load shedding process is sudden and redistribution is difficult leading to a sudden and brittle overall panel failure. In stronger boards load shedding is more easily achieved and higher loads and more ductile failures are possible. In the more ductile boards like plywood and B11B the failure response is quite plastic.

The weakness in the bottom rail connection is so predominant at zero vertical load that high stress is very rarely noticed in other parts of the panel unless failure is ductile and racking deflections very high. Even then, it is unlikely for damage to be recorded before the maximum load has been reached. The secondary failures will be similar to those, noted below, under high vertical loads.

The failures described above refer to the standard test panels which have been bolted to the base of the test rig. Different types of base fixing can change the failure mode if they have a weaker resistance to uplift than the horizontally driven nails attaching the sheathing to the bottom rail (e.g. vertical nailing of the bottom rail to
the sole plate).

It is notable in Table 6.50 that sliding deflections are low and are in proportion to the racking load. This indicates the adequacy of the base fixing in resisting shear. Weaker connections such as nailing or holding down straps allow much larger slips.

6.9.4 Panel Behaviour At Failure Under 5kN/Stud Vertical Load

At 5kN/stud loading the uplift of the board is greatly reduced and similar failures to those at zero vertical load occur only after much increased racking deflection and thus at higher loads. Much of the extra load is carried into the second of the 1.2m wide sheets cladding the panel and the joint of the two boards on the centre stud is an area of weakness because the nails have had to be driven within 10mm of the board edge. Relative movements at the joint can be as much as 20mm for a racking deflection of 70mm with the leading board dropping and the trailing one lifting by approximately equal amounts under the rotational effect of the racking load. Large nail displacements soon become evident resulting in either brittle failure of the board edge or a buckling out of the trailing board as it tears through several fixings due to the high compressive stresses built up in its top corner due to the restraint on the board imposed by the fixings to the top rail. It is significant that a similar failure does not occur in the bottom edge of the leading board although there is a tendency with brittle materials for the top corner to fail in tension. The types of failure mode along the centre stud are dependent partially on board properties, i.e. brittle boards will break about their edges and corners because they cannot accept high nail movements whereas weak boards like B11B, and occasionally
less dense boards like plywood, allow the nail heads to be pulled through their thickness. But in general the displacement in due to nail movement and catastrophic failure is not reached consequently the maximum load will still be limited by the bottom rail fixing weaknesses previously noted.

It is noticeable in Table 6.51 that a high percentage of the maximum load is obtained after only 20mm of racking deflection. At this stage very little relative movement will have been noted along the board joint. Deflections are likely to double, for all sheathings, before a significant drop off in load is noted. Here table 6.51 does not do justice to medium board or B11B because deflection would typically exceed 50mm. It is notable that prior to failure, in either case, uplifts under the 5kN/stud loading regime are approximately half those at zero vertical load. Sliding is again proportional to the applied racking load. Factors of safety will be much higher at this vertical load and only brittle boards such as plasterboard and weak B11B will have their design values governed by failure.

Figure 6.57 details frame and board movements at failure for a hybrid sheathing under a 5kN/stud vertical load. The sinusoidal movement of the top rail is quite marked and is due to the relative movements of the sheathing towards and the action of the vertical loads clamping the top rail to the studs. Uplift of the studs is less noticeable and normally affects only the first two studs. At the leeward end of the panel the rotation of the boards forces the last stud down onto the bottom rail without any relieving moment from a subsequent board. A significantly high load is induced and is often sufficient to cause crushing of the bottom rail directly under the stud. The sheathing movements illustrate the points noted previously.
At zero vertical load there is less movement between the boards and thus virtually no sinusoidal movement of the top rail. Stud uplifts are greater and are possible in all but the last stud, bottom rail crushing under this stud is not common.

6.10 WALL UNITS OTHER THAN STANDARD PANELS

6.10.1 Long Plain Walls

The initial movements of long plain walls were analysed in section 6.9.2. At failure their behaviour is even more closely related to that of the standard panel. At zero vertical load, the uplift in the first board increases slightly with length and the uplift of studs is noticeable further from the leading edge. The boards initially behave as one with little detectable relative movement between them but as deflection increases they begin to rotate independently and, due to the greater lever arms in longer walls, this occurs at decreasing deflection levels as overall length increases. Failure again occurs in the leading board at its joint with the bottom rail but, due to the subsequent boards, a greater number of fixings will be affected before maximum load is reached and thus the failure is more ductile.

