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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VOLUME II
# LIST OF CONTENTS

## VOLUME I

### Chapter 1: INTRODUCTION

1.1 Timber Frame Construction ........................................ 1  
1.2 The Forces on a Timber Framed Building ....................... 3  
1.3 The Need for a Design Method for  
    Timber Frame Walls ........................................... 4  
1.4 The Background to the Investigation .......................... 6  
1.5 The Aims of the Investigation ............................... 7  

### Chapter 2: THE DEVELOPMENT OF TIMBER FRAME WALL CONSTRUCTION

2.1 History of Timber Frame  
    2.1.1 Traditional timber frame construction .................. 11  
    2.1.2 Modern Methods of timber frame  
         construction ........................................... 13  
    2.1.3 The development of platform frame  
         construction in Britain ................................. 15  

2.2 Timber Frame Construction Details  
    2.2.1 General .................................................. 18  
    2.2.2 Foundations and base fixings .......................... 18  
    2.2.3 Walls and panel fixings ................................ 21  

2.3 Regulations Covering the Construction  
    of Timber Frame Houses  
    2.3.1 General .................................................. 29  
    2.3.2 Construction controls in Britain ..................... 29  
    2.3.3 Wall design and test regulations abroad ............... 32  

2.4 Timber Frame Panel Test Methods  
    2.4.1 General .................................................. 35  
    2.4.2 The development of the British  
         racking test ........................................... 36
2.4.3 The development of American test methods 40
2.4.4 Racking tests in other countries 42

2.5 Summary 45

Chapter 3: A REVIEW OF TIMBER FRAME WALL INVESTIGATIONS

3.1 Introduction 56
3.2 Early work 57

3.3 Analytical Research
3.3.1 Linear nail slip analyses 59
3.3.2 Non-linear nail slip analyses 62
3.3.3 Japanese one-third scale tests 66
3.3.4 Finite element analyses 67
3.3.5 Other analytical methods 68

3.4 Test Programmes
3.4.1 Panel tests carried out abroad 70
3.4.2 British test work 73
3.4.3 Whole House testing 76

Chapter 4: AN INTRODUCTION TO THE DESIGN OF TIMBER FRAME HOUSES

4.1 The Need for a Design Method 89
4.2 Problems Associated with Assessing the Performance of Buildings 90
4.3 Assessing Building Adequacy Through the Strength of Individual Walls 92
4.4 Wall Design 97
4.5 The Standard Panel Test 99
4.6 Small Scale Tests 102
4.7 Design by Computer Based Analysis 103
4.8 Summary 105
Chapter 5: THE EXPERIMENTAL RACKING TEST

5.1 Test Procedure
   5.1.1 Introduction 113
   5.1.2 The original Princes Risborough laboratories test 115
   5.1.3 The procedure for a standard test programme 119
   5.1.4 Variations for investigative test programmes 124

5.2 Reduction of Test Results
   5.2.1 Introduction 126
   5.2.2 The PRL/U of S test 128
   5.2.3 The reduction procedure for a standard panel test programme 130
   5.2.4 Presentation of standard test results 134

5.3 The Racking Test Rig
   5.3.1 General introduction 138
   5.3.2 The University of Surrey test rig 141
   5.3.3 The loading system 145
   5.3.4 Deflection measurement 147

5.4 Test Panels
   5.4.1 General details 150
   5.4.2 Panels for standard tests 151
   5.4.3 Panels for investigative work 153

Chapter 6: TEST RESULTS AND DESIGN DATA

6.1 Introduction 174

Section A STANDARD PANEL TEST RESULTS AND FACTORS AFFECTING BASIC RACKING RESISTANCE

6.2 Results of Standard Panel Tests 177
6.3 Discussion of Results

6.3.1 Plywood 181
6.3.2 Mediumboard 182
6.3.3 Bitumen impregnated insulation board 184
6.3.4 Tempered hardboard, chipboard and waferboard 185
6.3.5 Plasterboard 186
6.3.6 Building boards 189

6.4 Basic Racking Resistance For the Standard Use of the Sheathings In Tests

6.4.1 Single sheathings 190
6.4.2 Datum racking resistance 196
6.4.3 Combined sheathings 197
6.4.4 Materials modification factors 202
6.4.5 The frame 207
6.4.6 Summary 209

Section B FACTORS AFFECTING WALL RACKING PERFORMANCE

6.5 The Major Test Programmes 235

6.6 Wall Design Modification Factors
6.6.1 Reduction of results 249
6.6.2 Vertical load 250
6.6.3 Wall length 256
6.6.4 Plain walls 266
6.6.5 Openings 278
6.6.6 Summary of wall design factors 295

Section C FACTORS AFFECTING THE USE OF THE WALL

6.7 External Panel Fastenings
6.7.1 Introduction 352
6.7.2 Joints between panels 353
6.7.3 Base fixings
6.7.4 Return walls

6.8 Secondary factors
6.8.1 The external brick skin
6.8.2 The wetted panel effect
6.8.3 Load duration effect

Section D PANEL BEHAVIOUR DURING RACKING TESTS

6.9 Standard Panels
6.9.1 General details
6.9.2 Panel behaviour in stiffness tests
6.9.3 Panel behaviour at failure under zero vertical load
6.9.4 Panel behaviour at failure under 5 kn/stud vertical load

6.10 Wall Units Other Than Standard Panels
6.10.1 Long plain walls
6.10.2 Walls with openings

VOLUME II

Chapter 7: A COMPUTER BASED STRUCTURAL ANALYSIS

Section A THE COMPUTER PROGRAM

7.1 Introduction

7.2 The SADT Computer Program

7.3 Alterations to the SADT Program

Section B SMALL SCALE NAIL TESTS AND TRIAL RUNS

7.4 Nail Tests
7.4.1 Introduction
7.4.2 Cover nail tests
8.2 Wall Design
8.2.1 Philosophy
8.2.2 The maintenance of safety factors throughout the alternative design methods
8.2.3 Material and wall modification factors
8.2.4 Vertical load
8.2.5 Wall length
8.2.6 Wall openings
8.2.7 Damage and load duration

8.3 The Summation of Wall Loads
8.3.1 Introduction
8.3.2 Plasterboard lined walls
8.3.3 Brick skin walls

8.4 Design of Buildings
8.4.1 Applied loads
8.4.2 Moment considerations
8.4.3 Inadequate resistance

8.5 The Evaluation of Wall Racking Loads
8.5.1 General
8.5.2 Datum racking resistance
8.5.3 Basic racking resistance
8.5.4 Modification factors
8.5.5 The contribution of plasterboard to racking resistance
8.5.6 The contribution of masonry veneers to racking resistance

8.6 The Simplified Design of Timber Frame Walls
8.6.1 Introduction
8.6.2 Design values and modification factors
8.6.3 Base fixings
Chapter 9: SUMMARY AND CONCLUSIONS

9.1 The Current Investigations 583

9.2 Recommendations for Further Work 590

9.3 Finale 591

BIBLIOGRAPHY 592

APPENDIX A A1–A24

APPENDIX B B1–B7

APPENDIX C C1–C7

APPENDIX D D1–D27
CHAPTER 7

A COMPUTER BASED STRUCTURAL ANALYSIS

SECTION A

THE COMPUTER PROGRAM

7.1 INTRODUCTION

It was noted in Chapter 3 that simplified analytical techniques are not wholly suited to covering the complexities of the timber frame wall. The American proposals for analytical methods (Itani et al., 1982; Easley et al., 1982; Gupta and Kuo, 1984; etc.) and those developed by Burgess (1982) are all unsuitable as they are based on the ASTM test procedure and ignore the important factor of vertical load when assessing panel performance.

The empirical method of analysis covered in Chapter 6, based on a wide range of tests, has virtually eliminated the need now for a simplified theoretical analysis linked to the British assumptions of panel behaviour and requirements for design. There remains a need, however, for a more accurate and complete method of analysis in order to:

(i) develop a greater understanding of the behaviour of the wall units,

(ii) check the acceptability of the empirical design factors and their inter-relationship,

(iii) extrapolate the test data so that the design method can have a wider range of coverage without the need for further major test programmes.

The complexity of the wall structure and the large number of variables that must be considered have meant that only a computer based method of analysis, probably using finite elements, would meet the requirements. The preparation of
an independent analysis was out of the scope of the current investigation and thus an attempt was made to find a tried and tested structural analysis computer package which could accept the full range of variables and which could be developed to cover the design of timber frame walls.

Four alternatives were considered at the end of a preliminary investigation. They were:

(i) the "Lusas" structural analysis suite already set up for running on the University of Surrey's Prime computer network,

(ii) the "Imperial College" finite element package at present being modified in the College as part of a separate research project on timber frame walls,

(iii) the Colorado State University program "WANELS" detailed by Castillo and Gutkowski (1984),

(iv) the Canadian Forest Products Laboratory program "SADT" detailed by Foschi (1977).

The first two options had to be eliminated as they were not available in a suitable form at the time of the investigation. The "WANELS" program, although highly praised by researchers in the field and dedicated to timber frame walls, had been written to model the ASTM test and had then only been checked using the small scale tests undertaken at Colorado by Patton Mallory (1983). It was unclear if the program could readily be adapted to vertical load modelling and if its capacity could be extended to cover the 4.8m plus wall lengths.

The suitability of the SADT program was discussed in person with its originator who was optimistic that it could be adapted to meet the requirements of the British test method. A copy of the program in its original form was found to be available in Britain through TRADA. Taking the decision to proceed with the computer based analysis using
the SADT program, a copy was transferred from the TRADA VAX computer to the University of Surrey Prime System.

A set of requirements was drawn up for the use of the program as part of the current investigation, they were as follows:

(i) that it could be run with satisfaction on the Prime System,

(ii) that it would model the British test procedure (i.e. it should take account of vertical load and should be able to predict both panel stiffness and panel strength, preferably in a single program run),

(iii) that it could be run to check the principal variables in timber frame wall design, namely materials (frame, sheathing and fixings), length, loading and openings,

(iv) that the input data should be obtained either from Codes of Practice, learned papers and manufacturers' information or by conducting a series of simple small scale tests.

It was clear that a lot of input data would need to be generated from tests, particularly those for fixings. It seemed likely that the program would need to be altered substantially to meet all these requirements. Consequently a schedule was set up to commission the SADT Program such that:

(i) the validity of the program and its reliability were checked using a recent series of test data which included internal panel movements such as nail slip and board rotations as detailed in Chapters 5 and 6,

(ii) the program was used to model a standard 2.4m long plywood panel under conditions of varying vertical load,
(iii) the variables used in the program were tested in turn to check their sensitivity and thus allow better use to be made of the input data together with a clearer understanding of the reliability of the results when widening the scope of the tests,

(iv) data files were established to model standard tests on plywood, mediumboard, BIIIB and plasterboard. The DOE tests undertaken in 1984 (see Appendix 2) were chosen for this comparative study between test and theory because all the materials used in the tests remained in the laboratory and could be used in small scale tests to determine input data,

(v) the test variables were extended to cover plain wall lengths from 0.6 to 6.0m and 2.4m window panels, once more allowing comparison with test results,

(vi) the program was enlarged to allow prediction of wall behaviour for typical house units.

Before the results of the analysis using the SADT program are detailed, it is of value to outline the method of analysis incorporated in SADT, the alterations carried out to the program and the commissioning procedure. Then in Section B of this chapter the small scale tests used to define the material parameters are described. Finally, in Section C, the results of the analytical tests are covered in detail, examining separately the factors affecting wall design and making direct comparison with test results and the empirical design theory outlined in Chapters 4 and 6. The analysis section concludes by discussing the benefits and limitations of the SADT program and suggesting areas for further work.

7.2 THE SADT COMPUTER PROGRAM

The 'SADT' computer program was developed by Dr. R.O. Foschi in the Western Forest Products Laboratory in Vancouver to cover the analysis of wood diaphragms and trusses.
Only the analysis concerned with diaphragms and more particularly wall diaphragms is considered here. The analysis incorporates the four basic structural elements which make up the timber frame wall, i.e. the cover, the frame, the connections between frame members and the cover/frame connection. The basis of the computer program has been described in detail by Foschi (1977). Some relevant aspects of his model are summarised here.

The cover, which consists of sheet material, is assumed to be an orthotropic linear elastic material in a state of plane stress. The principal axes of elastic symmetry (x', y') may lie anywhere in the plane of the cover. Cubic isoparametric rectangular finite elements (see Figure 7.1a) are used, with a single element being sufficient to model a simple rectangular sheet. The 'L' or 'U' shaped sheets around openings require a greater number of elements due to their increased complexity (Figure 7.1b). The elastic material properties required for a cover element are:

(i) the Young's moduli, $E_{x'y'}$, and $E_{y'x'}$,
(ii) the shear modulus, $G_{x'y'}$,
(iii) the Poisson's ratios, $\nu_{x'y'}$ and $\nu_{y'x'}$,
(iv) the angle $\psi$ between the elastic principal axis x' and the global axis x.

Different elastic properties may be specified for different elements.

The frame is modelled using elastic beam elements with three degrees of freedom, two translational and one rotational, at each end. Where frame members meet at a nailed joint a different node number is assigned to each side of the joint.

Both the cover-frame and the frame-frame connections are modelled using non linear elastic springs with the load-deflection relationship described by (see Figure 7.2):

$$F = (p_0 + p_1 \Delta) (1 - e^{-k\Delta/p_0})$$

- 7.1
where: $k$ is the initial spring stiffness,
$p_l$ is the stiffness at large deflections,
$p_o$ is the intercept on the load axis,
$F$ is the load in the spring and
$\Delta$ is the deflection of the spring.

The frame - frame connection is assumed to consist of three discrete springs resisting relative axial, shear and rotational displacements between the pair of frame joints. A different set of $(k, p_o, p_l)$ values is assigned to each of the three springs. Different sets of parameters may be assigned to different joints.

A cover-frame connector is modelled using equation 7.1, where $\Delta$ is the absolute magnitude of nail slip, which makes an angle $\alpha$ with the axis of the frame member, and $F$ is the force in the nail in the direction of slip. The parameters $(k^\alpha, p_o^\alpha, p_l^\alpha)$ are related to the two sets of parameters $(k^o, p_o^o, p_l^o)$ appropriate to nail slip along the axis of the frame and $(k^{90}, p_o^{90}, p_l^{90})$ appropriate to nail slip perpendicular to the axis of the frame member using an equation of the form:

$$k^\alpha = k^o, k^{90}/ (k^o \sin 2\alpha + k^{90} \cos 2\alpha)$$

The values of the parameters $(k^o, p_o^o, p_l^o)$ and $(k^{90}, p_o^{90}, p_l^{90})$ are obtained from laboratory tests. As the cover frame nails are distributed along the length of the frame member and the side of a cover element, the computer modelling involves numerical integration along the connected length.

Modelling the four structural elements of the timber frame wall in the manner described and applying appropriate displacement boundary conditions gives rise to a non linear set of stiffness equations of the form:

$$[K] \{\delta\} - \{W(\delta)\} = \{R\}$$

where $\{\delta\}$ is the vector of displacement,
$\{R\}$ is the vector of applied loads,
$[K]$ is the initial stiffness matrix at zero load and
$\{W(\delta)\}$ is a force vector containing the non linear terms associated with the connectors.
The equations are solved using incremental loading and initial stress approach* without any recomputation of the [K] matrix. The procedure is one of iteration, where:

$$\{S_{i+1}\} = [K]^{-1} \{R\} + [K]^{-1} \{W(S_i)\}$$  \hspace{1cm} 7.4

with the initial elastic assumption:

$$\{S_0\} = [K]^{-1} \{R\}$$  \hspace{1cm} 7.5

For each load increment iteration to reduce the residual force vector $\{W(S)\}$ is continued until the ratio of vector norms is:

$$\|S_{i+1} - S_i\|^2 / \|S_i\|^2 < \varepsilon$$  \hspace{1cm} 7.6

where $\varepsilon$ is a given tolerance, usually 0.001.


7.3 ALTERATIONS TO THE SADT PROGRAM

The "SADT" program was initially transferred on to a VAX computer which had a compiler similar to that of the TRADA PDP 11 system. Using the documentation for the program and a data file, both supplied by TRADA, the trial runs were successfully completed. Three main problems were manifest.

(i) The incremental loading procedure in the program caused all loads, both vertical and racking, to be increased by a single load factor. Simulation of a racking load test under constant vertical load was not possible in a single run of the program.

(ii) In modelling frame nail behaviour, the response of the joint to axial loading was described by a single set of parameters. In practice the response is very different in tension and compression.
(iii) The iteration procedure failed to converge before the load-deflection curve became sufficiently flat to suggest that failure was being approached.

Problem (i) could be overcome in principle by running the program separately for a series of increasing racking loads. This would require considerable computer time and would run into convergence difficulties at an early stage of loading due to the increasing step size of the racking load. The program was altered so that, for a constant vertical load, the racking load could be applied incrementally with iteration to convergence at each load step.

Referring to problem (ii), both the stiffness and strength of a nailed frame-frame joint is much smaller when the joint is in tension rather than in compression. The tension joint is characterised by the withdrawal resistance of the nails, in contrast to the compression joint where the bearing contact between the two members gives rise to much greater stiffness and strength. If all joints remained in either tension or compression throughout the analysis, then the joint parameters could be assigned accordingly. However for the common case of vertical loading of the studs, the frame joints between the bottom rail and the leading studs go from compression into tension as the racking load increases (Figure 7.3). To explicitly change the joint characteristics as the joints go into tension during loading would have necessitated extensive changes to the computer program and led to a far more tedious analytical procedure. It was decided to include the vertical stud loading as a prestressing force in the joint and thus to vary the joint parameters for different vertical loads. The theoretical changes necessary to the frame nail behaviour are shown in Figure 7.4. The additional vertical load effect is given an extremely high stiffness otherwise it will behave as if applied progressively as uplift increases instead of instantaneously. Thus a practical value of 100,000 N/mm was applied to $k''_u$. To avoid confusion between the applied vertical load and the load
resistance of the spring the following definitions are introduced.

(i) Vertical load - the load applied externally to the panel.

(ii) Nail withdrawal load - the maximum practical resistance offered by the frame nail against uplift of the studs.

(iii) Uplift load - the withdrawal load assigned to the bottom rail joints in the computer model to include both (i) and (ii) above.

Clearly this alteration applies only to tension joints, joints in compression will automatically transfer the vertical load without additional effect and consequently vertical load can be eliminated from the computer model. Care is then needed in dealing with openings and some suggestions are given in Figure 7.5.

The convergence difficulties, noted as problem (iii) above, were found to be largely associated with the cover nail performance which was the primary source of the non linear panel behaviour. The problem became more serious when it was decided, following a programme of laboratory tests discussed in Section 7.4.2 to assign a very high initial stiffness to the cover nails. As the initial stiffness formed the basis of the iterative procedure, convergence became increasingly difficult to achieve as the racking load increased and the tangential stiffness of the cover nail joint dropped dramatically (see Figures 7.6 and 7.7). As a compromise between the initial high and later low joint stiffness, a new parameter was introduced as a numerical device into the computer program. This stiffness was chosen so as to be just high enough to enable convergence during the initial loading stages and low enough to improve the convergence of the iterative procedure at higher racking loads. This new trial stiffness became the fourth parameter in the definition of the cover nail. The most appropriate value for the parameter was
determined by a trial and error process. It had been noted in full scale tests that nail movements were very much higher in the bottom rail than in either the studs or the top rail. As the programme allowed more than one type of cover nail to be entered for a panel, the opportunity was taken to enter the bottom rail nails separately. They are identical in performance parameters but in general are given a lower trial stiffness so that the larger deflections may be more easily monitored.

Incorporated with these changes was a fifth parameter to represent the failure behaviour of the boards where nail movements towards the edge of a board are large. The parameter entitled "bang" in the program limits the deflection allowed in a cover nail joint. When the value of "bang" is exceeded the capacity of the nail becomes zero representing a very brittle type of board failure. In practice the "bang" condition only affects the 'perpendicular to grain' loading of the nails in the bottom rails of the panels where, due to tension in the frame joints, relative movement is high, particularly at zero vertical load. Board failure is thus identical to practice and the sudden loss of load in a nail does not cause a similar failure in the panel if its load can be redistributed. Careful selection of the value for "bang" means it can be used to balance the practical nail performance as shown in Figure 7.8.