At 5kN/stud the board movements are more independent and follow the pattern shown in Figure 6.57 with the subsequent boards behaving in a similar manner to the second board. Vertical displacements between boards will be noticeable long before maximum load is reached and the amounts will be similar at each joint. The crushing of the bottom rail under the trailing stud will become more evident in longer walls.
The increasingly independent nature of board rotation in longer panels is clearly in agreement with the linear wall length analysis outlined in section B of this chapter.

6.10.2 Walls With Openings

The behaviour of walls with openings can be checked for either standard 2.4m panels when the specific effect of a window or a door has been measured, or 4.8m walls when the general effect of multiple openings has been noted.

A window in a 2.4m panel reduces the width of the full height panels to such an extent that bending is clearly visible in the frame and, to a lesser extent in the sheathing. Thus racking deflection is due to shear, rotation and bending, and consequently the first two are reduced in importance. This is seen in the reduced uplift recorded for given racking deflections in window panels. The bending in the frame is noticeable immediately above the bottom of the opening, and a reverse curvature may be seen close to the lintol, particularly if it is deep, of solid construction and well fixed to the wall pillars. The sheathing follows the bend of the frame, but being more rigid, does so by rotation; this causes crushing of the board at the lower leading window corner, bowing of the sheathing in the window pillar and the opening out of the joint at the lower trailing corner of the window. (Figure 6.37). The movements of the sheathings above and below the opening are noticeably small relative to the frame. As noted in Section 6.6 window panels are strengthened if the sheathing is continued around the window. The same failures noted above are obtained but the movements are different because the bending in the window pillar is now resisted by the full board width under the window which
also reduces the rotation of the board. The panel uplift lies between that of the rectangular sheathed opening panel and the plain panel. (Figure 6.37).

Door openings cause a major break in a wall and, unless the lintol is either deep or built into the door pillar, the following part of the wall will behave in a similar manner to an unrestrained test panel. The door lintol will then hinge upwards to accommodate the uplift of the panel. The shorter the lintol or the stronger it is built into the door pillar, the more restraint it will offer against uplift and thus the stronger will be the wall. When a door opening is close to the leading edge of a wall, the leading pillar will behave as an individual panel, thus if it is 600mm or less it will show very little resistance to racking load; uplifts will be very low because shear and even bending dominate such panels. These results indicate the importance of positioning holding down straps on either side of door openings if they are being used to improve the racking capacity of a wall.

Both door and window panels exhibit the vertical load effects noted in plain panels whereby the uplift is further reduced and a more ductile failure mode occurs. It should be noted that because of the reduced uplift and the effects of bending short window panels exhibit very large deflections before the racking load reduces. Clearly this is much more than could be withstood by the glazing unit. To illustrate the effect of openings on overall performance Table 6.55 and Figure 6.58 compare a plain 4.8m wall with one containing two openings. The maximum racking load is not reached in either case. The more linear load/deflection response of the panel with openings is noticeable and can be shown to be typical hence there is less problem with failure loads in perforate walls. The reduction in uplift and sliding in the wall with openings is clear at both vertical loads. The difference in the
front and rear deflection is of interest; for the wall with openings and for the plain wall under 5kN/stud load the deflections are almost identical, however, at zero vertical load in the plain panel the rear deflection is less than 90% that of the front. This is probably due to the higher uplift of the leading stud and the resultant greater rotation of the top rail because no restraint is offered in the form of an applied vertical load. In typical walls the more general points noted in the standard length panel tests can be observed. Thus the following points are a summary of the work on panel behaviour.

(a) Framework tend to lozenge and bend, the shorter the full height length the greater the bending and thus the weaker the racking resistance.

(b) Sheathings tend to rotate but will also shear and bend if used in narrow widths when they are particularly weak. If two thin sheathings abut one another over their full length the vertical shear motivated at the joint may be sufficient for them to behave as a single board. This effect has already been noted in the behaviour of larger boards in plain walls under zero vertical load.

(c) The wider the sheathing the greater its racking resistance due to the couple motivated between its leading and trailing edge and the greater the moment developed by the fixings to the top and, in particular, bottom rail.

(d) At zero vertical load the wall tends to act as a single unit so that uplift is noticeable in most of the studs. Very little resistance to rotation is achieved in the leading sheathing as it is effectively only fixed along its bottom and trailing edges.

(e) At higher vertical loads the boards act more independently because the studs are restricted in their
uplift; this strengthens the wall because more work is done in the vertical joints between sheathings. Uplifts are reduced and failures become more ductile.

(f) Windows have the effect of breaking walls into narrow full height panels which reduce their strength and also decrease uplift and make the walls more ductile at lower vertical loads.

(g) Door openings effectively break up walls into separate units particularly if the opening is long and the lintel weak in its connection to the plain wall.
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Table 6.49 Comparison of Panel Displacements for Different Sheathings in Stiffness Tests

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Table 6.50 Comparison of Panel Displacements for Different Sheathings In Zero Vertical Load Failure Tests
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Table 6.51 Comparison of Panel Displacements for Different Sheathing in SkN/Stud Vertical Load Failure Tests
Figure 6.46  Racking Load Versus Deflections And Uplift Versus Reflection Plots for Failure Tests on Standard Panels Sheathed With Plywood

Figure 6.47  Racking Load Versus Deflection and Uplift Versus Deflection Plots For Failure Tests on Standard Panels Sheathed With Mediumboard
Figure 6.48 Racking Load Versus Deflection and Uplift Versus Deflection Plots For Failure Tests on Standard Panels Sheathed with Bl1B

Figure 6.49 Racking Load Versus Deflection and Uplift Versus Deflection Plots For Failure Tests on Standard Panels Sheathed With Plasterboard

-411-
Figure 6.50 Theoretical Movements on a 1.2m Long by 2.4m High Panel
(i) **Forces**

Major characters represent frame member:

- TR - Top rail
- BR - Bottom rail
- LS - Leading stud
- IS - Intermediate stud
- CS - Centre stud
- TS - Trailing stud

Subscripts detail direction and location of forces:

- H - Horizontal
- V - Vertical
- L - Leading stud end
- T - Trailing stud end
- U - Upper
- B - Lower

(ii) **Dimensions**

The direction of the dimension is related to a standard cartesian system (x, y etc) and the dimension is referenced by a subscripted number.

---

Table 6.52 Details of Notation Use in Figures 6.51 and 6.52
Vertical load is considered to pass directly through the studs and does not affect panel behaviour.

All internal forces must be in equilibrium.

All external forces are in equilibrium.

Internal and external forces must equate.

In practice $LS_{HU}$, $CS_{HU}$ and $TS_{HU}$ together with $LS_{HB}$, $IS_{HB}$ and $TS_{HB}$ depend on how much load can be transferred through pin joints in the frame. In reality the pin joints represent the framing nails and the horizontal forces are those which can be taken by the nails in shear. Noting that the nails act in the end grain of the stud these loads will be small and the horizontal stud loads can be discounted. The monitoring system does not allow any checks of such strains for practical reasons and as a result of the very small values of strain detected in early tests (see Chapter 5).

Figure 6.51 Theoretical Nail Resistance Model
Note: no external loads are transferred directly into the sheathing

(a) Frame details

As noted in Figure 6.51, all internal and external forces must be in equilibrium

**Horizontal Equilibrium**

Applied Load = TR₀ + LS₀
  = BR₀
  = Base reaction

(this applies to both Case A and B)

**Vertical Equilibrium**

Case A  \( BR_{VL} = TS_V \)

Case B \( BR_{VL} + LS_V + TR_{VL} = BR_{VT} + TS_V + TR_{VT} \)

**Rotational Equilibrium**

Found by taking moments about zero strain point on the bottom rail and using \( x_1, y_1 \) etc for lever arm distances.

Case A  \( BR_{VL} \cdot x_1 = TR_H \cdot y_1 + LS_H \cdot y_2 \)

Case B \[ BR_{VL} \cdot x_1 + LS_V \cdot x_2 + BR_{VT} \cdot x_3 + TS_V \cdot x_4 = TR_H \cdot y_1 + LS_H \cdot y_2 + TR_{VL} \cdot x_5 + BR_{VT} \cdot x_6 \]

Figure 6.52 The Practical Nail Resistance Model
Figure 6.53 Internal Displacements in Standard Mediumboard Sheathed Panels

Panel A: Mediumboard, first cycle.
Zero vertical load upper, 5kN/stud lower.