The final changes made to the program were required to simplify its use and extend the scope of the wall size that could be analysed. In its original form "SADT" had a limit on the number of equations which could be solved; this meant that a 3.6m plain panel was the largest that could be analysed. Thus the program had to be modified to cover longer plain panels and the smaller window panels, which are very much more complex in their framework and sheathing, requiring a substantial increase in nodes and joints and consequently, in the number of equations generated to solve the analysis. Thus the storage in the program was required to be very large which made the solving of short plain walls very inefficient. The solution was to set the storage
variable for the calculations using a second date file which could be edited dependent on panel type. While these alterations were made, the program was edited to remove all unnecessary subroutines, so that in its final version it was only capable of analysing timber frame walls. The editing also enabled the program to be read by a wider range of Fortran compilers. The program and typical data files are available, through the author, at the Department of Civil Engineering of the University of Surrey.
\[ X = X_c + \varepsilon a \]
\[ Y = Y_c + n b \]

(a) Isotropic element representing a complete sheathing board

(b) 'L' shaped sheet often used at window openings showing the use of three elements and numbered as appropriate to the Foschi mesh generation system

Figure 7.1 Cover Finite Elements (after Foschi (1977))
(a) The spring model

(b) Typical connector response

Figure 7.2 Frame Member Connectors (after Foschi (1977))
Non zero vertical load case includes effects of vertical load, i.e. resistance to uplift plus resistance to nail withdrawal.

Zero vertical load case includes the effect of resistance to withdrawal only.

(1) Theoretical curve

(2) Initial trial solution where

\[ p'_{ou} = p_{ou} + V \]

\[ p'_{iu} = p_{iu} \]

\[ k'_{u} = k_{u} \times (p'_{ou}/p_{ou}) \]

(3) Practical solution where

\[ k'_{u} = 100 \, 000 \, \text{N/mm} \] to ensure instantaneous effect of vertical load

NB: in practice \( p_{iu} = p'_{iu} = 0 \)

Figure 7.4 Incorporation of Vertical Load in the Resistance of the Frame Nail to Withdrawal
Vertical load = \( V \) on each stud

Nail withdrawal load = \( W \) at each joint

Uplift load = \( U_n \) where:
\[
U_1 = V + W
\]
\[
U_2 = \frac{3}{2} V + W
\]

Compression Load = \( C \)

(maximum value independent of vertical load)

Figure 7.5 Vertical Loads Over Panel Openings
(a) Foschi nail curve shown for tests on the early stiffness of a structure

(b) Foschi nail curve in practice when testing a structure to failure with accurate initial stiffness

(c) Balanced guess for Foschi nail curve

Figure 7.6 - Modelling Problems When Using The Foschi Nail Equation
(a) Finding the deflection for a given load step by overcutting when the trial stiffness is greater than the panel stiffness

(b) Finding the deflection for a given load step by undercutting when the trial stiffness is less than the panel stiffness

Figure 7.7 The Iterative Process Used to Find the Deflection for a Given Load Step
'Bang' parameter located to balance failure performance.

Practical nail performance showing drop off in load that cannot be monitored by the Foschi curve and is not accurately modelled by the Kimber curve.

Figure 7.8 The Effect of the 'Bang' Parameter on a Typical Nail Curve
SECTION B

SMALL SCALE NAIL TESTS AND TRIAL RUNS

7.4 NAIL TESTS

7.4.1 Introduction

The "SADT" program required nail data for five separate cases:

(i) deflection of the cover nail parallel to the grain of the frame material,
(ii) deflection of the cover nail perpendicular to the grain of the frame material,
(iii) withdrawal of the frame nail from the studs,
(iv) shear of the frame nail in the line of the top and bottom panel rails,
(v) rotation of the frame nail in the plane of the panel due to the lozenging action of the frame.

Full scale testing experience indicated that the behaviour of the cover nails was paramount in assessing panel performance. On its own the frame of a wall has very little stiffness and strength as the joints are primarily subjected to a rotational force. However in a clad panel it is likely that the resistance of the frame nails to withdrawal will contribute to the overall performance, particularly at low vertical loads.

Nail values were required for the four sheathing types most commonly used in this investigation, viz plywood, mediumboard bitumen impregnated insulation board and plasterboard. As the majority of the published work on joints between board and solid timber members covered only plywood, it was decided to carry out an independent programme of nail tests. The conditions set down for these tests were that they should be:
(i) simple to set up, using equipment readily available in the laboratory,
(ii) quick to perform, thus allowing a higher number of repetitions,
(iii) reasonably accurate, as judged by repeatability.

With regard to the third condition, it must be noted that these tests were to provide trial data for the computer program and not definitive performance figures. Although the sheathing types could not be graded for quality, it was important that attention was given to stud quality. Each stud type came from a single grade of the material and care was taken in manufacturing the samples to avoid knots when driving the nails. The most convenient method, therefore, of classifying the studs was by density. Samples for tests were divided into three roughly equal groups by weight and at least two samples from each group allocated to each specific test. In addition, the ring spacing and orientation during testing for each specimen was recorded in an attempt to relate nail performance to stud density and the cross-sectional appearance of the stud. An investigation of the early test results showed that no definite relationships existed and no trends were established. Thus as the information was of secondary importance to the objectives of the tests, the additional details were noted, briefly checked against performance and then stored for future use.

7.4.2 Cover Nail Tests

The test setup for the "parallel to grain" and "perpendicular to grain" tests are shown in Figures 7.9 and 7.10 respectively. In both cases the test specimens were loaded to failure at a deflection rate of 4mm/minute. Applied load was plotted against cross head deflection on a standard X-Y chart plotter and after testing the specimens were examined and the modes of failure noted. When all similar tests had been completed, the chart plots were overlain and traced onto a single plot. At this stage care was taken to remove the initial slip in the structure on take up of load. The result showed a marked variability which could not be linked to
stud density or any obvious single test factor. The curves were examined and a mean line drawn through coordinates interpolated at 1 mm intervals with an additional point at 0.5 mm. Typical sets of results are shown in Figure 7.11. Mean values were used throughout the computer modelling exercise but further work could be undertaken on lower and upper bound curves in an attempt to explain variations in full scale wall performance.

Throughout the tests the cover nails used were 63 mm long 3.25 mm diameter electro galvernised wire nails, identical to those used in the later full scale tests. In practice these nails are gun driven but for the small scale tests they were often hand driven, when accuracy in position was important. A few tests were carried out for comparison purposes on the other nails commonly used in the current investigation, viz 50 mm long 3.25 mm diameter hot dip galvanised clout nails and 50 mm long 2.87 mm diameter wire nails. Although these results have not been used in the computer analysis, the averaged performances have been compared with the standard nail for the plywood sheathing with SPF frame tests. All the plasterboard specimens were nailed with the standard 40 mm long 2.6 mm diameter hot dip galvanised plasterboard nails.

The parametric study using the computer analysis, carried out in parallel with the nail tests revealed two requirements for the cover nail models:

(i) a high "parallel to the grain" performance in comparison with that perpendicular to the grain,

(ii) a very high initial stiffness.

As a result two further nail tests were carried out to check if, by oversimplification, a function of the nail performance in either type of test had been omitted. The additional tests covered two points, the combined action of nails when working in rows and load cycling. The test specimens are shown in Figure 7.12; the compression model included two columns of three nails at 75 mm centres and the tension model
had one row of three nails also at 75mm centres. The nail spacing had to be reduced to keep the size and proportion of the test specimen within reasonable limits. It was considered that the reduced spacing might compensate for the loss of continuity in testing only three nails in a line. However in the test only the central nail is protected from the "parallel to nail" forces set up in the test which tend to peel the board away from the stud and can be responsible for premature failure of the joint. The reduced spacing had no effect on BIIB which was used with a standard spacing of 75mm but it was considered that the sheathing was comparatively so weak that the parallel to nail force set up in these tests would be negligible.

Load cycling was covered by the test procedure used on these specimens whereby they were loaded first to 5kN (2.5 kN in the tension test where half the number of nails were employed) then the load was removed and finally the load was reapplied and the test continued to failure of the joint. The test results were plotted and reduced as before and could then be compared with the single nail tests. A typical result of the load cycling is shown in Figure 7.13. The initial stiffness in the two cycles was similar and the second part of the curve was the same in both cases. However, the set between the two cycles, typically 2mm for both types of test, meant that the later performance of the second cycle test would appear stiffer due to the change of datum. Results for the two load cycles are included in Figure 7.14. For later curve fitting it was considered that the first cycle result should be dominant but that use could be made of the area bounded by the two cycles.

Comparison with the single nail tests, also shown in Figure 7.14, revealed that the "perpendicular to grain" resistance was not affected by the number of nails, i.e., they all acted independently and had no effect on the fixing either side. The "parallel to grain" performance of each nail was enhanced in the column of nails test. Previously the single nail performance had been similar to that of the nail
perpendicular to the grain but the three nail tests gave improvements of 12% - 15% throughout the comparison with the single nail test.

It was not possible at this stage to repeat all the tests using the multiple nail joint and a decision was taken to use the results of the single nail "perpendicular to grain" tests and to enhance the results of the "parallel to grain" tests by 12%, to take account of the effect of nail joint continuity, in preparation for the curve modelling exercise.

The conclusions drawn from the nail test work were that:

(i) the decision to use a very simple test had been correct as a great deal of reliable information had been very quickly generated,

(ii) the single nail test was suitable for "perpendicular to grain" resistances,

(iii) "parallel to grain" resistance should be measured using the more elaborate three nail column tests,

(iv) the results allowed the behaviour of the nail to be modelled to failure but in view of the importance of the initial stiffness any refinements to the test should concentrate on results within the first 1mm of deflection.

7.4.3 Modelling the Cover Nail Results

To model a panel test up to maximum load, it is necessary for at least one part of the panel structure to fail. The areas where damage is commonly noted are:

(i) in the cover nails where very large slips indicate the maximum resistance of the nail to have been exceeded,

(ii) in the breaking of the sheathing around the cover nails due to large movements related to short edge distances in the
direction of nail movement (e.g. around the nails connecting the cover to the bottom rail of the frame under the leading stud); this failure indicates the nail load immediately reduces to zero.

Other failures, notably in the bottom rail or in the withdrawal of the frame nails, can usually be associated with failure of the cover nails and either occur after maximum cover nail resistance has been reached or are insignificant in the overall performance of the panel. To achieve failure of any of the nails the $p_1$ value in the Foschi curve (equation 7.1) has to be zero. Early experience of the nail tests indicated that it was also necessary for the initial nail stiffness to be comparatively high. Neither of these requirements suited the Foschi curve to the modelling of the nail behaviour (see Figure 7.6) because increasing the difference between initial and final stiffness had the effect of making the curve between the asymptotes more sharp and pronounced. Thus the initial stiffness and failure could be modelled but the mid range loads required to give the 5mm and 7.2mm deflection loads were excessively high. A number of attempts to find an approximation to cover both high stiffness and a failure load based on the nail results all failed and it was obvious that the Foschi equation was unsuitable for modelling wall performance from very low loads through to failure.

An alternative three parameter curve was required for direct substitution in the SADT program. Two curves were suggested:

\[
(\text{i}) \quad y = \frac{ax}{(1 + bx)^c} - 7.7 \\
(\text{ii}) \quad y = axe^{-bx^c} - 7.8
\]

Preliminary tests showed the second to be more promising and, for simplicity in referencing, it has been named the "Kimber" curve after Dr. A.C. Kimber who suggested its use. The
important characteristics of the curve are:

(i) \( y_{\text{max}} \) occurs at \( x = \left( \frac{1}{bc} \right)^{1/c} \)

(ii) \( b = \frac{1}{c}(x_{y_{\text{max}}})^{-c} \)

(iii) the initial slope is 'a' where
\[
a = \frac{y_{\text{max}} e^{1/c}}{x_{y_{\text{max}}}}
\]

Noting that 'x_{y_{\text{max}}}' is the value of 'x' relating to the maximum value of \( y \). Thus if 'x_{y_{\text{max}}}' and 'y_{\text{max}}' are fixed then for a given value of 'c', 'a' and 'b' will also be fixed. The significance of these results is shown in Figure 7.15.

In part a, by plotting load as a proportion of maximum load and keeping 'c' constant, the effect of changing the 'x' value at which 'y_{\text{max}}' occurs can be seen. Thus the curve can be flattened by extending the failure deflection. The effect of 'c' is shown in part b by keeping the failure deflection constant. It can be seen that reducing 'c' steepens the curve indicating a higher nail stiffness and increasing 'c' correspondingly flattens the curve. All the curves are coincident at the origin and at \( (x_{y_{\text{max}}}, y_{\text{max}}) \).

In practice curve fitting is easy. The maximum load \( (y_{\text{max}}) \) is taken direct from the test data (including the 12% enhancement where necessary) and a good estimate can be made of 'x_{y_{\text{max}}}'. A guess is made of 'c' and the curve plotted from which a better value for 'c' can be determined. Experience showed that values of 'c' close to 0.25 were suitable for stronger board/nail joints whilst for weaker boards 'c' needed to be increased up to a value of 0.5. Clearly the flatness of the curve at maximum load allows some tolerance in the choice of 'x_{y_{\text{max}}}' which can be adjusted with 'c' to suit the results. The nail models are compared with the interpolated test results in Figures 7.16 to 7.19 which also show the maximum deflections allowed before total failure of the joint due to breakdown of the board (the 'bang' term introduced into the SADT program.). Experience has shown that the maximum deflection limit should really be applied only to the "perpendicular to grain" tests to cover failures along the bottom rail of the panel but its inclusion
in "parallel to grain" tests does not affect the computer analysis.

7.4.4 Double Sheathings

At present the 'SADT' program cannot deal directly with panels including two structural sheathings (i.e. external walls with a structural plasterboard lining or internal partition walls with two sheets of plasterboard). This problem can be overcome in part by using cover nail data based on small scale tests modelling the complete proposed wall system. To achieve such results the "parallel to grain" test remained as before except that one of the boards nailed to the stud was replaced by the second sheathing material. The design figure is then the test result divided by the number of nails used in each row. The tension test is slightly modified with the second sheathing included opposite the primary board and the two boards pulled together.

The results of these tests confirmed the full scale findings in that the combination results could not be related to the individual board performances. However, two problems were revealed.

(i) The performance for internal walls using the same sheathing was double that of the single board whereas racking tests had shown this was not the case. Fortunately only the cover nail performance is doubled and thus in the analysis the frame performance is the same for both cases and thus the full scale tests could still be correctly modelled.

(ii) The simple test method could not cover two sheathings with different spacing of fixings, e.g. BIIB and plasterboard external walls. To overcome this problem, for the simple case when the spacing is halved, double the nails are used for the weaker sheathing. The nail parameters are based
directly on test performance but
the nail spacing input to the analysis is
that of the stronger material, i.e.
150/300 mm centres.

7.4.5 Frame Nail Tests

Three tests were set up to measure withdrawal, shear
and rotational resistance of the frame nails. The withdrawal
tests were sub-divided to check differences between end stud
and centre stud behaviour. Details of the test rigs are
shown in Figures 7.20 to 7.22. The load in the withdrawal
and shear tests was applied hydraulically to achieve a
deflection rate of 4mm/min in both cases. The rotational
resistance test specimens were loaded with dead weights
due to the very low loads involved. In all cases deflections
were measured using LVDTs and compared with load on an X-Y
plotter. The curves were traced on to a single graph for
each type of test from which the mean performance curve
could be interpolated.

The nails used in the standard tests for use in the
computer analysis were 80mm long 3.25mm diameter electro
galvanised wire nails as used in recent full scale tests.
Additionally tests in withdrawal were carried out on 100mm
long 4.0mm diameter hot dip galvanised nails and 80mm long
3.25mm diameter hot dip galvanised ring nails. Typical
results for the three tests are shown in Figures 7.23 - 7.25.
The Foschi curve was found to be suitable for analysing the
results in each case which had the advantage that no changes
were necessary to the "SADT" program. The shear and
rotation equations were readily interpreted from the test
data based on a single nail performance. The design values
used in the "SADT" data file then included a multiplication
factor for the number of nails in each joint. No further
shear or rotation tests were carried out on other types of
frame nail as the parametric study carried out concurrently
(see Section 7.5) showed the information to be immaterial to
panel performance.
The results of the withdrawal tests were more difficult to interpret and the parametric study showed the nail withdrawal load to significantly affect panel performance. The problems were increased by the need to include the vertical load behaviour within the equation for nail withdrawal.

The test results showed that for a two nail joint the maximum load in withdrawal for the standard 3.25mm diameter 80mm long wire nail was 1.4 kN when averaged over five tests. Similar size ring nails showed a 25% increase in load and the use of 4.1mm diameter 100mm long hot dip galvanised nails (i.e. with a rough finish) doubled the withdrawal load. The end stud tests were more suited to the needs of the program because in practice end stud uplift is critical; they showed a 15% reduction in performance compared with the identical nail tested in a central position. The joint loads were therefore significant when compared with the applied vertical load.

These results did not agree with findings from full scale tests where the following general points had been observed as a result of many years test experience:

(i) Vertical load has a major effect on performance. A 5 kN/stud load on a standard panel increases performance by over 75% compared with the zero vertical load case.

(ii) Changing either the type or quantity of the frame nails has very little effect on panel performance.

(iii) Frames tested without sheathing have essentially no racking strength at any vertical load.

Points (ii) and (iii) suggest that the loads carried through the frame nails in joining the stud to the bottom rail are insignificant in comparison with those transferred by the gusset action of the sheathing. Therefore they should not
be represented by a load equivalent to a vertical load of 1.0 kN/stud as suggested by the tests. This finding is confirmed by Code of Practice guidelines on nail resistance (BS5268 Part 2, 1984) where no load is allowed for withdrawal from end grain. It is probable that the tests, by isolating the withdrawal load, exaggerate its effect whereas the secondary effects of shear and rotation would reduce resistance to withdrawal.

It is concluded that the nail withdrawal test is of very limited value in determining the uplift load and stiffness required by the analysis. The hypothetical effect of the vertical load, introduced in Section 7.3 to simplify the method of analysis, is of paramount importance to these values and therefore the inclusion for nail withdrawal will have to be based on the parametric study for vertical load.

7.5 TRIAL RUNS AND THE PARAMETRIC STUDY OF MATERIAL VARIABLES

7.5.1 Initial Trials: The Effect of Uplift Load

Prior to the main parametric study, a number of analyses had to be run to determine if the program in its amended form was suitable for modelling failure tests. The program was used to check racking load versus deflection and uplift of the leading stud versus deflection (i.e. the principal readings) on a standard 2.4m long plain panel. Sheathing and frame parameters were taken from BS5268 Part 2 (BSI, 1984) and nail parameters were based on meaned values from the early test results (Section 7.4). The principal objective of the tests was to check vertical load behaviour because of the hypothetical treatment of vertical load by the analysis. The results showed that:

(i) a sensible maximum load could be achieved (defined by the load/deflection curve smoothly approaching a zero stiffness) if care was taken in estimating the mathematical parameters required to run the program, such as the trial stiffnesses,
(ii) as uplift load increased the improvement in panel resistance diminished (Figure 7.26a),

(iii) the vertical load effect underestimated that measured in full scale tests (Figure 7.26a),

(iv) alteration to cover nail parameters could enable accurate modelling of zero vertical load performance but the 5 kN/stud performance within the stiffness test limit (i.e. a deflection of 7.2mm) was always low,

(v) uplift deflections were low in comparison with full scale results particularly at higher vertical loads and at racking deflections less than the 20mm.

These results indicated that the vertical load hypothesis was not perfect, and for best results required the nail withdrawal load to be a minimum. A withdrawal load of 0.2 kN was chosen as a practical minimum. This value represents a single nail withdrawal load of 0.1 kN, a standard two nail joint, and results in an uplift load of 5.2 kN/stud to model a 5 kN/stud vertical load.

The discrepancy in uplift was inevitable because vertical load has been assumed to be transferred directly through the stud to the bottom rail. The experimental investigation into panel behaviour has shown (Chapter 6, Section D) that the high practical uplifts and low nail slips in the leading stud can only be explained if the vertical load is assumed to be transferred, in part, by the top rail and the sheathing to the bottom rail. It is therefore necessary to assume that even if the analysis accurately predicts the overall behaviour of a wall, in terms of racking load and horizontal deflection, it will be unable to model the internal behaviour of the panel noted in the board rotation and nail slip tests. Thus little use can be made
of uplift deflection data and it has been considered unnecessary to output and analyse the internal stresses and strains in the panel members although these results are easily accessed from the program.

7.5.2 Parametric Study of Materials

The parametric study of the materials is included for the following reasons:

(i) to check the influence of the parameters on the analysis and determine where possible if the same degree of influence could be expected in practice,

(ii) to determine the most suitable values for the material factors related to their effect on the 2.4m long panel results; in this way the suitability of the nail test values and standard material data could be checked.

The study was divided into the same four areas used by the analysis to cover the structural components of the timber frame wall, i.e. the frame, the frame-frame connections, the sheathing and the sheathing-frame connections. The material parameters have been varied separately based on a standard 2.4m long plain panel made up of 9.5mm plywood fixed to a 90 x 40mm spruce/pine/fir framework with 63mm long 3.25mm diameter gunned wire nails.

The following observations were made, relating to changes in frame properties:

(i) altering the orientation of the studs, from a width by depth of 40mm x 90mm to 90mm x 40mm had no effect on performance,

(ii) reducing the modulus of elasticity of the wood in the frame by 35% caused less than a 5% reduction in performance.

The frame parameters were, therefore, considered to have
little effect on model performance. Consequently the nominal frame size of 40mm x 90mm was applied to all models and the mean modulus of elasticity for GS grade SPF (8500 N/mm²) was used to cover all the frame materials modelled in the analysis.

The trial runs relating to changes in frame nail parameters indicated that:

(i) increasing the uplift load in the bottom rail connections improved panel performance, however the greater the initial uplift load the less significant the change, particularly in terms of initial stiffness,

(ii) increasing the initial stiffness of the same frame nail in its resistance to withdrawal improved panel stiffness and showed greater significance at higher vertical loads,

(iii) reduction in the rotational resistance of the frame nails to zero had no effect on panel behaviour,

(iv) increasing the shear resistance of the nails by ± 20% representing the upper and lower bound performances of the nails shown in Figure 7.24, had no effect on panel performance.

As a result of (iii) and (iv) above the test values recorded in Section 7.4.5 were used to represent all frame nail connections in both tension and compression. It is clear from both the analytical and the full scale test results that these parameters are not important to the overall panel performance.

The effect of uplift load noted in (i) above has already been analysed resulting in the low nail withdrawal load of 0.2 kN being attributed to tension joints. The 0.2 kN value is required to run the program satisfactorily when modelling zero vertical load and is used for all types of
frame-frame connector. It is substantially lower than the value predicted by the tests but compares with a zero allowance permitted by the Code (BS5268 Part 2, 1984).

The effect of reducing the initial stiffness of the uplift load was considered in Section 7.3. The uplift load acts as a prestressing force to model the vertical load. A very high stiffness is required to model an immediate take up of the prestress load otherwise the analysis models a panel in which the vertical load is increased with the application of racking load. The stiffness of 100,000 N/mm has been used for all frame-frame connectors in tension. In compression joints the equation for movement parallel to the nail (i.e. withdrawal when in tension) has been given the maximum possible strength and stiffness. This assumes that there will be no compression of the bottom rail. In practice this is seen often not to be the case, however, the small movements usually occur after maximum racking load has first been approached and have little effect on panel behaviour. Therefore this movement need not be modelled.

Varying the sheathing parameters during trial runs indicated that:

(i) increasing the modulus of rigidity increased the initial stiffness of the panel particularly at high vertical loads,

(ii) changing the elasticity of the board had insignificant effect,

(iii) variations in sheathing parameters were secondary to variations in cover nail parameters.

Defining the sheathing parameters for boards other than plywood proved difficult as in-plane moduli are rarely quoted by manufacturers or researchers. The plywood moduli were based on the BS5268 Part 2 values for the material. These figures were enhanced by 50% in an attempt to achieve a better interpretation of the likely mean performance of the
material in a short term test. Hence the moduli used for the plywood were:

(i) \( E_{\parallel} = 8650 \text{ N/mm}^2 \)
(ii) \( E_{\perp} = 4600 \text{ N/mm}^2 \)
(iii) \( G = 650 \text{ N/mm}^2 \)

For fibre building boards, Lundgren (1969) suggests the relationship \( G = 0.4E_{\text{ten}} \) but in quoting design figures he increases the factor to 0.5. His design value for the modulus of elasticity of medium hardboard (similar to the medium density fibreboard used in this investigation) was 900 N/mm\(^2\) based on a short term test value of 3000 N/mm\(^2\) (reducing slightly with increased moisture content). His design value for modulus of rigidity was given as 450 N/mm\(^2\).

Tensile tests were carried out at Surrey on the board used in the test panels and a stiffness of between 2,500 and 3,000 N/mm\(^2\) was measured. Hence Lundgren's values could be used but they were factored by 1.5, as done with the plywood, to make them more applicable to short term wind loading tests in comparison with their use as long term design values. Thus the values used for the mediumboard in the computer analysis were:

(i) \( E_{\parallel} \) or \( E_{\perp} = 1350 \text{ N/mm}^2 \)
(ii) \( G = 650 \text{ N/mm}^2 \)

The modulus of fibreboard is very much more dependent on load duration than that of plywood. As racking loads are short term it is possible that the mediumboard moduli could be further increased.

Manufacturers of insulation boards quote figures for the modulus of elasticity of the board in bending of 500 ± 20 N/mm\(^2\) lengthwise and 400 ± 30 N/mm\(^2\) crosswise. Lundgren (1969) suggests that the tensile modulus will lie between 80 and 90% of the bending modulus, depending on the variation through thickness of board density. Thus tensile modulus values for the test boards of 300 N/mm, as determined at Surrey, were in reasonable agreement. No design values were quoted for BIIB and thus it was decided to use the test values direct although they are probably too high for design purposes. Thus the
moduli used in the computer analysis for BIIB were:

(i) $E_{||} \text{ and } E_{\perp} = 300 \text{ N/mm}^2$

(ii) $G = 150 \text{ N/mm}^2$

No information could be obtained for plasterboard and test results at Surrey were not conclusive. Thus, noting the lack of significance attached to the modulus of rigidity, the figures used during the computer runs were:

(i) $E = 5000 \text{ N/mm}^2$

(ii) $G = 1000 \text{ N/mm}^2$

The $G$ value was chosen to give a higher initial stiffness than that of plywood or MDF in the knowledge that the failure would be governed by the cover nail parameters. Where two different sheathings either side of the frame were being modelled, a modular ratio approach based on the modulus of rigidity of the primary sheathing has been adopted to determine an effective board thickness for the combination. Thus all the board parameters can be based on the primary sheathing.

In conclusion, it is thought that the moduli of elasticity and shear for all boards, with the exception of BIIB and plasterboard, have been too low in value. In view of the importance of rigidity in improving the stiffness of panels at high vertical loads, it is recommended that more attention be paid to their value if further use is made of "SADT" to analyse timber frame walls.

The cover nail parameters were defined by their effect on the nail slip model rather than by the 'a', 'b' or 'c' values of their Kimber curve. Thus initial stiffness and failure load for the nail were varied for both the parallel and perpendicular to the grain loading case. The results showed that:

(i) stiffness parallel to the grain had little effect at zero vertical load but was significant at 5 kN/stud,

(ii) stiffness perpendicular to the grain exhibited a contrary behaviour with much greater effect at zero vertical load,
(iii) increasing the failure load parallel to the grain had little effect on the zero vertical load plot but improved the 5 kN/stud performance when the deflection was greater than 15mm,

(iv) increasing the failure load perpendicular to the grain increased failure performance at zero vertical load and, to a lesser extent, at 5 kN/stud load,

(v) variations in the value of 'bang' the deflection at which nail load becomes zero, simulating failure of the sheathing, had much less effect on the strong boards than on the weak BIIB and the brittle plasterboard. This is because the latter boards reach maximum load at a lower deflection and consequently have a lower value for 'bang',

(vi) the value of 'bang' is of greatest significance at zero vertical load when uplift deflections are higher for a given racking deflection.

Taken together, the cover nail results indicated that the best overall model would be obtained by using a high parallel stiffness in combination with a low perpendicular stiffness; this would achieve maximum separation between vertical loads, and an average failure condition. However, if failure in the computer analysis does not match test performance then firstly the performance at zero vertical load should be adjusted by means of the perpendicular nail failure value and then the highest vertical load case may be altered using the parallel nail failure value. In general the 'bang' value related well to the deflection at failure of the perpendicular nail test. Lowering the value slightly due to the low uplifts recorded in the computer runs had the beneficial effect of reducing the deflection at failure without seriously affecting the load, but was responsible for a more sudden type of failure. A better solution would be achieved if the nail resistance could be
reduced in steps rather than immediately to zero, but as the program did not allow modelling past the point of maximum load this refinement was considered unnecessary.

7.5.3 Final Trials: The Problem of Load Stepping

A fourth series of trials analysed standard panel tests on the four sheathing types to check the ability of the program to model initial stiffness, failure load, failure deflection and uplift. The BIIB test included a different nail pattern with the spacing reduced to 75/150mm. A further variation was included by analysing a 1.2m long plywood panel; thus all the major materials variables were covered in this series of trials.

No difficulties were experienced in the handling of the data variables. The results gave rise to optimism that the computer analysis would provide a reasonable model of wall behaviour. The results for plywood sheathed 2.4m long panels were typical and are shown in Figure 7.27. The modelled behaviour can be seen to be sandwiched between that of high and low test values based on results recorded in Table 6.1a. It is noticeable that the analysis predicted a relatively strong behaviour at zero vertical load, particularly in initial stiffness, but a more average response at 5 kN/stud. The results, in general, emphasised the need to improve the differentials between vertical load performance.

A further problem was noted in these tests. The early computer runs had all been carried out with a 0.1 kN load step. Then, because there was no plotting procedure incorporated in the program, the output was stripped and the results plotted at 1.0 kN intervals until failure was approached when all results were noted. Close to failure a pattern known as "load stepping" had been noted. Here a large deflection from a single load increment was followed by very small increments for the successive load increments (Figure 7.28). It had been thought that this phenomenon was due to instability in the program at failure, noting that the program is based on load increments and cannot model loss in load which happens at or close to failure. During these tests load
stepping was noted at much lower loads, long before failure was reached; this had remained hidden in previous trials due to the output stripping procedure. The discovery cast doubt on the stability, and thus suitability, of the program.

The results of failure plots in the earlier trials indicated that the load stepping problem was associated with the trial stiffness incorporated to enable the program to run. By varying only this value in a series of runs at different vertical load conditions and by plotting the results for 0.1 kN load increments, it became apparent that there was an unique curve for each set of panel parameters. Variation in trial stiffness altered the position of load stepping; if this occurred at low loads compared with failure the steps were balanced about the unique curve (Figure 7.28). Only at failure were major discrepancies noted when either the panel failed prematurely or, following a large deflection for a single load step, the deflection then increased in single units for each load step. In the latter case failure could, however, be estimated at the point of the last significant (but less than 5mm) deflection jump. These tests proved that the trial stiffness should be kept as low as possible to model failure more accurately, however, the stiffness must also be sufficiently high to trace the initial deflection of the panel (Figure 7.7). With care it was then possible to proceed with the main series of trials. Later another series of test runs was carried out to investigate further the effect of trial stiffness; the results are reported separately in Section 7.9.

The parametric study was used to determine the final values for the materials parameters for the main series of tests. The comments regarding the cover nails led to the second series of nail tests investigating the combined action of nails in rows (Section 6.4.2) which resulted in parallel performance being increased by 12%. This approach infers that parameters were determined by back analysis of the full scale test results. In part, this is true and was necessary to overcome the slight inaccuracies both in the measurement of the parameters and the modelling method which
resulted from time limitations. However, the accuracy of the method of analysis was checked in the main program of trials when the complexity of the model was varied substantially to cope with plain walls varying in length between 0.6 and 6.0m, and with window panels. The suitability of the "SADT" program could then be judged by its ability to model the wall design modification factors tested in Chapter 6.
Tensile load applied through machine jaws gripping sheathing

Solid timber rail clamped to base of test bed

Figure 7.9: Details of Cover Nail "Perpendicular to Grain" Tests

Compressive load applied through crosshead

Minimum edge distance consistent with joining two boards on one stud

Crosshead deflection measured

Fixed base

Grooved base to prevent splaying of boards

Side elevation       End elevation

Figure 7.10 Details of Cover Nail "Parallel to Grain" Tests
Figure 7.11 Sheathing Nail Test Results for Single Nails
(a) Six nail "parallel to grain" test investigating load cycling and continuity of nail line

(b) Three nail "perpendicular to grain" test investigating load cycling and continuity of nail line

Figure 7.12 Cover Nail Test Specimens for Investigating the Effect of Lines of Nails
Figure 7.13 The Effect of Load Cycling On Nail Tests
Figure 7.14 Comparison of the Single Nail and Line of Nails Test Results for Plywood on Red/White Wood Studs
(i) There is a unique curve between zero and maximum load for a given value of 'c'.

(ii) This curve can be moved up and down by respectively decreasing or increasing the value of 'c'.

Figure 7.15 Properties of the 'Kimber' Curves
Figure 7.16 Nail Test Results For 9.5mm Plywood on SPF Timber

Figure 7.17 Nail Test Results For 9.0mm Mediumboard on SPF Timber
Kimber Curve Parameters

\[ y = ax^b \times c^x \]

**parallel**
- \( a = 500 \)
- \( b = 0.82 \)
- \( c = 0.5 \)

**perpendicular**
- \( a = 430 \)
- \( b = 0.82 \)
- \( c = 0.5 \)

Value of bang = 7mm

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**Figure 7.18** Nail Test Results For 12.5mm B11B on SPF Timber

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Kimber Curve Parameters

\[ y = ax^b \times c^x \]

**parallel**
- \( a = 1890 \)
- \( b = 1.79 \)
- \( c = 0.3 \)

**perpendicular**
- \( a = 570 \)
- \( b = 0.89 \)
- \( c = 0.5 \)

Value of bang = 6.0mm

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**Figure 7.19** Nail Test Results For 12.5mm Plasterboard on SPF Timber
(a) Standard test

(b) Simulated end stud condition

Figure 7.20 Details of Tension/Withdrawal Test on Frame Nails

Figure 7.21 Details of Shear Test on Frame Nails

Figure 7.22 Details of Moment Rotation Test on Frame Nails
Figure 7.23 (top left) Results of the Tension/Withdrawal Tests on Frame Nails

Figure 7.24 (bottom left) Results of the Shear Tests on Frame Nails

Figure 7.25 (top right) Results of the Moment Rotation Tests on Frame Nails
Note poor modelling of initial stiffness

Test on sheathing as shown below
Test on sheathing with higher initial stiffness due to increase nail resistance
Analysis including improved values for sheathing nail

Good modelling at zero vertical load

Key

Test result
Analysis

Greater effect of change in vertical load at lower values of vertical load

Family of curves can be reduced by reduction in nail parameters

Figure 7.26 Results of Initial Trial Runs of the Analysis Compared With Test Data
(Not to scale)
Figure 7.27 Results of Final Trials on Plywood Sheathing Compared with Standard Panel Test Data

Figure 7.28 Load Stepping During the Computer Based Analysis
7.6 The Test Programme.

In the preparation of the data files for the main series of trials using the "SADT" program the aims were as follows:

(i) to enable direct comparison with full scale tests,
(ii) to cover the principal sheathings used in timber frame walls,
(iii) to cover the main parameters in panel and wall design considered in the test section,
(iv) to check the ability of the program to analyse extreme cases,
(v) to extend the test information in areas where testing would be either difficult or expensive.

The first aim has meant that the approach to the coverage of the variables in wall design has followed that used in the full scale tests with the materials parameters checked using the 2.4m standard test panels. The number of variables has been considerably reduced from those detailed in the design method because each change in material type or size would have necessitated a further set of small scale tests to determine the effect on the cover nail parameters. The standard panel variations that have been covered in detail are:

(i) 9.5mm plywood nailed to 90 x 40mm SPF studs with 63mm long 3.25mm diameter gunned wire nails at 150/300mm centres,
(ii) as (i) above but with nail centres reduced to 75/150mm,
(iii) 9.0mm medium density fibreboard, with all other details as in (i) above,
(iv) 12.5mm bitumen impregnated insulation board, with all other details as in (i) above,
(v) 12.5mm bitumen impregnated insulation board, as in (iv) above but with nail centres reduced to 75/150mm,
(vi) 12.5mm plasterboard nailed to 90 x 40mm SPF studs using 40mm long 2.65mm diameter hot dip galvanised wire nails at 150/300mm centres;
(vii) 9.5mm plasterboard, with all other details as in (i) above,
(viii) 9.5mm plywood nailed to 90 x 40mm redwood/whitewood studs with 63mm long 3.25mm gunned wire nails at 150/300mm centres.

Types (i), (iii), (v) and (vi) simulate the tests used to determine the datum racking resistances of the four principal sheathings. Type (ii) was included to test nail spacing and the effect of strengthening a panel by additional nailing, the choice of materials allowed direct comparison with full scale tests. Type (iv) also covered nail spacing examining the effect of weakening the panel by reducing the number of nails. This problem was not examined in tests carried out as part of this investigation. Type (vii) was included to investigate the effect of board thickness, however, the nail parameters were not changed from those used with the 12.5mm board. This simplification may be acceptable for plasterboard but cannot be used with wood based sheathings. Type (viii) was included to determine whether or not the change of frame material (as denoted by cover nail resistance) would affect the results as much as had been noted in practice.

The wall parameters considered in detail were those of vertical load and length. It was not possible to consider openings in the detail covered by the full scale tests because of the complexity of the model which greatly magnified the size of the data file, the storage capacity required in
the computer and the time to run. The problem of modelling openings was covered by three separate trials on a 2.4m window panel where only the layout of the sheathing was altered.

The vertical load cases covered were restricted to those used in the standard tests, 0, 2½, and 5 kN/stud. In some secondary tests the 2½ kN/stud case, only, was considered because this was thought to represent the most common loading condition. One further vertical load condition was examined, this represented the ASTM test where the leading stud was prevented from lifting by making its connection to the bottom rail both very strong and very stiff. The ASTM configuration was only tested for 2.4m panels in an attempt to relate ASTM test data to basic racking resistance.

The effect of panel length has been tested in the range from 0.6m to 6.0m. It is in trials on the longer panels that the computer analysis can be used to maximum advantage. Therefore the work on wall length has been extended, from the materials covered in the test programmes reported in Chapter 6, to include a very strong sheathing arrangement (plywood closely nailed) and a much weaker combination of materials (BIIB with nails at 75/150mm centres).

In all wall tests the computer model represents a single panel with joints between sheathings made on a single stud. This has been shown, in full scale tests, to be an acceptable model for any type of wall construction using 1.2m wide boards, in addition it greatly simplifies the analysis.

Finally trials have been made to model combinations of sheathings using the method described in Section 7.4.4. As this method is a simplification of the practical arrangement trials have only been conducted on 2.4m panels.

Full details of all the material parameters used in the data files for the trials noted above are given in Table 7.1. These values are then referred to in the details of the model test program given in Table 7.2. This table also gives
information on the parameters used to run the analysis; they are as follows:

(i) the load step interval,

(ii) the trial stiffness values; this is given first for the bottom rail nails where more movement can be accepted and secondly for all remaining nails.

Finally in table 7.2 the lower bound design values are recorded. These figures are the single most important results of the computer analysis; their calculation is discussed in Section 7.7 below.

7.7 REDUCTION OF RESULTS

The output of the computer analysis has been reduced to detail the racking load, the racking deflection and the uplift of the leading stud. Three plots have then been drawn from each output they are:

(i) racking load to failure versus racking deflection,

(ii) racking load versus racking deflection to 10mm deflection,

(iii) uplift versus racking deflection to failure
Plots (i) and (ii) can be compared with the failure and first cycle stiffness test graphs for the standard tests but it is necessary to extract uplift data from the tables of test results to make comparison with plot (iii). (Note a typical output for a standard panel test is shown in Figures 5.5 a to c and Tables 5.3 a to d with explanation in Section 5.2.4). In the following pages the computer results are compared in this way with the standard panel results which have been recorded in abbreviated form in Tables 6.1 to 6.6.

Plot (i) represents the mean performance curve that could be expected for the wall. These results may be used with the safety factors noted in Section 5.2.3 and illustrated in Figures 6.24 a and b. Thus the maximum stiffness design load is the same as the mean performance load and the failure strength design load should approach seventy five per cent of the mean performance failure load.