Panel B: Mediumboard, first cycle.
Zero vertical load upper, 5kN/stud lower.
Figure 6.54 Internal Displacements in Standard Plywood Sheathed Panels

Panel C. Plywood, first cycle.
Zero vertical load upper, 5kN/stud lower.

Panel D. Plywood, first cycle.
Zero vertical load upper, 5kN/stud lower.
Figure 6.55 Internal Displacements for Special Cases
Using Standard Mediumboard Sheathed Panels
Table 6.53 Comparison of Summed Nail Slips On The Central Stud of Standard Panel With Applied Racking Load

<table>
<thead>
<tr>
<th>Panel</th>
<th>Sheathing</th>
<th>Zero vertical load</th>
<th>5kN/Stud Vertical Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Sum of Nail Slips On Central Stud (mm)</td>
<td>Racking Load (kN)</td>
</tr>
<tr>
<td>A</td>
<td>Mediumboard</td>
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<td>3.17</td>
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<tr>
<td>B</td>
<td>Mediumboard</td>
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<tr>
<td>C</td>
<td>Plywood</td>
<td>0.354</td>
<td>2.87</td>
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<td>D</td>
<td>Plywood</td>
<td>0.203</td>
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<tr>
<td>Average</td>
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<td>0.287</td>
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</table>

Table 6.54 The Relationship Between Nail Slips In Panel Rails and Panel Studs

* Nail Slip Modulus in y axis = \[
\frac{\text{sum of nail slips in top and bottom rails}}{\text{board height}}
\]

* Nail Slip Modulus in x axis = \[
\frac{\text{sum of nail slips in front and back rails}}{\text{board width}}
\]

+ Nail slips summed algebraically in terms of rotation caused about the centre of nail slip
KEY

Vertical load

0.75 0.06 0.17 0.52

5kN/stud

0.06

0.61

0.63

1.35

1.29 0.19 0.39 0.75

Vertical movements calculated for the board edge (mm)

0.86 0.05 0.28 0.63

Uplift of leading stud (mm)

0.92 = 0.86 + e_1 - 0.06

0 = 0.11 - (0.63 + e_4)

0.31 = (0.39 + e_2) - (0.19 + e_3)

Compatibility equations

e_1 = Too little uplift

e_2 = Too much uplift

e_3 = Too little uplift

e_4 = Too much downthrow

Nail slip between board and stud (mm)

Tails show movement of board

Nail slip between boards on the centre stud (mm)

Measured vertical movements 150mm in from board edge (mm)

Vertical movements calculated for the board edge (mm)

0.01

0.07

0.31

1.29

0.19

0.39

0.75

1.35 = 1.29 + e_1 - 0.01

0 = 0.07 - (0.75 + e_4)

Note: * Assumes stud in contact with the bottom rail and unable to move down. In practice there will be a small constructional tolerance.

Figure 6.56 Vertical Movement Discrepancies Noted When Comparing Board Rotations with Nailslips

(For Panel A but typical of all four panels)
Sinusoidal deflection of toprail under combined vertical and racking loads.

Top rail
- Compression failure zone
- Tension failure
- Large relative movement on central stud

Well restrained bottom rail
- Uplift of studs
- Trailing stud crushes bottom rail

Deflection greater than 50mm at failure.

Uplift and initial failure zone

Figure 6.57 Typical Frame and Board Movements At Failure In a Standard Panel
Figure 6.58 Comparative Results For 4.8m Walls
(Failure Conditions Not Reached)
### Details for Wall A1+A2+A3+A4 at 6kN/Stud

<table>
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<tr>
<th>Racking deflection at front of panel (mm)</th>
<th>Racking load (kN)</th>
<th>Racking deflection at rear of panel (mm)</th>
<th>Uplift of leading stud (mm)</th>
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### Details for Wall A1+A2+A3+A4 at 5kN/Stud

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(a) Imperforate wall

Table 6.55 Comparative Test Details For 4.8m Walls (Failure Conditions Not Reached)
### Details for Wall G1+H1+F11+E11 at 0kN/Stud

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<tr>
<th>Racking deflection at front of panel (mm)</th>
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<th>Racking deflection at rear of panel (mm)</th>
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(b) Perforate wall

Table 6.55 Comparative Test Details for 4.8m Walls (Failure Conditions Not Reached)