Using the computer analysis it is not possible to consider the effect of load cycling where the fourth cycle (or lowest) racking load at 5mm deflection is used to predict the load at the allowable deflection of 7.2mm (0.003 times the panel height). However, the analysis allows the 7.2 mm deflection load to be directly measured. Therefore in order to cover the practical test data and the theory, the racking stiffness load in the analysis has been calculated as the lower of:

(i) \(1.25\) times the 5mm deflection load (denoted as (a) in the tables of results 7.3a to 7.3d),

(ii) the 7.2mm deflection load (b).
This may result in the analysis predicting a load slightly higher than would be expected from tests where the losses due to load cycling average 5%. This loss will be reduced if either the 7.2mm deflection load or the failure load govern the analytical design. Loads (a) and (b) have been determined using plot (ii), noted above, and together with the failure load (c) (or maximum racking load) shown in plot (i) have been used to calculate the following mean design properties.

(i) The test racking stiffness load \( (d) \) which is the lower of (a) and (b).

(ii) The test racking strength load \( (e) \) which is 0.75 times the failure load \( (c) \).

(iii) The test racking design load \( (f) \) which is the lower of \( (d) \) and \( (e)/1.6 \).

A lower bound design value has also been calculated to allow direct comparison with the test results where a 0.87 partial safety factor was applied to all the stiffness results because only two panels were tested in each programme. Then the following calculations have been used.

(i) The test racking stiffness load \( (d') \) is the lower of 0.87 x (a) and 0.87 x (b).

(ii) The test racking design load is the lower of \( (d') \) and \( (e)/1.6 \).

The lower bound value is not suitable for direct comparison with the datum racking resistance values which are based on average values for a much wider range of boards. The information abstracted from the test plots (which have not all been included) is given in Table 7.3 together with the intermediate calculations and both the maximum and lower
bound design values. Factors of safety are noted for the following cases:

(i) failure load in comparison with 5mm deflection load; a value commonly recorded in standard panel tests,

(ii) failure load in comparison with maximum design load; here a value greater than 2.13 infers that panel stiffness is the governing factor in design.

The factor of safety for the lower bound case has not been recorded, because only the stiffness load has been reduced. Instead the factor governing design (either stiffness (S) or failure (F)) is noted after the design value.

The lower bound results will be used in all the comparisons with full scale test performance.

The determination of design values for the 0.6m panels is not reliable, particularly at zero vertical load due to difficulties in achieving a satisfactory program run because of the weakness of the panel. A number of general points are noted from Table 7.3. Firstly little difference exists between the 7.2mm deflection load and 1.25 times the 5.0mm run deflection load. Although in many analyses the former result governs, the likely reduction to the latter due to load cycling would normally alter the situation and justify the practical procedure. Secondly, the factors of safety are clearly increasing with panel length, which is not seen in practice. This is not a problem in comparing the analysis with full scale test data when in both cases stiffness governs design. However, the result indicates an inaccuracy in the modelling technique which will be examined in the sections dealing with vertical load and wall length.
7.8 COMPARISON OF RESULTS WITH FULL SCALE TESTS

7.8.1 Standard Panels

The results of the analysis compared with standard panel tests are given in detail in Appendix D and are summarised here. Allowing for the variability to be expected in the results due to limited testing (Figure 6.24) it can be seen that the method of analysis gives a reasonable prediction of panel performance from the unloaded condition to failure throughout the range of vertical loads. There is a greater accuracy in the plywood results where the input data for the board and the cover nail behaviour were very much more reliable. It is therefore assumed that a more accurate assessment of the shear modulus of the board and the behaviour of the cover nail when used in long rows would greatly improve the performance of the model.

The results of the analysis show two major problems.

(i) There is insufficient differentiation between racking performance at higher vertical loads particularly with respect to initial stiffness.

(ii) There is insufficient uplift in the panels to match the test results.

The first problem will be covered in detail with respect to panel length in a later section but two points may be considered here. Firstly, the vertical load improvement factor at 5kN/stud is on average 1.55 compared with the 1.77 noted constantly throughout standard panel tests. It is therefore unreliable to calculate basic racking resistance values using the K302 modification factors noted in Chapter 5. Consequently, all comparisons have been made directly between results, and single value design figures
have been ignored. Secondly, the vertical load behaviour is different. This is illustrated in Figure 7.29 where it is seen that, unlike full scale tests where the initial stiffness increases with vertical load, using the analysis there seems to be an unique load deflection curve relating to a very high vertical load from which performance curves at lower vertical loads become progressively detached. This greatly reduces the significance of stiffness at higher vertical loads. Of the analytical models tested, the ASTM case will ultimately generate the highest vertical load effect because its value directly increases with the restoring moment required to prevent uplift which itself is related to the racking load. The ASTM curve therefore represents the unique curve in the analysis, however in practice at low racking loads the vertical load is reduced hence the initial stiffness of the curve will be low but will increase with racking load.

The vertical load response is closely associated with the second problem; that of uplift. This is illustrated in Figure 7.30 where the uplift versus racking deflection is recorded for the analysis of the main standard panel configurations (noting the mediumboard results to be similar to those of the plywood using the same nails, spacing of nails and frame material). The overlay shows the test results. The graphs show the relative effects of rotation and shear on panels. In general the uplifts from the computer analysis are lower than those recorded in tests indicating that the "perpendicular to grain" nail performance is high in relation to that of "parallel to grain" nails and, panel shear. At zero vertical load there is an approximately constant percentage error in each plot. It is notable that both the plywood tests, where nail values were determined more accurately, are in very close agreement with the test results. The mediumboard results are underestimated by 20% and those of the BIIB by 16%. In both test and analysis...
the behaviour at zero vertical load is independent of nail spacing. Under the 5kN/stud load the uplifts are clearly less as the vertical load reduces rotation, however, the relative behaviours are different; the tests show an initial rate of change of uplift which increases gradually with racking deflection whereas the analysis shows a reduced uplift at small deflections but with the rate of change tending to that of the test results at larger deflections. It is probable therefore that a further factor is influencing the results for small deflections and is due to the simplified technique for modelling vertical load. In general, at greater deflections, the computer analysis underestimates uplift by 10% for plywood and 20% for mediumboard but is accurate for BIIB. At lower deflections the percentage error is increased particularly for BIIB. Both test and analytical results for close spaced nails show them to be less influenced by rotational restraint than their standard spacing counterparts (i.e. the improvement in load is due to improved shear capacity as vertical load increases).

The effects described above can also be seen in the comparisons of racking load versus uplift behaviour recorded in Figure 7.31. There is much less difference between the two graphs than in figure 7.30 although initially uplifts are reduced at higher vertical loads.

In general the results indicate that the uplift is critical to the modelling and may explain the poor vertical load improvement factors. The reduced uplift both decreases the stiffness of panels, particularly at high vertical loads, and increases the failure load which is likely to be critical at low vertical loads, and thereby greatly reduces the vertical load factor, as seen in the results.
The weakness in modelling causing this problem is a result of the following points.

(i) The analysis measures only the uplift of the stud relative to the bottom rail whereas the test measures the movement relative to the base of the test rig when there is likely to be a component due to the uplift of the bottom rail itself. This is because the first bolt fixing to the baseplate is 170mm behind the leading edge of the panel.

(ii) The vertical load in the analysis is carried directly through the stud resulting in very low initial uplifts at high vertical loads. Test evidence shows an immediate stud uplift in these cases which suggests that some vertical load is transferred via the top and bottom rails and the sheathing alone.

To overcome the problem the analytical model will have to be altered to trace more accurately the vertical load path. Thus the vertical load will have to be applied to the top rail and all frame joints will have to be defined for both compression and tension movements and will have to respond automatically in the correct direction to the summation of the forces on the joint caused by the vertical and racking loads. For greater accuracy a gap component can be added to the compression case in order to model the constructional inaccuracy when the sheathing is nailed to the framing leaving a small gap (probably less than 1mm) between the stud and the rail. The final requirements for the frame nail response to loads parallel to its line of action are shown in Figure 7.32.

The incorporation of these changes into the program together with the improvements in accuracy of the
nail modelling and shear modulus data should substantially improve the quality of the analysis and would allow it to be treated in an identical manner to a set of tests conducted on five or more identical panels.

The effect of nail spacing on design values is set out in Table 7.4. However, the results are of limited use because it has been seen that the analysis will overestimate failures, and thus the enhancement factor, at zero vertical load in plywood and will underestimate the increase in stiffness at high vertical loads, thus affecting the 5kN/stud result for plywood. The stiffness differences are likely to be less significant and it is likely that the 1.43 factor proposed from tests will achieve a small but adequate margin of safety. In the BIIB case where uplift is of limited importance due to the weakness of the board in resisting nail movement the results of the analysis are likely to be more accurate and the 37% reduction used in the design allows an adequate margin of safety on tests.

The above comments may also be applied to the combined sheathing cases also detailed in Appendix D. A further analysis has been carried out on combined sheathings, which indicates that the nail parameter equations for the individual boards can be summed to determine the nail properties for the combined case. This would greatly reduce the requirements for nail tests because all combinations of board could be covered by their individual performances. It would even be possible to consider different nail spacings for each board if they were related to that of the stronger board. For example if plywood with nails at 75/150mm centres is used with plasterboard nailed at 150/300mm centres the plywood nail data would be added to half the plasterboard nail figures to generate an equation to be applied to a 75/150mm spacing in the analysis. At the same time the board properties could also be summed by the modular ratio method.
7.8.2 **Vertical Load**

The standard panel results indicated that the computer based analysis did not predict such large differences in racking performance for changes in vertical load as would be expected in full scale tests. The vertical load performance for all types and lengths of walls is shown in Table 7.5. The variation factor quoted is:

\[
K_{VL} = \frac{K_{VL}}{K_{VL_{0}}}
\]

and therefore represents the analytical requirement for the vertical load factor \(K_{110}\), noting that corrections must be made for stud loading conditions using equation 6.12. The following points are noted:

(i) the computer results are unreliable for 0.6m panels due to difficulties in operational procedures,

(ii) the results are variable for the 2.4m panels and will normally be higher if failure governs the design value at high vertical load,

(iii) the vertical load enhancement factor decreases rapidly with wall length which is in agreement with the test results although the differences between the analysis and the test become more marked with increasing length.

Racking load versus panel length plots are shown in Figures 7.33 a to c and give a clearer indication of the vertical
load behaviour. It is seen that the plots for different vertical load conditions are essentially parallel, except for the short panels where the lines converge to the origin. This infers that the design equation for plain walls would take the form:

\[ W_{RL} = (BRR \times L \times KL) + (VL \times KVL) \quad - 7.9 \]

zero vertical load design effect

It is notable that the increase in performance due to vertical load should be totally independent of length because the inclusion of the wall length component in \( K_{110} \) has, in part, the same effect in comparison with the original \( K_{100} \) modification factor used in the Code. The tests results clearly justify \( K_{110} \), however, and do not agree with those of the analysis.

Two explanations may be given for this discrepancy. The first, concerning the inaccurate modelling of the vertical load, has already been discussed. The second relates to the low initial stiffness achieved by the panels at high vertical load which has been attributed to an inadequate compensation for the effect of nails acting in rows when using the single nail "parallel to grain" test. The design values achieved by the analysis (Table 7.3) show that the longer the wall the more dominant is the initial stiffness on the design value. Thus the vertical load factor is increasingly influenced by the poor modelling of panel stiffness as lengths extend.

For many years it has been considered desirable to be able to compare directly the results of the standard British test with those of the ASTM test. This would greatly increase the information available for the determination of the design factors. However, experimenters were normally strongly biased to one of the
methods and there did not seem to be a sufficient overlap in the results to warrant an attempt at comparison. The computer based analysis now makes comparison possible because trends can quickly be checked for a wide range of materials. The results of the ASTM comparisons noted in Appendix D have been assembled in Table 7.6. In order to compare the ASTM test the load versus deflection plot has been reduced in a similar manner to the British test. The following comparisons have been made:

(i) failure loads at both zero and 5kN/stud,

(ii) design loads at both zero and 5kN/stud.

The 5kN/stud cases have been included because the vertical load factors are known to be wrong and differentials will be less at the higher vertical loads.

Table 7.6 indicates that failure result comparisons are of limited use because they are variable and are markedly different from the design value comparisons which also include stiffness details. The latter case indicates that separate factors will be needed for different boards. The BIIB fares particularly badly in the ASTM test because of its weakness in nail resistance. This fact has also been noticed in testing (Anderson, 1965), and is considered to be an additional deficiency in the ASTM test because BIIB can certainly be shown to be an effective sheathing for use in low loading conditions such as in single storey construction.

Taking account of the deficiencies in the vertical load analysis it is possible that the modification factor for ASTM restraint tests related to basic racking resistance (i.e. at zero vertical load) would be:

(i) 2.10 for normal sheathings,
(ii) 1.85 for less dense sheathings.

It is noted that the work on ASTM tests has been restricted to 2.4m square panels, the size defined in ASTM.E.72 test method, although previously (Anderson, 1965) longer panels were tested. Thus the factors noted above could be used with ASTM test results reduced by the standard method defined in Chapter 5 to determine basic racking resistance values for direct use in wall design.

7.8.3 Wall Length

The effect of wall length on the computer based analysis is considered in Table 7.7 by comparing results at a given length with those of a standard panel for each condition of vertical load. Thus the comparison factors represent:

\[
\text{Length Factor} = \frac{K_{110L/V} \times K_{111L} \times L}{K_{1102.4/V} \times K_{1112.4} \times 2.4}
\]

and they may be compared with design values calculated using that equation. Here the most accurate assessment of \( K_{111} \) has been used based on equations 6.36 taking the power curve solution up to 1.4m length and a linear solution thereafter to 4.8m; no enhancement is allowed after 4.8m.

The results of the analysis are seen to be reasonably consistent for different sheathings and nail spacings although it is noticeable that the weak BIIB exhibits lower safety factors. The values for the 0.6m panels are again not reliable due to convergence difficulties. The averaged results compare well with the design values particularly at zero vertical load where the modelling is most efficient. The test results are underestimated at higher vertical loads in longer walls,
but this is undoubtedly due to the difficulties already noted in the vertical load analysis.

The accuracy of the modelling at zero vertical load for such a wide range of walls gives a very positive indication that the analytical method is satisfactory and that much more accurate results are possible with improved materials parameters and when the vertical load modelling problem has been overcome.

It is notable that the length factors are lower for BIIB which suggests that weak sheathings are less able to take full advantage of strengthening factors, in this case the wall length, due to the inherent weakness in the nail holding capacity of the board. This gives further support to the argument given in Section 6.4 which resulted in a reduced value of datum racking resistance for BIIB but does not agree with the general trend for greater improvements in initially weak materials combinations. It therefore indicates the need for some long wall tests to check the suitability of the length factor for use with weak materials.

The wall length behaviour of the different combinations can be seen in Figures 7.33a to c. It is clear that the effect of length reduces as length increases. Although there is no marked change between 4.8 and 6.0m, in all the cases tested the response after 4.8m is less than the proportional approach proposed by the design method where:

\[ WRL = BRR \times 1.30 \times L \times (K_{VL}) \]

Thus if the computer analysis is to be accepted the length modification factor \( K_{111} \) should be altered to:

\[ K_{111} \quad (L>4.8) = 1.30 \left( \frac{4.8}{L} \right)^{x} \]

-7.11
where $x$ is less than one.

The length results, at present, are not sufficiently conclusive to adopt this change but they do illustrate the dangers in extrapolating the power equation for lengths greater than the 4.8m value tested, as is done in the Code of Practice.

7.8.4 Openings

The panels analysed to investigate the effect of openings are shown in Figure 6.39. In addition an intermediate panel using 'L' shaped sheathings, similar to the 'C' shape but without any board over the lintol, and a singly sheathed panel, with a hole cut out for the window, were analysed. The frame details for the analysis are shown in figure 7.34. The positions of the tension joints were fixed from experimental knowledge and the size of the lintol member immediately over the window was checked and found to have no significant effect on the racking resistance, hence in the final analysis it was input as a 90 x 40mm standard section. The design values and a comparison with test data are given in Table 7.8. The load/deflection and uplift/deflection curves to failure are shown in Figures 7.35 and 7.36 respectively.

The correlations between the tests and the analysis are remarkably good considering the complexity of the problem. The window panels have been noted (Chapter 6) as being more heavily influenced by shear movements than overturning in comparison with plain panels. This means they are better matched to the computer model and this is confirmed in the uplift and vertical load results which are very much more accurate than in plain panel tests.

The failure to model the uplift of the panel with three rectangular sheathings in the latter stages of the
zero vertical load analysis is probably due to a feature of panel testing which is omitted from the analysis. This relates to the additional restraint caused by direct frictional contact between the boards and is seen to an exaggerated scale at the leading bottom corner of a window panel as shown in Figure 6.37. The version of the SADT program used in the analysis does not take account of sheathing interaction and theoretically allows the boards to overlap. This alters the mode of resistance of the panel and can lead to a reduction in the effectiveness of the analytical model. The problem can be overcome by using a gap element between the sheathing boards which initially allows freedom of movement, but for larger deflections when the boards come into contact, provides a high resistance to further horizontal movement of the board without a similar movement in the adjacent board. Gap elements have been used by Foschi (1977), in dealing with trusses, and by Castillo and Gutkowski (1984) for the purpose noted above. The gap element is similar to that previously described for dealing with frame joints. Its omission will also have affected plain panel analysis but, in general, its effect is of secondary importance. Hence, as it was not directly available for use in the SADT analysis of walls, it was omitted from the preliminary trials.

The vertical load factors for the analysis between zero and 5kN per stud are 1.52 and 1.45 for the three sheathing and 'C' sheathing cases, whereas in testing the results are 1.35 and 1.42 respectively. For the 'L' and single sheathing analysis the factors are 1.43 and 1.42. It is probable that in the analysis the factor reduces with increasing zero vertical load resistance. The test values, however are very low in comparison with plain panels (expected value 1.77) but this is due to the short length of panel and subsequent narrow window pillars which are prone to greater shear and bending. The low factors are confirmed in the full scale tests but it is also notable that in longer perforate walls the vertical load factors
were similar to those obtained for plain walls.

Comparing the four sets of analytical data the computer can be seen to model adequately the changes in the sheathing configuration. The increase in resistance of the single sheathing is notable, although it is not a practical solution for boards produced in standard 1.2m widths. However, the result is of use to the chipboard industry where single boards can be manufactured to cover whole walls. Here the possible benefits of such walls could be predicted very quickly by the SADT program. The result could also be of use in determining the advantages to be gained, in terms of strength, from the taped plasterboard joints.

7.9 TRIAL STIFFNESS TESTS

7.9.1 The Effect of Input Parameters on the Requirements for Trial Stiffness

The primary consideration when using the SADT program to achieve a satisfactory analysis was found to be the trial stiffness of the cover nail. The success of a particular estimate for this value is checked by investigating deflections for successive load steps. Deflection intervals should increase slowly at first, and then in a fairly even manner up to 40mm total deflection when the program should fail to reach the next load step within the allowed number of iterations. The maximum sensible deflection depends on the 'bang' term; a reduction from 12.5mm (standard for plywood) to 6.0mm (for plasterboard) will reduce the failure deflection to 25mm although failure cannot be accurately determined as the load cannot be made to decrease. If the trial stiffness guess is not perfect load stepping will occur throughout the plot. This is not a problem if the steps do not start within a deflection of 7mm and the failure pattern is
normal. However, poor estimates may result in load steps in which deflection decreases; in this case programs should be rerun with a new trial stiffness. In some outputs small back steps have been allowed if it can be seen that they do not affect the overall performance of the panel. Usually backstepping has occurred in panels where it has proved difficult to obtain a fully satisfactory program run.

Two problems are associated with failure to achieve a successful analysis. The first is when the program fails to iterate a load step very early in the run when failure is catastrophic; clearly a better trial stiffness is required. The second occurs at large deformations whereby after a large deflection the load rises in single iteration steps beyond the expected failure load. This problem is overcome by selecting a suitable deflection step to represent failure and then ignoring the rest of the output.

During the main program runs it was thought that the first problem indicated two high a trial stiffness and the second too low a value. Corrections based on these principles worked satisfactorily but later tests showed this premise to be only partly true, as described later in this section.

Trial stiffnesses were selected by trial and error. The values used in the successful runs are shown in Table 7.2 along with the standard load step. Normally, a logical progression could be used to guess suitable values particularly at high loads and with long panels. Difficulty was experienced with the shorter panels and thought had to be given to the factors likely to affect the trial stiffness. These factors are split into three groups based on program running parameters, panel length and loading, and materials. They are as listed below.

(a) Tolerance for convergance.
Foschi's recommendation of 0.001 was adhered to throughout the main program runs. Reducing the factor was unnecessary in terms of accuracy and increased the number of iterations required at each load step and is then costly in computing time while increasing its value increased the likelihood of single step iterations and thus gave problems in defining the failure point for the panel.

(b) Number of iterations. Foschi suggested a value of 20 but he was probably not considering the need to trace maximum load. The value was increased initially to 50 when the programme was run interactively and later to 150 when it was run on batch and time considerations were not important. The greater the number of iterations the better the run behaved close to failure.

(c) The load step. Originally, when the tracing stiffness was the initial stiffness of the nail slip curve, the load step had to be kept very small; 0.1kN or lower. The introduction of the trial stiffness component allowed the steps to be increased. For standard 2.4m and also 1.2m panels the step was kept at 0.1kN but it was increased up to 1.0kN for 4.8m and longer panels. As stronger combinations were tested the load step was increased to cut down the number of data points e.g. for ASTM tests on 2.4m panels, 0.25kN increments were used. Conversely 0.05kN increments were used on 0.6m panels where the total load was extremely low. The greater the load step the lower the trial stiffness required leading to better convergance of the program by avoiding short periods of instability. However, large loads reduce the accuracy with which failure can be traced.

(d) The initial load step When the trial stiffness parameter was introduced it was thought that a large initial load step would allow a lower trial stiffness which would help in tracing the failure
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(d) The initial load step
When the trial stiffness parameter was introduced it was thought that a large initial load step would allow a lower trial stiffness which would help in tracing the failure
load. Subsequently it was noted that the initial load step did not affect trial stiffness. However, it was noted that the smaller the load step the more important it was to make the initial value a multiple of the load step, consequently 1kN was often used.

(e) Panel length.
In theory the longer a panel the greater its strength and stiffness. Thus, for a given step size, the trial stiffness should increase with panel length. However, in practice, the load step has also increased with panel length and has greater significance than the change in length, thus the trial stiffness has often been reduced.

(f) Vertical load.
Theory suggested that higher vertical loads would require higher trial stiffness values. In practice this was found to be untrue; it was found that there was a wider tolerance in trial stiffness to successfully run an analysis for high vertical load and that a subsequent zero load analysis could normally be run using an increased value of trial stiffness.

(g) The behaviour of the cover nail.
Full scale tests indicated that the deflections of the bottom rail cover nails were very different from those of the other cover nails because they were predominantly perpendicular to the grain. It was decided therefore to enter them separately into the program so that they could be given a different trial stiffness although their general nail slip behaviour determined by the Kimber curve parameters was identical to the other nails. Initially, it was thought that a lower trial stiffness should be used for the bottom rail nails and in practice a 1:2 or 1:3 ratio was used in estimating the stiffnesses of the two sets of nails. Later experience suggested that this was only necessary at zero vertical load when uplifts were comparatively high. The effect of changing the ratio
between the two nails is also investigated later in this section.

(h) Parallel and perpendicular to grain properties of the cover nail.
For very small nail movements the behaviour of the Kimber curve is based on the 'a' value (equation 7.10). Often this parameter has very different values for the parallel and perpendicular to grain behaviour of the same nail in a board material. However, after deflections of 0.25mm the 'a' term is less critical and there will be normally less than 25% difference between the levels of performance. Therefore, because the main problems concerned the deflections at failure, it was decided to use the same trial stiffness for both parallel and perpendicular nail properties. It was noted that where there were large differences in the value of 'a' there were particular problems in getting the program to calculate the first load step particularly if the step was small.

(i) Frame nail and frame materials.
The frame nails used the Foschi parameters and did not have a separate trial stiffness. It was thought that the sudden change in stiffness of the nail caused by the parameters chosen to model vertical load would cause problems in running the program. These doubts were not realised and, within practical limits, the tests showed that neither nail nor material parameters in the frame affected the running of the program.

7.9.2 Testing the Effect of Changes in Trial Stiffness

A series of tests was carried out on the doubly clad panels to trace the relationship between trial stiffness and maximum racking load. The tests were restricted to 2.4m panels but covered all the main boards and, therefore, large differences in nail parameters. Because these panels
were relatively strong a 0.25kN load step was adapted together with an initial 1.0kN step. At first the trial stiffness ratio between the standard nails and those in the bottom rail was fixed at 1:2.

The tests on the ply and plasterboard panel (Figure 7.37) showed that the higher the vertical load the more easy it was to find a suitable trial stiffness. The two tests where the ratio between nail types was changed indicated that the bottom rail nail stiffness was the critical factor.

The tests on the mediumboard and plasterboard and the B11B and plasterboard panels did not appear to give the same response. There was no correlation between failure load and the trial stiffness assigned to either the bottom rail nail or the standard nail. The changes in failure load were remarkable for very small changes in stiffness. Tests on the insulation board combination showed that this peculiarity was not affected by changes in the ratio of nail stiffness. It became clear that the full response could not be determined without plotting results for very small increments in stiffness at all three loads, and for different stiffness ratios.

This was not practical within the scope of the investigation and the work on trial stiffness had to be abandoned. The general conclusion is that no simple relationship exists between trial stiffness and failure load for either bottom rail nail or standard nail behaviour. Changes in behaviour are so sudden that no reliance can be put on any trend. It is potentially dangerous to join any points on the racking load versus trial stiffness graphs and even the plywood curves must be regarded as fortuitous.
7.9.3 General Conclusions Concerning the Choice of Trial Stiffness

The vertical load results in particular, suggest that the factor which causes the major problem in the effectiveness of a trial stiffness value is the range of deflections experienced in the bottom rail fixings perpendicular to the grain of the frame timber. This is due firstly to the effect of the rotational movement on the extreme fixings and secondly the reduced restraint (particularly at zero vertical load) afforded by the other frame members under the leading stud of the panel. This indicates that the trial stiffness values should be related as much to the direction of nail movement as to the location of the nail. This change could significantly improve modelling at zero vertical load and in short panels. The other conclusions that may be drawn from the work on trial stiffness are that:

(i) it must be increased for higher 'a' values in the 'Kimber' modelling equation,

(ii) it can be decreased as load interval increases,

(iii) it should be increased if the program continues to load step in single iterations in the failure region.

The trial and error approach to selecting trial stiffness has proved acceptable in the initial tests covered by this investigation but is unsatisfactory if use of the program is to be developed. Thus a further investigation of trial stiffness will be a principal requirement of development work.
7.10 SUMMARY

The aim of this section of the investigation was to determine if a computer based structural analysis package could be developed to comprehensively model timber frame wall behaviour and also be compatible with the British test method and the design parameters for the materials and the walls covered in the empirical design and test section. Because this was a secondary part of the overall project it was necessary to find a computer program that could be developed without undue difficulty to model wall behaviour covering all the constituent structural parts of the wall. The SADT program had been developed for the design of timber diaphragms and was considered a very suitable starting point. A number of major modifications had to be made to the program and a few simplifications had to be included to achieve the main aim of the work. The simplifications, such as the method of application of vertical load and the omission of load cycling did not affect the fundamental approach of the analysis and could be modified at a later date if the method of analysis proved successful.

A major difficulty was experienced in obtaining the required input data for the range of materials necessary to check the ability of the analysis to model extreme cases. Many of the boards used are considered non structural and do not have their properties fully defined. The nailing information required was not available from other sources due to the very specific needs of the analysis in covering behaviour through to maximum load. The data had therefore to be generated as part of the investigation and had to be rudimentary in form.

In the limited time available it was possible to make only an initial check of the input parameters and their effect on standard panel tests. The main program runs were then undertaken to extend the analysis to cover the major
aspects of wall design. It has not been possible to take advantage of these results to improve the input data and thus the accuracy of the model.

The overall performance of the method of analysis has exceeded expectations. The analysis has coped with all the material modification factors and the wall design factors through a very wide range of tests and maintained an accuracy within ±10% of the test results in the majority of cases. This indicates that the modelling techniques used in SADT are, in general, acceptable and are sufficiently rigorous in their approach. The representation of a rectangular sheathing by a single finite element is one area of concern with regard to accuracy. In general this has proven adequate but in the window panel tests where three or more elements were needed to model the more complex sheathing shapes the accuracy of the analysis was commendable even though the panel was very intricate. The accuracy of the window panel clad with rectangular boards was less impressive, due to the more complex behaviour of the narrow boards on either side of the opening. It is possible therefore that the response of such panels may be improved by breaking the boards into more than one finite element if their width is 600mm or less and they are full height. However, it is unlikely that this alteration will have significant effect on full length walls unless the boards are isolated between openings. For modelling standard 1.2m wide sheathings the single cubic isoparametric finite element may be regarded as wholly satisfactory.

Three main areas of inaccuracy were found in testing the analysis. They related to the following.

(i) The vertical load factor, where the analysis was unable to model the full variation in racking performance experienced during testing.
(ii) Low initial racking stiffness of panels, particularly under high vertical loads.

(iii) The increasing factor of safety noted in longer panels which directly contradicted test evidence.

Two further problems were encountered which affected the use of the analysis, they were:

(i) difficulties in getting the program to run, normally due to the choice of trial stiffness,

(ii) the inability of the program to model panel behaviour after maximum load had been achieved.

Proposals for the development of the SADT program to improve the accuracy of the modelling and its ease of use can be divided into three categories, they are:

(i) changes to the data files fundamental to achieving adequate accuracy of modelling,

(ii) major alterations to the program to improve the quality of the model and its ease of use,

(iii) additions and minor amendments to the program and data files to give improved accuracy of modelling.

However, most of the problems noted above are interlinked to two specific areas of the analysis, i.e. the cover nails and the application of vertical load. Proposals to overcome the problems, can be made in more than one of the categories for development.
It has been shown that inaccuracy in the cover nail data for the "parallel to grain" nails significantly affects the stiffness performance of the panel under analysis. Increasing stiffness at higher vertical loads will also improve the vertical load performance and reduce the factor of safety in longer panels. Cover nail data related to the Kimber curve parameters and the 'bang' term can readily be improved by reviewing the nail tests. Firstly, the load parallel to the grain nail data for all sheathings must be more accurately determined with respect to the performance of rows of nails rather than individual fixings. More information is necessary regarding the nail resistances at very low deflections (i.e., less than 1mm) which are critical to the initial panel behaviour. Finally, more care must be taken in the assessment of the 'bang' function relating it to the initial drop off in load rather than a later more substantial load reduction; this will achieve a more brittle failure at low vertical loads which will be in closer agreement with test results. These changes all fall within the first development category.

The Kimber curve is not a perfect model of the nail performance, hence the need for the 'bang' term, however, it is reasonably accurate through a very wide range of deflections which was essential to the SADT investigations. If substantial alterations were made to the program a better nail modelling technique could be developed. The most suitable form would be based on the work of Castillo and Gutkowski (1984) whereby the nail behaviour is modelled by a series of data points forming a "stepwise curve" (see Figure 7.38). The benefits are greater accuracy; ease of modelling after maximum load and that the trial stiffness can be more easily related to the stiffness between data points. Further alterations to the SADT program would enable it to model panel performance after maximum load had been reached so that it took full advantage of the nail behaviour data and also modelled the redistribution of load after nails had failed. This could
be done using the present load-step system such that if an increasing load step cannot be traced the load increment is removed. However it may be advantageous to convert the analysis to a deflection stepping system because failure is such an important factor in panel design.

Minor improvements in cover nail behaviour could be made by utilising more sophisticated nail models within the deflection limits to which they apply.

The vertical load problem requires major program alterations so that the vertical load can be applied to the top of the panel and can follow a path through the sheathing as well as through the studs. This alteration should result in the greater initial uplifts necessary to model test behaviour and reduce both failure loads and factors of safety for long walls. The change will necessitate the definition of the frame to frame connector in both tension and compression. Greater accuracy can then be achieved by incorporating a gap element in this connector to simulate construction tolerances.

A gap element could also be included along the joint between sheathing boards to more accurately model the practical situation. A similar approach to that adopted by Castillo and Gutkowski (1984) should prove suitable but would not have a major effect on performance.

An accurate assessment of the sheathing parameters, such as modulus of rigidity, is urgently required if SADT is to be developed, particularly in respect of boards other than plywood. At present reduced values for the modulus have been used whereas to accurately model panel behaviour the mean performance level should have been used. A final development necessary for the accurate assessment of racking load and uplift is the inclusion of an element to cover base connections so that the bottom rail of the panel is not considered to be rigidly fixed to the foundations.
underneath each stud connection point.

The following changes are not essential to the use of the SADT analysis but will be of increasing advantage as the program's use is extended to cover typical wall situations. They are:

(i) to rationalize the computational method to help in the analysis of walls with many openings which quickly become very complex in terms of numbers of joints and equations that have to be solved,

(ii) to rationalise imput data so that full length walls can be input more quickly,

(iii) to include a brick tie element to enable composite brick wall design to be undertaken.

In conclusion it should be noted that the results obtained in the main trial runs are not sufficiently accurate, because of the input data, to be used to change the design parameters found from full scale testing. However, the results of the analysis do confirm the general trends included in the design proposals. It is thought that the method of analysis could be developed, incorporating the amendments given above, and used to verify, and perhaps to improve, the datum racking resistances and modification factors covered by the code as well as extending the application of the modification factors. Further work would be needed to enable the program to cover the third set of wall design parameters such as external fastenings but then the scope of its use would only be limited by the available nail test data which could easily be extended to cover wetted panels and new sheathing materials.
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Table 7.1 Materials Parameters Used in the Main Analytical Test Programmes.

1. Frame nail properties
   - for all types are identical and based on the Pariser curve:
   - withdrawal (along frame)
   - shear (perpendicular to frame)
   - rotation
   - $k_1 = 100,000$ (N/mm)
   - $k_1 = 1400$ (N/mm)
   - $V$ is the vertical load in newtons
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Table 7.2a Details of the Main Analytical Test Programmes
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Table 7.2b Details of the Main Analytical Test Programmes
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Table: 7.3 Reduction on Analytical Results to Give Standard Design Values

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Table 7.3: Reduction of Analytical Results to Give Standard Design Values

(b) Mediumboard Walls and Plywood Window Panels
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<th>PANEL TYPE</th>
<th>PANEL FILE NO.</th>
<th>VERT. LOAD kn/stud</th>
<th>RACKING LOAD at 5mm DEFL. (a) kn</th>
<th>RACKING LOAD at 7.2mm DEFL. (b) kn</th>
<th>RACKING LOAD 1.25 x a (c) kn</th>
<th>FAILURE LOAD Sim. Defl. (d) kn</th>
<th>FACTOR OF SAFETY (e)</th>
<th>MAXIMUM DESIGN RACKING LOAD (f) kn</th>
<th>FACTOR OF SAFETY (g)</th>
<th>LOWER BOUND RACKING LOAD (h) kn</th>
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Table 7.3 Reduction of Results to Give Standard Design Values
(c) BLIB Walls and Combined Sheathing Walls
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<th>PANEL FILE NO.</th>
<th>VERT. LOAD</th>
<th>RACKING LOAD at 5mm DEFL. (a)</th>
<th>RACKING LOAD at 7.2mm DEFL. (b)</th>
<th>RACKING LOAD 1.25 x a (d)</th>
<th>FAILURE LOAD (c)</th>
<th>FACTOR OF SAFETY failure load (f)</th>
<th>MAXIMUM DESIGN RACKING LOAD (kN)</th>
<th>FACTOR OF SAFETY failure load (f)</th>
<th>LOWER BOUND DESIGN LOAD (kN)</th>
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<td>8.91 F</td>
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</table>

Table 7.3 Reduction of Results to Give Standard Design Values

(d) Plasterboard Walls
Mk with Vertical Load

a) Test Result

b) Analytical Results

Figure 7.29 Comparison of Load/Deflection Behaviour for Different Vertical Load Conditions
Figure 7.30 Comparison of Uplift Versus Racking Deflection Behaviour
Figure 7.31 Comparison of Racking Load Versus Uplift Behaviour

Table showing relative performance at different vertical loads

<table>
<thead>
<tr>
<th>Uplift</th>
<th>Racking load for 5kN/stud vertical load</th>
<th>Racking load for zero vertical load</th>
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<td>1.77</td>
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</tbody>
</table>

(a) Computer analysis

(b) Standard panel tests
Notes

(i) Allowable withdrawal load for nails in joint: at present 0.2kN.
(ii) Construction tolerance gap between stud and rail.
(iii) Stiffness of joint to initial nail withdrawal which, being based on frictional shear motivated between the fixing and the stud material, can be applied to small movements in both tension and compression when all the load is transferred through the fixing.
(iv) Final high compressive stiffness when frame and rail members come into contact.

N.B. Vertical load is applied independently to the frame joint

Figure 7.32 Revisions to the Frame Joint Model
<table>
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<tr>
<th>Board Type</th>
<th>Vertical Load kN/Stud</th>
<th>Nail Spacing</th>
<th>Percentage Change From Standard Spacing</th>
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Table 7.4 Result of Analyses Concerning Nail Spacing
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<td></td>
</tr>
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<tr>
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<td>Ply/SPF</td>
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<td></td>
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<td>1.77</td>
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<td>1.54</td>
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</table>

* Inaccurate due to poor convergence in the analysis.

Table 7.5 Vertical Load Variation Factors For Lower Bound Design Values Based On the Computer Analysis.
Figure 7.33 Racking Load Versus Panel Length Results For The Principal Sheathings Showing the Effect of Vertical Load on Wall Performance
<table>
<thead>
<tr>
<th>Board Material</th>
<th>Nail Centres</th>
<th>ASTM Test Results</th>
<th>Standard British Test Results</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Failure Comparison</td>
<td>Design Comparison</td>
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<td>Zero Vert.</td>
<td>SkN/Stud</td>
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<tr>
<td>Plywood</td>
<td>150/300</td>
<td>2.27</td>
<td>1.69</td>
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<tr>
<td>Mediumboard</td>
<td>150/300</td>
<td>1.89</td>
<td>1.23</td>
</tr>
<tr>
<td>B11B</td>
<td>75/150</td>
<td>1.90</td>
<td>1.27</td>
</tr>
<tr>
<td>Plasterboard</td>
<td>150/300</td>
<td>2.04</td>
<td>1.29</td>
</tr>
<tr>
<td>Plywood</td>
<td>75/150</td>
<td>2.37</td>
<td>1.56</td>
</tr>
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Table 7.6 Comparison of Analyses Based on the ASTM Test Restraints And The Standard Vertical Load Cases

<table>
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<tr>
<th>Panel</th>
<th>Vert. Load</th>
<th>Length</th>
<th>0.6</th>
<th>1.2</th>
<th>2.4</th>
<th>3.6</th>
<th>4.8</th>
<th>6.0</th>
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<tbody>
<tr>
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<td>2.17</td>
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<tr>
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<td>1.82</td>
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<td>1.00</td>
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<td>2.43</td>
<td>3.00</td>
</tr>
<tr>
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<td>0</td>
<td></td>
<td></td>
<td></td>
<td>1.74</td>
<td>2.43</td>
</tr>
<tr>
<td>Average For Computer Analysis</td>
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<td>1.00</td>
<td>1.82</td>
<td>2.59</td>
<td></td>
</tr>
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<td>2.5</td>
<td>0.08</td>
<td>0.35</td>
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<td>1.63</td>
<td>2.12</td>
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<td>1.00</td>
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<td>Proposed Design Values</td>
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<td>2.26</td>
<td>2.72</td>
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</tbody>
</table>

Table 7.7 Wall Length Modification Factors For Lower Bound Design Values Based On The Computer Analysis
T = Tensile joint along the bottom rail  
Performance varies with vertical load  

T' = Tensile joint (of different stiffness to T)  
Performance does not vary with vertical load  

C = Compressive joint  

Figure 7.3 Frame Joint Behaviour In The Window Panel Model

<table>
<thead>
<tr>
<th>Window Type</th>
<th>Vertical Load kN/Stud</th>
<th>Design Racking Load (kN)</th>
<th>Percentage Difference in Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Computer Analysis</td>
<td>Full Scale Test</td>
</tr>
<tr>
<td>Three</td>
<td>0</td>
<td>2.92 (S)</td>
<td>3.10 (S)</td>
</tr>
<tr>
<td>Sheathing</td>
<td>2.5</td>
<td>3.81 (S)</td>
<td>3.91 (S)</td>
</tr>
<tr>
<td>Boards</td>
<td>5</td>
<td>4.45 (S)</td>
<td>4.18 (S)</td>
</tr>
<tr>
<td>L-Shaped</td>
<td>0</td>
<td>4.10 (S)</td>
<td>5.07 (S)</td>
</tr>
<tr>
<td>Sheathing</td>
<td>2.5</td>
<td></td>
<td>5.87 (S)</td>
</tr>
<tr>
<td>Boards</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-Shaped</td>
<td>0</td>
<td>4.73 (S)</td>
<td>4.48 (S)</td>
</tr>
<tr>
<td>Sheathing</td>
<td>2.5</td>
<td>6.04 (S)</td>
<td>5.76 (S)</td>
</tr>
<tr>
<td>Boards</td>
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<td>6.86 (S)</td>
<td>6.35 (S)</td>
</tr>
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<td>Hole Cut</td>
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<tr>
<td>For Window</td>
<td>5</td>
<td>8.65 (S)</td>
<td></td>
</tr>
</tbody>
</table>

Table 7.8 Results of The Window Panel Analysis Compared With Test Results
Figure 7.35 Window Panel Performance Comparison: Panel With Three Rectangular Boards
Figure 7.36 Window Panel Performance Comparison
Panel With 'C' Shaped Boards
36.5mm
25mm
20mm
Nail 2 = 1500
Nail 2 = 1100

Note: Standard nail stiffness is twice bottom rail nail stiffness unless otherwise stated.

Figure 7.37 The Effect of Bottom Rail Nail Trial Stiffness on Racking Performances: Ply and Plasterboard Results

Load (KN)

Nail performance curve modelled by step-wise data points.

Figure 7.38 Modelling Nail Performance as a Step-wise Curve
CHAPTER 8

THE RACKING RESISTANCE OF BUILDINGS

8.1 INTRODUCTION

Following the reduction of the test results and the work on the computer based analysis, the racking resistance of timber frame buildings can now be evaluated. The procedure follows that outlined in Chapter 4 whereby the resistance component for each sheathing on each wall affected by the applied load is determined and summed, taking account of any restrictions in use of materials considered to be of only partial structural value. The total resistance can be checked against the applied load which, if necessary, can be reduced if it can be shown that in part the load is resisted by other elements of the structure before reaching the timber frame.

Considering first the design of the individual walls, many of the factors have been evaluated in Chapter 6 and are used here without further proof. Very limited use has been made of the computer based analysis because the initial trials at modelling recorded in Chapter 7 were not of sufficient accuracy. However, some data, in areas where test results have had to be extrapolated for design purposes, can now be incorporated to improve either safety or efficiency. Finally some factors will need to be reconsidered in this Chapter in the light of the overall design information. As a result of this work a comprehensive method of appraisal of wall resistance can be presented which will allow the maximum resistance of all structural walls in a building to be evaluated.

The second area, the summation of the wall components is more difficult because it depends on the allowable contributions from plasterboard and brickwork. Here a
number of alternatives will be considered and an intermediate solution based on an average viewpoint and a logical set of guidelines is carried forward. It is stressed that this situation is contentious and the approach adopted represents the author's viewpoint as to what is at present commercially and statutorily acceptable.

The performance of buildings is the end result of the design procedure but the additional factors that have to be considered at this stage are outside the scope of this thesis. Consequently they cannot be evaluated and are assessed in terms of their use and general effect only.

Finally a simplified method of wall design is proposed, based on the test results given in Chapter 6, which is intended as a less technical method of assessment which should be suitable for the majority of house designs allowing timber frame to be treated more equitably with its competitors.

8.2 WALL DESIGN

8.2.1 Philosophy

The overall racking resistance of a building at a given floor level is the sum of the components of the various structural walls acting in the direction of the applied wind load. To simplify the design procedure it is proposed that these walls be divided into the following four types:

(i) Structurally sheathed walls \(\text{WRL}_{SS}\), i.e. timber frame walls clad with an accepted wood based sheet material.

(ii) Separating walls \(\text{WRL}_{SP}\), i.e. timber frame walls clad principally with plasterboard but, if necessary, with secondary structural components.
added (such as diagonal bracing or wood based sheet material) to allow safe use of the plasterboard.

(iii) Plasterboard linings WRL_{PL}. These may be in the form of either additional contributions to structurally sheathed walls (e.g. provided by the internal lining to the external timber frame wall) or structural partition walls where the term structural is applied to the ability of the wall to resist racking forces primarily and NOT vertical forces.

(iv) Brick walls WRL_{BW}, which are directly attached to structurally sheathed walls.

It is clear that a fifth category of walls exists, i.e. non-structural. These walls will not contribute to racking resistance and if they are also non structural in terms of vertical load, then they can safely be altered without affecting the structural performance of the building.

The total resistance in the direction of the wind load, the floor level racking load (FRL) can then be found by summing the wall components in this direction, i.e.

\[
FRL = \xi_{WRL_{SS}} + \xi_{WRL_{SW}} + \xi_{WRL_{PL}} + \xi_{WRL_{BW}} - 8.1
\]

The advantage of this method is that restrictions to the use of plasterboard and brickwork can then be applied independently.

A typical timber frame wall will be clad with boards on both faces and in the case of external walls it is likely that the boards will represent different categories of wall, as noted above. It is therefore necessary to treat
them separately but because the combined performance is not the sum of the individual parts the wall will be considered to have a primary and secondary cladding. The primary cladding, which will be the sheathing in sheathing/lining combinations or the stronger board if both are either sheathings or linings, will be assigned its full capacity while the secondary board is considered to take the additional load resisted by the combination, i.e.

\[ WRL_{\text{secondary}} = WRL_{\text{combination}} - WRL_{\text{primary}} \]  
- 8.2

Using the design method noted in Chapter 4 the component parts can be calculated to be:

\[ WRL = DRR \times \text{Material} \times \text{Wall}_\text{primary} \times \text{Modification}_\text{primary} \times \text{Wall}_\text{board} \times \text{Modification}_\text{board} \]  
- 8.3a

\[ WRL = DRR \times \text{Material} \times \text{Wall}_\text{Secondary} \times \text{Modification}_\text{effect of secondary board} \times \text{Modification}_\text{secondary board} \]  
- 8.3b

They can be summed to find the maximum capacity of the wall, necessary to the calculation of base fixings, but may also be used independently to find the total capacity for each wall type. For a given wall the length and wall modification factors should be identical for the two boards but the material modification factors can be expected to be different.

The test results in Chapter 6 have been used to determine values for datum racking resistance and modification factors so that equations 8.3a and 8.3b could be evaluated. In general the values were analysed independently and before they can be accepted into a design procedure it is necessary to re-examine a few factors in the light of later results.
8.2.2 The Maintenance of Safety Factors
Throughout the Alternative Design Methods

The safety factors considered in this section are not directly those against overall failure because it has been shown in Chapter 6 that all the test results used in the determination of design values and modification factors have been checked for this case. Some concern is felt over the use of the test data to provide design values for a very much wider range of situations covering:

(i) uses of material which are either generically or structurally identified as being similar to those tested but which perform differently in practice,

(ii) the combination of materials modification factors each identified individually,

(iii) the extrapolation of the standard panel data to wall design based on a limited series of tests.

The two most significant results concerning the points above are firstly the variability in racking resistance recorded for similar grades of timber, due to the variability in density/nail holding capacity of the wood, and secondly that the stronger combinations of materials exhibit lower improvement factors when used in stronger situations, e.g. with reduced nail spacing, in longer lengths etc.

The datum racking resistance values include a factor of safety of approximately 22% for the four principal boards. The factor is slightly higher for plasterboard and will be much higher for the other boards placed into the two board categories because the principal wood based sheathings have been used to define the lower bounds of these categories. The additional safety factor covers all
the points considered above. The weakest combination of materials, based on very weak frame timber, have been shown to be safely modelled by the DRR value. The large safety factor allows the wall modification factors to be based on average performance levels (i.e. with no further safety included) safe in the knowledge that:

(i) stronger boards will have a high safety in DRR which outweighs the loss in safety in the wall modification factors,

(ii) weaker boards with low safety in DRR will gain in safety from using the material and wall modification factors.

There is still a slight worry concerning the use of materials modification factors and the additional racking resistance for the secondary board in that they could very significantly enhance the DRR value of the primary sheathing such that the design values for plain walls using the combined sheathing could be compromised. A method of limitation whereby the secondary sheathing is ignored if DRR for the primary board exceeded 2.50 kN/m was proposed in Chapter 6. Three alternative courses of action may also be considered.

a) The factors causing the greatest increase in the combined BRR value should be reduced. These are the datum racking resistance for the effect of the additional board and the nail spacing modification factor.

b) Reducing the improvement in the BRR value after a certain level is reached. This level could be taken approximately 50% above the DRR value for strong category sheathing, i.e. BRR = 2.5, with a correction factor based on the format:

\[
BRR = 2.5 + x(BRR_{primary} + BRR_{add.eff} - 2.5) - 8.4
\]
where the correction factor \( x \) would have a value in the region of 0.5. (The method stated in Section 6.4.2 is of similar form to this equation).

c) To do nothing noting that the most likely cause of the high value of BRR is the additional effect of plasterboard which will be seen later to be limited in its application by other means.

Option (a) is conservative, and will be unnecessary in many situations, and option (b) will be difficult to apply when the BRR value for the combination has to be considered as separate parts. The original proposal is easier to apply but is more conservative and is perhaps unnecessarily restrictive in view of the other safety factors built into design which cannot be evaluated and the very limited likelihood of the situation occurring in practice. Option (c) has therefore been adopted in the design.

The datum racking resistance and materials modification factors can be substituted in the design process by a test value for basic racking resistance. It is possible for this test result to represent the average performance of the material combination either by conducting a large programme of tests or by statistical chance if fewer tests are used. Although no safety factor has to be considered for the applicability of the results to different sheathing and fixings it will be necessary to consider:

(i) the extrapolation to wall design,

(ii) the possible changes in frame material and its effect on panel performance,

(iii) the possibility that the test panel will be of above average quality.

It is probable that tests will be conducted in cases where higher values than those of DRR are required, it is then
that the wall modification factors are compromised. Hence it is essential that a factor of safety be built into the test resistance to cover extrapolation of results and factors (ii) and (iii) above. The following equation is suggested.

\[ BRR_{\text{design}} = 0.87 \times BRR_{\text{test}} \]  

- 8.5

Clearly this reduces the benefits to be gained from testing but restores the safety margin and emphasises the status of the DRR values. It would be possible to extend the test method to allow the use of material modification factors but here a further safety margin would be needed which suggests the equation

\[ DRR_{\text{design}} = 0.80 \times \frac{BRR_{\text{test}}}{\text{Material Modification Factors Appropriate To The Test Case}} \]  

- 8.6

It should be noted that if tests are carried out on a singly sheathed panel the design value of BRR may be used to replace that of DRR multiplied by the material modification factors in equation 8.3. However, if tests are carried out on combinations of boards they are of no direct use unless they also cover the use of the primary sheathing alone. Then the BRR value for both the primary and the additional effect of the secondary sheathing can be calculated noting that an increased safety factor against failure may be necessary for the additional board in sheathing/lining combinations as noted in Chapter 5.

The third method of assessing wall performance is to test the wall itself. Here the safety factors built into the test will be adequate to ensure a satisfactory design load because the result may not be extrapolated to consider other types of wall.

Finally in examining the safety margins it is important to consider the base fixing design which could
also limit the capacity of the wall. It has been suggested that these fixings be designed for the maximum capacity of the wall based on the applied vertical load, i.e. it includes for full use of any plasterboard component. The design is based on BS 5268 part 2 (BSI, 1984) values for nail (or bolt) shear which are likely to include a greater factor of safety than those used in the calculation of wall racking resistance. The tests show that if the wall racking load is matched by the shear resistance of the base fixings then the wall capacity need not be reduced except in very short panels at zero vertical load where uplift is a major problem. It is noticeable that base fixing design is critical at zero vertical load where the shear capacity of vertically driven nails does not check the principal problem of uplift. However the results are in general adequate without the need for further checks and at higher vertical loads the additional factor of safety in the fixing design is noted as the wall performance can be shown to improve on the standard test case with bolted connections.

In Chapter 6 a proposal was given for a K116 factor to cover the problem of short walls. Later work on return walls indicated that, although they were unlikely to provide sufficient restraint to justify an improvement to racking performance their behaviour, especially on short walls, reduced the need to apply specialist reduction factors such as K116. Thus the factor is excluded from the final design method.

8.2.3 Material and Wall Modification Factors

Taken independently these factors were shown to be acceptable for the narrow range of variations allowed by the limits over which the factors may be applied. The use of these factors in combination can now be considered acceptable in view of the safety factors applied to datum and basic racking resistance values.
The wall modification factors determined in Chapter 6 can also be considered acceptable within the limits tested but as they have the effect of greatly extrapolating the standard panel test load and are often greatly simplified to meet the requirement of the design method they are examined in greater detail.

8.2.4 Vertical Load

The vertical load modification factors have been proven adequate over a wide range of tests and, if the length factor is included without restriction, they can safely be used to cover all practical lengths. However as the vertical load factor can greatly enhance the racking resistance of the wall it is necessary to consider what loads are applicable. Two cases are considered:

(i) applied loads,
(ii) equivalent loads due to the motivation of holding down forces.

The applied loads are due to self weight, other dead loads and the vertical load effect of the wind itself. Live loads such as snow etc. will have to be ignored and great care taken in the assessment of dead load. No design case has been considered for an overall uplift on a panel and this must be avoided by the provision of holding down straps which should be attached to the upper structure so that no tensile loads are applied to the timber frame. Equivalent vertical loads are provided by holding down straps connecting the wall studs to the substructure; the resistance may be calculated from the shear (or tensile) capacity of the connector but engineering judgement is necessary in assessing if that uplift can be motivated by the wall. Thus the influence of holding down straps may need to be reduced if the panel is subjected to a very large imposed load. Normally this will not be a problem because holding down straps are positioned at points of
greatest uplift and the equation for the modification factor takes account of reduction in effect at higher loads.

The equivalent vertical load factor may not be applied between vertical joints in panels in the plane of the wall but could take advantage of fixings to return walls if it can be shown that the return wall provides adequate resistance to uplift which is unlikely unless it is strongly held down to the substructure very close to its junction with the racking wall. As mentioned in 8.2.2, it is considered that the effect of return walls be ignored until data directly relevant to their use is made available.

8.2.5 Wall Length

The wall length modification factor is the major cause of enhancement of the well tested datum racking resistance. It must therefore be treated with considerable care. The test results allow confidence in the modification factor up to 4.8m length but the dangers in its extrapolation were noted in the results of the computer based analysis.

Based on the factors for K111 given in equation 6.36 four alternatives are considered for the modification factor after 4.8m.

a) To continue with the same equation i.e.

\[ K_{111} = \left( 1.6 - \frac{1.44}{L} \right) \]

- 8.7

This results in an increased racking resistance in longer walls and is definitely unsafe.

b) To use the racking resistance at 4.8m as a constant for longer walls i.e.

\[ K_{111} = 1.3 \]

- 8.8
This was proposed in Chapter 6 as a logical approach based on a uniform application of racking load but has since been noted as unsafe based on the results of the computer analysis.

c) To restrict the wall load to that at 4.8m whereby:

\[ K_{111} = 1.3 \frac{(4.8)}{L} \]

This approach is clearly too conservative because most walls fall outside the 4.8m length and will be reduced in capacity due to window openings.

d) To adopt an intermediate solution to (b) and (c) such that the resistance of walls reduces with length until a maximum load is reached. The modification factor is given by:

\[ K_{111} = 1.3 \left(\frac{4.8}{L}\right)^x \]

where \(0 < x < 1\) and \(x = 0.5\) is a satisfactory solution.

The results of the four approaches are shown in Figure 8.1. It has been decided to continue with solution (b) for design purposes for the following reasons.

(i) The computer based analysis in its present form does not accurately model wall behaviour.

(ii) Typical long external walls will be reduced in capacity by openings.

(iii) Typical long internal walls will be considerably reduced in capacity by door openings. (See section 8.2.7).
Typical long separating walls already include a high margin of safety. However in using this factor it may be necessary to reduce the effective length of the wall if, due to the aspect ratio of the horizontal diaphragm, a uniform application of load to the wall cannot be guaranteed.

8.2.6 Wall Openings

The wall opening modification factor proposed in Chapter 6 was a mean solution for the test results. However, it was based on the weakest method of sheathing around openings and the weakest layout of the opening i.e. many small openings breaking the wall up into short full height units. It may therefore be considered a lower bound solution. The sheathing problem can be overcome by the approach summarised in equation 6.58 where the window opening is reduced in effective area if the pillar sheathings are continuous above and below the window. (Figure 6.37 c). At present there is insufficient data to evaluate this factor. The location of openings can be overcome by allowing the wall to be designed in separate parts so that full advantage can be taken of longer lengths of plain wall panel. This alternative approach does not require any additional design rules but its benefit in certain situations needs to be made fully evident to the designer.

The wall opening modification factor assumes continuity of the structure above and below the opening (in the case of doors this continuity is supplied by the substructure); if this is not achieved the wall must be designed as separate structures either side of the break. This form of construction is particularly evident in non vertical load bearing internal walls where door openings are not bridged by a structural lintol. Figure 8.2 illustrates design requirements.
8.2.7 **Damage and Load Duration**

The modification factors for damage (wetted panels) and load duration discussed in Sections 6.8.2 and 6.8.3 form a third group of modification factors based on wall uses. As the damage factor $K_{114}$ is related to the type of board it must be considered together with the material factors because in the combined sheathing condition the factor may be different for the two boards. The load duration factor $K_{115}$ may be considered along with the wall modification factors but its use is noted to be very unusual because racking resistance relates primarily to wind loads.

8.3 **THE SUMMATION OF WALL LOADS**

8.3.1 **Introduction**

In equation 8.1 only the racking load attributable to the structural sheathings may be used in total in resisting applied load. The plasterboard walls and brick wall components will have to be analysed separately and in the majority of cases reduced to satisfy design restrictions which are necessary as a result of:

(i) the doubts concerning the structural reliability of plasterboard,

(ii) the traditional use of plasterboard as a non structural material,

(iii) the limited knowledge of the interaction of brick skin and timber frame walls.

The background to these restrictions was noted in Chapter 4.

8.3.2 **Plasterboard Lined Walls**

Plasterboard is used on three different types of walls in timber frame buildings viz:
(i) as a lining to external walls which are structurally sheathed on their outer face,

(ii) as a lining both sides of internal partitions,

(iii) as a double thickness lining to separating walls.

In external and separating walls the linings will be regarded with greater importance; they cannot be omitted, the frames to which they are attached are structurally important and will be properly attached to the horizontal diaphragms above and below, the overall quality of workmanship is more likely to be checked and damage will need to be quickly rectified. Internal partition walls, however, are treated with less respect. If they are not load bearing a reduced fixing requirement is allowed and they may be removed or repositioned without any need for structural checks. Thus before internal partitions can be considered in accordance with external wall linings and separating walls there must be a change in concept whereby all internal walls are designated as structural, unless stated otherwise on the drawings (and thus excluded from calculations), and both lining and frame connections should be governed by the minimum acceptable standards laid down for structurally sheathed walls. It must be noted that whatever the structural capacity of the internal walls they will be subjected in practice to very high racking loads due to their position in the building unless they can be isolated structurally from elements transmitting the wind load to the side walls.

The contribution from plasterboard linings may be analysed in a number of different ways viz:

(1) plasterboard is treated in an identical manner to structural sheathings and is fully contributory,
(ii) the total contribution of the plasterboard is reduced by a secondary safety factor, i.e.:

$$\text{Plasterboard} = (\sum_{i} WRL_{PW} + \sum_{j} WRL_{SW}) \times Kp$$  \hspace{1cm} - 8.11

where $Kp$ is a factor between 0 and 1.

(iii) the total contribution of plasterboard may be limited by that of the structural sheathings, i.e.:

$$\text{Plasterboard} = \sum_{i} WRL_{SS} \times Kp_f (\sum_{i} WRL_{PW} + \sum_{j} WRL_{SW})$$  \hspace{1cm} - 8.12

where $Kp_f$ is a factor between 0 and 1.

(iv) the total contribution is restricted by the type and position of walls, e.g. all internal walls could be omitted from the calculation of racking resistance.

(v) Cases (ii) to (iv) above could be combined in parts, e.g. the separating wall is considered fully structural and a partial contribution is allowed from the external wall linings.

Case (i) above is the practical condition which is needed to determine the true racking resistance of the building. At present it is not acceptable, due to worries over the use of plasterboard and the limited public confidence in timber frame housing, but it may be used to justify the overall strength of buildings which fail to comply with the design regulations. Case (ii) could be obtained by reducing the datum racking resistance levels for plasterboard and then allowing all walls to be fully contributory but this restricts the positioning of the load where as, if the
reduction is made to the overall contribution, the allowable plasterboard resistance may be divided amongst the available walls (up to their maximum capacity) so as to better balance the applied racking force. Logically this solution is poor because the worries over plasterboard concern damage to specific walls, but the factors of safety being built into the use of the board are now very high and if an essential wall is damaged only the horizontal moment on the building due to the wind (see section 8.4) will be increased. Case (ii) allows a plasterboard contribution without any requirement for structural sheathing. This is important in centre of terrace buildings where there are no external walls to resist face loading but would be unsatisfactory to many designers unless the value of Kp was so small as to make the benefits of the design method of little consequence.

Case (iii) cannot be used on its own due to the problems in terraced houses noted above. However if separating walls can be treated independently in a similar manner to structural sheathed walls then the method has many advantages. The arguments for its use, based on damage criteria, have been detailed in Chapter 4 and a value for $K_p^1$ of 0.5 has been derived.

Case (iv) is a logical approach based on the reliability of plasterboard in different locations. It can also be adapted into a much simpler design method whereby the sheathing components of the wall do not have to be separately totalled. The method is therefore suitable for the simplified design procedure allowing the structural use of only plasterboard on external walls to the dwelling unit. The overall safety can be altered as necessary by the choice of datum racking resistance values and the method has the advantage of not having to lay down any regulations concerning internal walls.

Case (v) allows many variations, but of greatest importance is the one at present adopted in the draft Code.
whereby separating walls are considered structural and independent of other plasterboard linings which are then analysed using case (iii) above. The separating wall can then be considered in two ways; firstly as a structurally sheathed wall whereby:

$$w_{WRL PW} \leq 0.5 (w_{WRL SS} + w_{WRL SW})$$

or as an independent wall such that

$$w_{WRL PW} \leq 0.5 \times w_{WRL SS}$$

The current thoughts of the Code of Practice drafting committee favour the former solution however the author prefers the latter in view of the substantial changes, already made in the use of plasterboard, over a short period of time, and the potential for criticism, even if unfounded, from the competing brick and blockwork housing industry. Thus it is proposed that separating walls be treated independently and are assigned a datum racking resistance as shown in Chapter 6. This value includes an increased safety factor which is also matched in test requirements. Design procedure would follow standard practice for plain walls. It is the author's opinion that separating walls could be lined with plasterboard alone, and do not need any additional restraint against racking force, for the following reasons:

(i) the thickness of the walls makes them less prone to damage,

(ii) the design values include a much higher safety factor,

(iii) traditionally separating walls are regarded as an important part of the structure and will be treated as such by the builder, the inspector and the householder.
Against point (iii) it is noted that the fixings of the first layer of board are hidden by the second board and thus attention should be drawn to this detail.

The concept for the design of separating walls does not depend on them being solely lined in plasterboard. If it were considered necessary to provide a further safeguard against damage one of the following clauses could be introduced,

(i) One layer of moisture resistant plasterboard should be included in the wall.

(ii) Full height diagonal bracing should be provided along the wall at a maximum of (say) 2.4m centres.

(iii) A 1.2m wide full height sheet of a strong category sheathing material should be provided at intervals no greater than 6.0m along the wall.

The additional material noted in (ii) and (iii) above would be considered part of the separating wall structure and would not be separately assessed; thus the racking resistance would remain as 0.9 kN/m.

The separating walls, although treated in the same manner as the structurally sheathed walls, are kept independent. In this way they cannot be used to enhance the structural use of plasterboard in internal walls which is related to the main structural sheathing only. The general design rules affecting the overall use of plasterboard are illustrated in Figure 8.3 which covers the design of terraced houses. The calculations can be seen to be clear and straightforward. The resulting design values should then be reasonably balanced about the centre line of the building unit which will reduce problems of
eccentricity caused by the applied wind load.

The final safety factor implicit in all the uses of plasterboard relates to the relationship between the datum (or basic) racking resistance and the test results. Here a greater factor of safety than normal (2.4) has been introduced to the results because of the brittle nature of the board which causes failure at relatively low loads. Because the design value for all plasterboard walls is invariably based on failure load it will be only 67% of that which would have been used with a wood based sheathing. Thus in the limiting case, a building which has few plasterboard walls such that they are all fully utilised to contribute the maximum allowable 50% racking resistance of the main sheathing, then the plasterboard contribution has an extra 1.5 safety factor compared with the main sheathing. In buildings with a higher proportion of plasterboard walls the factor of safety is increased as not all of the plasterboard is being used structurally.

It is notable that, due to the high stiffness of the plasterboard, the actual resistance of the board at the deflection limit of 0.003 times the panel height could be up to twice the design resistance of the board. Thus, in the building described above, the practical contribution of the plasterboard will be much higher, as noted in the Australian test results described in Chapter 4.

The arguments concerning plasterboard in this section refer to a standard grade of board. It is possible that more use will be made in future of moisture resistant board which, if it could be shown to have the same durability and secondary requirements as the wood based structural sheathings, could be used in a similar manner as an external wall linings together with ordinary plasterboard used internally. If it was then used internally it would not be subject to the restrictions noted above.
8.3.3 Brick Skin Walls

The brickwork has been shown (Chapter 6 and in work by Anderson*) to improve the racking resistance of timber frame walls depending on the type and density of wall ties. The improvement is independent of vertical load and the constitution of the timber frame wall. Resistances are known to be high in plain walls but no information is available concerning typical walls where the effects of length and opening could have been studied. Brick walls are optional exterior claddings and their location is independent of the structural sheathing. The information available allows a very conservative estimate for the brick wall performance to be included in the design, based on the lowest quality ties, their density and the total length of full height brickwork within a wall length. In order to cover peculiar design cases which could result in the brickwork contribution being high in respect to the structural sheathing an overall limit to the contribution of the brickwork must be set. This is directly linked to the structural sheathing on the timber frame wall to which it is attached, whereby:

\[
WRL_{BW} \leq WRL_{SS} \times K_B
\]  - 8.15

and must not be regarded as a general addition to racking resistance.

These methods of evaluation are at present included in BS 5268 part 6. The design values are based on a reputable source (Anderson*) and show a high factor of safety in comparison with the results of tests carried out as part of this investigation (Section 6.8). Both the design guidelines and the tabulated values are incorporated in the proposed design method detailed in Section 8.5.

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8.4 DESIGN OF BUILDINGS

8.4.1 Applied Loads

The horizontal loads causing wall racking are normally attributed to wind. The velocity of the wind and its effective pressure on a building may be calculated with reference to CP3 Chapter V Part 2: Wind Loading (BSI, 1972). In the design of detached houses the worst condition is likely to occur when the wind blows normal to one or other face of the building, the applied load can then be calculated using standard force coefficients. Depending on the method of analysis adopted this load will affect either all walls acting in the direction of the applied force or, if the building is torsionally rigid and the applied load creates a moment, all the walls of the building. If it is necessary to check a wind load which is not normal to either set of walls the load may be resolved into components normal to the wall and the standard analysis completed. If the building is torsionally rigid the resolved forces must be considered together, otherwise they can be analysed independently.

In terraced houses, due to the structural break between properties it is advisable to use internal and external pressure coefficients rather than the more simple force coefficient for the reasons shown in Figure 8.4. A further use for external and internal pressure coefficients is in the calculation of roof uplift forces which could reduce the vertical load on a wall and thereby its racking resistance.

In Chapter 4 it was noted that not all the load applied to the building will reach the timber frame. The reduction in applied load is due to the shielding effect of the brick outer skin when it is used as an essentially non structural cladding to the timber frame to provide a traditional appearance to the building. At
present it is not possible to quantify the effect of shielding although unpublished work at the Building Research Establishment and the Polytechnic of the South Bank suggest that it could account for a high percentage of the applied load.

However, one method that is occasionally used by designers to reduce applied load, and is representative of shielding is to take account of the stiff corner effect of brick walls. Here the building corners are considered to be so stiff that their movement is insufficient to allow transfer of load on the face wall into the timber frame through the brick ties. The width of the face wall assumed to carry the applied load direct to the foundation is dependent on the width of full height wall of both the face and side walls at the corner and is also given an overall limitation. A typical set of guidelines, based solely on engineering judgement, is shown in Figure 8.5. As this procedure has not been justified by either test or theory and in many cases cannot be used, because there is no brick cladding, it will not be considered further in this investigation.

Once the applied load on a building has been calculated it may be distributed to the horizontal diaphragms as shown in Figure 8.6 c. In normal construction, where vertical studs support the sheathing, the face wall is considered to span between horizontal diaphragms and thus no load is transferred direct to the side wall. The total load in the racking walls at a given storey level is the sum of the loads in the horizontal diaphragms above that level. Thus the highest racking load will be in the ground floor panels. However, it may also be necessary to check the racking capacity of upper storeys due to the effects of vertical load and openings.

The line of action of the applied load at a given storey may be based on the summation of the effects of the applied load at each diaphragm level. This assumes
the loads to have been adequately balanced at each storey but allows for the design of non symmetrical face walls.

The applied load can then be checked against the total capacity of the structural racking walls at that level using equation 8.1 but including restrictions on the use of plasterboard and brickwork. Here the terms $\varepsilon_{WRL_{SS}}, \varepsilon_{WRL_{SW}}$ and $\varepsilon_{WRL_{BW}}$ are fixed in both magnitude and position but the term $\varepsilon_{WRL_{PL}}$ is known only in magnitude and can be distributed amongst the plasterboard walls as required, with the limitation that each wall is restricted by its maximum capacity.

8.4.2 Moment Considerations

In the majority of designs it will be adequate to prove that the total racking resistance of the walls exceeds the applied load and to check by visual inspection that the balance of the resistive forces about the line of action of the applied load is acceptable. This approach assumes that small out of balance moments can be resisted firstly in torsion by the horizontal diaphragm and secondly by the racking resistances of the walls at right angles to the applied load.

If the out of balance force has to be calculated the following procedure may be used:

a) Starting from the left hand wall of the building the maximum allowable load is applied to each wall in turn until the plasterboard component $\varepsilon_{WRL_{PL}}$ has been distributed in total.

b) Moments are taken about the right hand wall comparing the resistive moment of all wall components with the moment created by the load.

c) Procedures (i) and (ii) are repeated applying the plasterboard component from the right hand side and
taking moments about the left hand wall.

d) If in both cases the resistive moment exceeds the applied moment, the plasterboard resistance can then be distributed so as to accept the applied load without inducing out of balance moments in other walls.

e) If in one case the applied moment exceeds the resistive moment then the difference is the minimum possible out of balance moment. This must be checked in terms of the torsional resistance of the diaphragm and the restoring couple set up by racking resistance in the face walls of the building. If the induced racking load can be accepted by the face walls then the racking resistances of the side walls are adequate to withstand the applied loads.

A more rigorous approach to the distribution of racking resistance amongst the walls is not essential to the current investigation which represents standard design practice.

8.4.3 Inadequate Resistance

A number of timber frame designers have noted that the procedure given in the draft Code of Practice (BSI, to be published) is unable to prove the adequacy of narrow fronted houses with many face openings when the wind blows on the side wall although they have stood safely for many years and could be shown to be adequate using previous design methods (TRADA 1980 b). Where such buildings are detached units it is probable that the design method is correct in showing their inadequacy. However it can be accepted that the buildings have not shown the results of undue stress due to the inherent safety factors built into the design. These include:
(i) the general factor of safety against failure,

(ii) the safety factor (probably greater than 20%) included in the value for DRR,

(iii) the lower bound interpretation of wall sheathings,

(iv) the restricted use of plasterboard,

(v) the restricted use of brick wall resistance,

(vi) the shielding effect of brickwork,

(vii) box effect of the timber frame.

Points (i) to (vi) have all been detailed in the design section. The final point relates to two functions; firstly the effect of return walls (both external and internal) on the wall providing racking resistance and secondly the effect of internal members of the building, such as staircases, improving overall rigidity. Unpublished tests carried out by the Building Research Establishment imply that the box effect is substantial. The author is unable to comment on that work in relation to the current investigation but doubts the significance of the box effect. The effect of return walls has been analysed, and can be covered by the current design procedure. It is unlikely to provide much extra resistance unless the walls carry a low vertical load. The additional strengthening from internal elements is likely to be very variable, based on their location and extremely difficult to quantify.

It is not logical therefore to base an enhancement factor on the box effect alone. Such a factor is acceptable as an interim measure if it is used to cover all the unknown
safety factors although even then its use is doubtful because many of the factors can already be overcome, e.g. by using basic racking resistances from tests and the stiff corner effect. Fortunately the loss of factors (v) and (vi) in a building without brick walls would be compensated in part by the additional external cladding to timber frame walls.

The solutions to the problem of underestimating the design capacity of timber frame walls are:

(i) to apply a 10% construction factor as an interim measure,

(ii) to take full advantage in design of all restraints on the timber frame wall,

(iii) to consider the effect of constructional continuity in terraced construction which will immediately solve the major difficulty in house designs in the narrow fronted terrace house,

(iv) to improve the quality of plasterboard linings so that more advantage can be taken of the resistance they already offer,

(v) to evaluate the effect of brick shielding so that a simple factor such as the stiff corner effect can be included in the design procedure.

This concludes the introduction to building design. In the following section the design rules are laid out for the standard and simplified design procedures. Only those factors that can be substantiated by test data have been included, thus many of the points discussed in Chapters 6 and 8 have had to be omitted. Their inclusion would then be left to the discretion of the designer.
8.5 THE EVALUATION OF WALL RACKING LOADS

8.5.1 General

Wall design is based on the principal cladding attached to the timber frame; in sheathing/lining combinations this is the sheathing and in combinations of sheathings or linings it is the stronger board. Using the standard notation its wall racking load is calculated to be either:

\[ WRL = DRR \times L \times \text{Material \times Wall \times Use} \]  
\[ \text{(K101-K104) (K110-K113) (K114-K115)} \]  
or

\[ WRL = BRR \times L \times 0.87 \times \text{Wall Factors} \]  
\[ \text{(from tests) (K110-K113)} \]  

If a second board is fixed to the timber frame its racking load component is found by applying the datum racking resistance for the additional contribution of the board in equation 8.16. Equation 8.17 may be substituted only if the basic racking resistance contribution of the secondary board has been calculated from tests on the primary and combined claddings.

The maximum racking capacity of the wall is found by summing the components such that:

\[ WRL_{\text{max}} = WRL_{\text{primary}} + WRL_{\text{additional effect of secondary board}} \]  

The design load assigned to the wall will depend on restrictions on the use of plasterboard applied later in the procedure.

8.5.2 Datum Racking Resistance

The datum racking resistance is the safe design resistance for a sheathing or lining board when used in
a standard combination of materials, as defined by the type of board, in a 2.4m square plain wall panel. Values are quoted for four groupings of boards as shown in Table 8.1 noting the following standards that are implicit to the use of DRR in wall design.

a) Materials

(i) Board Type and thickness : as given in Table 8.1

(ii) Nail size and spacing : as given in Table 8.1

(iii) Frame Material : Strength class 3 timber with a minimum density of 425 kg/m³

(iv) Frame Sizes : The minimum size for all frame timber is 38 x 72mm allowing two boards to be joined on the narrow face.

(v) Frame Geometry : Studs at centres not exceeding 610mm should be square cut and fixed by, at least, 2 no. 3.75mm dia. 75mm long wire nails to the head plates and sole plates.

(vi) Sheathing Geometry : Boards may be laid vertically, or horizontally if 1.2m wide, and must be supported and fixed on all edges.

b) Panel Joints

Panels should be vertically joined using either:
(i) 3 no. M12 bolts spaced along the joint at 1.0m minimum centres.

(ii) 3.75mm diameter nails at least 75mm long at 300mm maximum centres.

Additionally the headplates of the panels should be joined by a structural member overlapping the joint by at least 600mm and fixed to the panel with 3.75mm diameter nails at 300mm centres.

Base fixings will need to be designed in shear to meet the maximum racking capacity of the wall determined in equation 8.18 but will not be less than the equivalent of 3.75mm diameter nails at 300mm centres.

c) Environmental

Panels must be dry; defined by a moisture content in the frame timber of less than 18%. They should have been protected and kept dry during storage but during construction, when in the upright position, they may be exposed to rain so long as their faces are protected and water is not allowed to pond. In this condition the moisture content of the timber should not exceed 18% for a prolonged period.

8.5.3 Basic Racking Resistance

The basic racking resistance of a given board type is the resistance appropriate to a 2.4m square plain panel for a combination of materials, all within set limits, it may be calculated using the following equation.

\[ \text{BRR} = \text{DRR} \times \text{Material Modification Factors} \]

\[ \text{BRR} = \text{DRR} \times \frac{\text{Material Modification Factors}}{K_{101} - K_{104}} \]

The limits for the variables are set by the modification factors.
Basic racking resistance can also be determined using the standard panel test described in Chapter 5. The design value is given by the equation

\[ BRR = 0.87 \times BRR_{\text{test}} \]

It may only be used for the same combination of materials as those tested and special care should be taken to ensure that the test materials are below the average quality of those used in practice.

8.5.4 Modification Factors

a) Materials Modification Factors

\( a_1 \) Applying to category 1 and 2 sheathings only and not for use with plasterboard.

**Variation in nail size:** For variations in nail diameter between 2.25mm and 3.75mm diameter the nail size modification factor \( K_{101} \) is given by:

\[ K_{101} = \frac{\text{Proposed nail diameter}}{3.00} \]

Nail length is determined by the required pointside penetration set out in BS 5268 part 2.

**Variation in nail spacing:** The nail spacing modification factor \( K_{102} \) is given by:

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

where \( A = \frac{\text{Proposed Perimeter Spacing}}{\text{Perimeter Spacing noted in Table 8.1}} \), such that \( 0.5 \leq A \leq 2.0 \)

\( a_2 \) Applying to all sheathings and linings.
Variation in board thickness: the board thickness modification factor $K_{103}$ is given by:

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

where $B$ = Proposed Board Thickness

Standard Board Thickness (noted in Table 8.1)

such that $0.7 \leq B \leq 1.3$

Variation in board orientation: no modification factor is required if sheathings/linings are supported on all edges, but if 1.2m boards are laid horizontally and are not supported along the central joint the board orientation factor $K_{104}$ must be used where:

$$K_{104} = 0.9 \frac{0.9F}{(K_{110})^{0.5}}$$

b) Wall Design Modification Factors

These factors apply to all sheathings and linings.

Vertical load: the modification factor for uniformly distributed vertical load $K_{110}$ is given by:

$$K_{110} = 1 + (0.09F - 0.0015F^2)(\frac{2.4}{L})^{0.4}$$

where 'F' is the uniformly distributed load and 'L' is the wall length.

Stud loads can be converted to uniformly distributed load, allowing the use of the $K_{110}$ factor by applying the equation:

$$F = \frac{V}{\text{Stud Spacing}} + \frac{V}{L}$$

where 'V' is the load in kN/stud
The factor may also be used with concentrated loads if they are converted using the formula:

\[ F = \frac{2a F_p}{L^2} \]  

- 8.27

where \( F_p \) is the total concentrated load and \( a \) is the distance of its line of action from the leeward edge of the wall.

A concentrated load can be assumed to be developed by vertical connections directly linking the wall panel studs with the substructure and foundations.

**Length**: the modification factor for length \( K_111 \) is given by:

\[
K_{111} = \begin{cases} 
\frac{L}{2.4} & \text{if } L \leq 1.45 \text{ m} \\
1.6 - \frac{1.44}{L} & \text{if } 1.45 < L \leq 4.8 \text{ m} \\
1.30 & \text{if } L > 4.8 \text{ m}
\end{cases}
\]  

- 8.28

where \( L \) is the wall length.

**Openings**: the modification factor for openings \( K_{112} \) is given by:

\[ K_{112} = (1 - 1.3p)^2 \]  

- 8.29

where \( p \) = Area of Openings in a Wall

\[
\text{Total Area of the Wall}
\]

The recommendation for openings are valid provided that:

1. all edges are supported by members at least as thick as the studs,
2. a means of transferring horizontal forces above and below openings is provided; where no such provision is made the wall must be designed as separate lengths either side of the opening.
Where an opening is less than 300mm from the corner of the wall and the depth of opening is greater than half the panel height then the length of that part of the wall, up to and including the opening should be disregarded when determining the total length of the wall.

Where two framed openings are separated by less than 300mm and the heights of both openings are greater than half the panel height then the area of openings should be taken as that of the rectangle which encloses both openings (Figure 6.2).

Small openings less than 250mm in either diameter or maximum length need not be framed or considered in the calculation of area lost to openings provided that:

(i) the clear distance between openings is not less than the greatest dimension of the openings,

(ii) the clear distance between the edge of the sheathing and the opening is not less than the greater dimension of the opening,

(iii) not more than one such opening occurs in any one 600mm width of sheathing.

The method of assessing the effect of wall openings takes account of the worst case of openings in a timber frame wall. Where higher values of racking resistance can be obtained by considering the wall as the sum of two or more separate parts then that approach should be adopted.

**Height** : the modification factor for height K113 is given by:

\[
K113 = \frac{2.4}{\text{Proposed Wall Height (m)}} - 8.30
\]
c) Wall Use Modification Factors

These factors are appropriate to all sheathings and lining.

**Damaged panels**: where category 1 sheathings have suffered minor damage due to wetting or knocks during construction the modification factor $K_{114}$ should be applied where:

$$K_{114} = 0.75$$

The modification factor must not be used with category 2 boards or plasterboard linings or with walls that have been subjected to prolonged wetting. Nor is the factor appropriate to the wall in the wetted condition. In these cases the value of $K_{114}$ is zero and when the board cannot be expected to recover the damaged area should be replaced.

**Load duration**: for loads other than short term wind loads the racking resistance must be multiplied by the load duration modification factor for timber frame walls $K_{115}$ which is given by:

$$K_{115} = \frac{K_3}{1.50}$$

where $K_3$ is the standard load duration factor given in BS 5268 part 2 (BSI, 1984).

8.5.5 **The Contribution of Plasterboard to Racking Resistance**

Plasterboard may be permitted to contribute to the racking resistance of a building if:

1. it is fixed in accordance with the requirements noted in Table 8.1,
(ii) the walls are fully supported throughout their whole length and are connected to the supports in such a way as to ensure the transfer of applied shear forces.

The plasterboard contribution may come from two separate sources and must be individually calculated, they are:

a) Separating Walls.

Timber frame separating walls are lined with two layers of plasterboard in accordance with Table 8.1. They are designed in a similar manner to structurally sheathed walls and no limitation is placed on their contribution. Separating walls are considered independent of structurally sheathed walls in the calculation of other plasterboard lining contributions.

b) Linings to External Structurally Sheathed Walls and to Internal Partitions.

The contribution of plasterboard as a lining to an external wall may be calculated using the datum racking resistance for the additional lining contribution, noted in Table 8.1, and by following the standard procedure.

The contribution of plasterboard in a partition wall may be calculated using the datum racking resistance for plasterboard plus the additional racking resistance for the second board. The standard design procedure is followed except that the continuity of the wall at joints with openings must be carefully checked. If the continuity is lost, for instance when a proper lintol is omitted, then the wall should be designed as a line of separate full height plain panels.

The total contribution of the plasterboard when used as a lining must not exceed 50% of the total racking
resistance of the structural sheathings acting in the same direction. Once the allowable plasterboard contribution has been determined it may be distributed amongst the appropriate walls as required so long as the calculated maximum contribution of the plasterboard in any wall is not exceeded.

8.5.6 The Contribution of Masonry Veneers to Racking Resistance

Masonry walls may be permitted to increase the racking resistance of an external structurally sheathed timber frame wall if the following conditions are observed.

a) Wall ties, with appropriate fasteners, have a minimum design horizontal shear strength of at least 150N at deformations of 5mm or more and a characteristic horizontal shear stiffness of not less than 30N/mm over the deformation range 0-5mm (when tested in accordance with Appendix A of BS 5268 part 6 draft for publication (BSI, to be published)).

b) The additional racking resistance applies only to those parts of the masonry wall that are the full storey height (normally 2.4m), are at least 600mm long, are backed by storey height timber frames and are tied to the timber with wall ties at a density not less than 3.7 ties/m².

c) The maximum contribution of the masonry veneer must not exceed 25% of the permissible racking resistance provided by the structural sheathing (i.e. excluding plasterboard lining) on the timber frame wall to which it is attached.

d) Thus the masonry component is calculated using the formula:

\[ W_{RLBW} = BRR_{BW} \times L_B \]

where \( BRR_{BW} \) is the basic racking resistance of the brick ties as given in Table 8.2 and \( L_B \) is the total length of
brickwork in a wall that complies with (b.) above, with the limitation that:

\[
WRL_{BW} \leq 0.25 \times WRL_{SS}
\]

for the same wall.

8.6 **THE SIMPLIFIED DESIGN OF TIMBER FRAME WALLS**

8.6.1 **Introduction**

The simplified design procedure is based on that of the standard method but the complexity is reduced by limiting the number of variables. The relationship between modification factors and wall variables is simplified and, where possible, equations are replaced by tables. As a result of the increased simplicity, design efficiency is lost; thus where buildings cannot be proven by the simplified method it will be necessary to revert to the standard procedure.

The modification factors can be divided into the same three groups used in the standard method, i.e. materials, wall design and wall use. In the first group the number of variables is considerably reduced by restricting the use of materials. Only the standard thickness of board is allowed, this can be laid either vertically or horizontally but must be fully fixed along all edges so that the board orientation factor can be eliminated. Nails are restricted to three sizes defined in BS 5268 part 2 (BSI, 1984) although intermediate and larger sizes can be used without enhancement in performance. Nail spacing is limited to three conditions and no allowance is given for spacings greater than the standard value. The wall design factors have already been discussed in Chapter 6 and are based on a less complex analysis of the full scale test results. The wall use factors for load duration and wetted panel factors are omitted, thus the method applies only to wind loading on perfect panels. They are replaced by new factors which cover the use of plasterboard and brickwork which will substantially
reduce the complexity of the calculation of building racking resistance, although in many cases this will remove the complexity of the contribution of these materials. By calculating the plasterboard contribution of linings (excluding separating walls) through the use of modification factors the table for datum racking resistance can be substantially reduced. The case where two sheathings of the same board are used can also be omitted as it is not common in practice. Finally the datum racking resistance may not be substituted by test results which will standardise the design and make checking easier.

The changes in the rules regarding plasterboard have the effect that only external walls to the buildings contribute to racking resistance. In normal circumstances it should be adequate to check that the combined resistance of the walls exceed the applied load along its line of action. If it is necessary to calculate the torsional moment and its effect on the face walls this can be very easily undertaken since the magnitude and location of all the forces will be known. As a final aid to simplifying design the process of determining the applied wind load could be reduced by restricting the variables given in CP3 Chapter V Part 2 (BSI, 1971).

8.6.2 Design Values and Modification Factors

In presenting the modification factors, guidelines to their use are only noted when they vary from those detailed for the standard design method.

a) Datum Racking Resistances

Datum racking resistance values are given in Table 8.3 and define the standard use of the boards. Because only one board thickness is allowed it is necessary to detail the minimum thickness in Table 8.3 rather than the nominal value previously used.
b) Materials Modification Factors

No modification factors for board thickness, board orientation or frame material are required.

Variation in nail size: values for modification factor K201 are tabulated below:

<table>
<thead>
<tr>
<th>Nail diameter (mm)</th>
<th>Value of K201</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.65 or greater</td>
<td>0.88</td>
</tr>
<tr>
<td>3.00 or greater</td>
<td>1.00</td>
</tr>
<tr>
<td>3.35 or greater</td>
<td>1.11</td>
</tr>
</tbody>
</table>

Variation in nail spacing: values for modification factor K202 are tabulated below:

<table>
<thead>
<tr>
<th>Nail Spacing (mm)</th>
<th>Value of K202</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category 1 boards:</td>
<td>Category 2 boards</td>
</tr>
<tr>
<td>150/300</td>
<td>75/150</td>
</tr>
<tr>
<td>100/200</td>
<td>50/100</td>
</tr>
<tr>
<td>75/150</td>
<td>-</td>
</tr>
</tbody>
</table>

b) Wall Design Modification Factors

Variation in vertical load: values for K210 are calculated using the equation:

\[ K210 = 1 + 0.06F \]

where 'F' is the uniformly distributed load (kN/m)

Concentrated loads may be included using the equation:

\[ F = \frac{2a_Fp}{L^2} \]
where $F_p$ is the total concentrated load, 'a' is the distance of its centre of action from the leeward edge of the wall and 'L' is the wall length. For stud loads at standard 0.6m centres the following equation can be substituted:

$$K_{210} = 1 + 0.112V$$

where 'V' is the stud load (kN)

Variation in length: values for $K_{211}$ are calculated using the following equations:

$$K_{211} = \begin{cases} 
1.6 - \frac{1.44}{L} & \text{if } 1.0 < L < 4.8 \\
1.30 & \text{if } L \geq 4.8 
\end{cases}$$

where 'L' is the wall length (m).

Variations due to openings: values of $K_{212}$ are calculated using the following equation:

$$K_{212} = \left( \frac{\text{Total length of full height panel in wall}}{\text{Total length of wall}} \right)^2$$

**c) Wall Use**

The design covers only short term loads on undamaged panels.

**Plasterboard lining**: where an external wall sheathed on the outer face with a structural board as defined in Table 8.3 is lined with 12.5mm plasterboard nailed at 150mm centres with 2.65mm diameter plasterboard nails the $K_{207}$ modification factor can be used as given below:

<table>
<thead>
<tr>
<th>Main Sheathing</th>
<th>Value of $K_{207}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category 1 boards</td>
<td>1.15</td>
</tr>
<tr>
<td>Category 2 boards</td>
<td>1.40</td>
</tr>
</tbody>
</table>
Masonry Claddings: where an external wall is clad in masonry which is tied to the timber frame with a minimum quality of tie (previously defined) at a density of not less than 3.7 ties/m² then the following equation may be used to determine the K208 modification factor:

\[ K_{208} = \frac{1.25 \times L_B}{L} \]

where \( L_B \) is the total length of full height brick wall, in lengths at least 0.6m long, which is attached to a full height timber frame.

8.6.3 Base Fixings

The design for basic fixings relates to the use of 3.75mm nails driven vertically through the bottom rail of the panel into an adequately tied down soleplate. Other fixings may be substituted if an adequate shear resistance (and resistance to uplift rotation) can be proven. The resistance of each 3.75mm nail is calculated to be 0.44 kN, assuming SC3 timber and using the short term load duration factor K48 (BS 5268 part 2). Then the nail spacing required in a wall is given by the equation

\[ \text{Nail Spacing} = \frac{0.44 \times L}{WRL} \]

where WRL is the wall racking load determined using the datum racking resistance, the panel length and the appropriate modification factors, but excluding K208, e.g.:

\[ WRL = DRR \times L \times K_{201} \times K_{202} \times K_{207} \times K_{210} \times K_{211} \times K_{212} \]

The checking procedure for buildings is now greatly simplified since internal walls have been excluded and the external walls have fixed racking values because the moveable plasterboard component has been omitted. The simplest check relates to rigid beam theory and treats loads in the 'x' and 'y' direction separately; the method is outlined
in Figure 8.7. The more complex rigid diaphragm theory can also be used if the rigid beam theory fails but the following equations are satisfied:

\[
\begin{align*}
V_x & \leq R_B + R_F \\
V_y & \leq R_L + R_R
\end{align*}
\]

The method of analysis is examined in Figure 8.8 in general terms and may be considered for either the first floor load or the more onerous ground floor load. The calculations may be applied either the independent worst loading case, such that when checking \(V_x\) then \(V_y\) is zero etc., or to the worst combined loading case, as would be applicable to end of terrace buildings.

The assessment of the applied loading presents a more significant problem to this method of analysis because it cannot readily be simplified in keeping with the rest of the procedure. Simplifications can be applied to the calculation of the dynamic wind pressure \(q\) (CP3 ChV Part 2) by restricting the variables. These could include:

1. A location map related to basic wind pressure instead of wind speed using the conversion factors in Table 4 of CP3 CLV Part 2 (see Figure 8.9).

2. A modification factor \((S_{11})\) for topography \((S_1)\) as used in the Code (see Table 8.3).

3. A modification factor \((S_{12})\) for ground roughness based on factor \(S_2\) in the Code but considering Class B only and a height of 10m. The variables may then, for convenience, be reduced to two (see Table 8.4).
(iv) A force coefficient $C_f$ based on the TRADA report (1980 b) but adapted to cover buildings with a more square aspect ratio (see Table 8.5) This could be incorporated as modification factor $S_{13}$ such that:

(v) The applied wind pressure may be calculated to be

$$W(\text{kN/m}^2) = \text{basic wind} \times S_{11} \times S_{12} \times S_{13}$$

The calculated applied wind pressure in the principal building directions would be sufficient for the design of single units. However the special case of the wind blowing on the face wall of an end of terrace building with discontinuities along its length requires the suction on the side wall to be evaluated. In view of the conservatism attached to the applied wind pressure, and the figures quoted in Table 7 of CP3 CHV Part 2 the side wall pressure may be taken to be a suction of $W/2$ kN/m².
Figure 8.1 The Extension of the Length Modification Factor to Walls Longer than 4.8m

\[ K_{III} = \begin{cases} 
1.6 - 1.45 \frac{L}{L} & \text{(I)} \\
1.3 & \text{(II)} \\
1.3 \left(4.8 - 0.5 \frac{L}{L} \right) & \text{(III)} \\
1.3 \left(4.8 \frac{L}{L} \right) & \text{(IV)} 
\end{cases} \]

Figure 8.2 Racking Resistance of an Internal Wall With Discontinuities Due to Door Openings

\[ \text{WAL} = \text{BRR} \times x \times \ell^3 \left( L \times K_{110} \times K_{111} \times K_{113} \right) \]

where \( \ell \) is the length of full height sheathed wall
Racking resistance contribution of walls in wind direction

(a) Centre of Terrace

- Structural sheathing = zero
- Separating walls = WRL(1) + WRL(2)
- Plasterboard (0.5 x structural sheathing) = zero

Total = WRL(1) + WRL(2)

thus wall 4 makes no contribution

(b) End of Terrace

- Structural sheathing = WRL 5
- Separating walls = WRL 3 + WRL 4p
- Plasterboard = WRL 5 + WRL 4p

Total = 1.5 WRL 5 + WRL 3

The plasterboard contribution is shared between walls 5 and 6p - but must not exceed their maximum allowable contribution. The plasterboard contribution must be reduced in the unlikely event that:

\[ \text{WRL}_5 + \text{WRL}_{6p} < 0.5 \text{WRL}_5 \]

*The brick wall contribution does not affect the argument concerning plasterboard but would increase the end of terrace racking resistance.

Figure 8.3 The Effect of Plasterboard Design Proposals on a Terraced House
Figure 8.4 Wind Loading on a Terraced House

(a) Wind on end wall (plan view)

Notes

(i) Wind pressure affects both end walls \(E_L, E_R\) but due to discontinuities the pressure on \(E_L\) causes racking in walls \(A_R, A_F\) only whilst the suction on \(E_R\) racks \(C_R, C_F\) only.

(ii) The end wall loads can only be calculated independently using internal and external pressure coefficients.

(iii) The wind suction on the front and rear walls should be equal and opposite so that no direct racking load is applied to the side or separating walls.

(b) Wind on face wall (plan view)

Notes

(i) Wind pressure affects the front and rear walls. As pressures on \(A_R, A_F\) sum to cause racking in walls \(E_L, SWA\) the external pressure coefficient can be used.

(ii) Wind suction on end walls \(E_L, E_R\) cannot be balanced due to structural discontinuities.

(iii) Racking loads on \(A_R, A_F\) are related to the load on end wall \(E_L\) which must be calculated using internal and external pressure coefficient.
wind pressure \( C_f \cdot q \) acting on wall height \( h \)

Load on timber frame not using effect of staff corner

\[
C_f \cdot q \cdot h \cdot x \cdot L
\]

Load on timber frame invoking stiff corner effect

\[
C_f \cdot q \cdot h \cdot (L-x_1-x_2)
\]

Shielded length \( x \) is limited by:

(i) \( a \)

(ii) \( b \times 2 \)

(iii) \( 2.0 \text{m} \)

Figure 8.5 The Stiff Corner Effect of The External Brick Wall
Figure 8.6 The Distribution of Horizontal Load Through a Typical House
<table>
<thead>
<tr>
<th>Board Thickness</th>
<th>Fixing</th>
<th>Principal Board on Timber Frame Wall</th>
<th>Additional Contribution of Secondary Board on Timber Frame Wall</th>
<th>Plasterboard or Category 2 Sheathing</th>
<th>Category 1 Sheathing</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CATEGORY 1 SHEATHINGS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.5mm Plywood</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.0mm Mediumboard</td>
<td>3.00mm diameter wire nails at least 50mm long</td>
<td>1.68kN/m</td>
<td>0.27kN/m</td>
<td>0.84kN/m</td>
<td></td>
</tr>
<tr>
<td>12.0mm Chipboard (Type III)</td>
<td>Maximum spacing 150mm on perimeter</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.0mm Tempered Hardboard</td>
<td>Maximum spacing 300mm internal</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>CATEGORY 2 SHEATHINGS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.5mm BIIB</td>
<td>3.00mm diameter wire nails at least 50mm long</td>
<td>0.90kN/m</td>
<td>0.45kN/m</td>
<td>see note</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Maximum spacing 75mm on perimeter</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>150mm internal</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>LININGS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.5mm Plasterboard</td>
<td>2.65mm diameter plasterboard nails at least 40mm long</td>
<td>0.90kN/m</td>
<td>0.45kN/m</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Maximum spacing 150mm.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>SEPARATING WALLS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>31.5mm plasterboard (1st layer 19mm plus 2nd layer 12.5mm) or equivalent</td>
<td>2.65mm diameter plasterboard nails at least 60mm long and at 150mm spacing in each layer</td>
<td>0.90kN/m</td>
<td>-</td>
<td>-</td>
<td></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 8.1 - Datum Racking Resistance Values for Standard Sheathings and Linings
<table>
<thead>
<tr>
<th>Minimum Tie Density (ties/m²)</th>
<th>Basic Racking Resistance for Brickwork BRRBW (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.4</td>
<td>0.5</td>
</tr>
<tr>
<td>3.7</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Table 8.2 Basic Racking Resistance Values for Brickwork

<table>
<thead>
<tr>
<th>Sheathing 1</th>
<th>Fixing Type</th>
<th>Fixing Spacing (mm)</th>
<th>Datum Racking Resistance (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lining Thickness (mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CATEGORY 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.5 Plywood</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(8.7 min thickness)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.0 Mediumboard</td>
<td>1.</td>
<td>3.00mm dia. wire nails at least 50mm long</td>
<td>1.68</td>
</tr>
<tr>
<td>(8.3 min thickness)</td>
<td></td>
<td>150/300</td>
<td></td>
</tr>
<tr>
<td>12.0 Chipboard</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(11.6 min thickness)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.4 Tempered hardboard</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(5.9 min thickness)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CATEGORY 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.5 Bitumen imp. insulation board</td>
<td>1.</td>
<td>3.00mm dia. wire nails at least 50mm long</td>
<td>0.90</td>
</tr>
<tr>
<td>(12.3 min thickness)</td>
<td></td>
<td>75/150</td>
<td></td>
</tr>
<tr>
<td>SEPARATING WALLS</td>
<td>2.</td>
<td>2.65mm dia. plasterboard nails at least 60mm long in each layer</td>
<td>0.90</td>
</tr>
<tr>
<td>30/32 Plasterboard</td>
<td></td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>(First layer 19 plus second layer 12.5) or equivalent</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes
1. Modification factors allow a change in nail size.
2. No change allowed.
3. Modification factors allow a reduction in spacing.

Table 8.3 Datum Racking Resistance Values for the Simplified Design Method

-578-
Note: $V_x$ and $V_y$ are normally considered as independent worst loading cases in the principal building directions.

First Floor Load

- $V_{FX} = V_x (A_R + \frac{h_f}{2} L)$
- $V_{FY} = V_y (A_R + \frac{h_f}{2} B)$

Ground Floor Load

- $V_{GX} = V_x (A_R + \frac{2h_f + h_g}{2} L)$
- $V_{GY} = V_y (A_R + \frac{2h_f + h_g}{2} B)$

Once the racking resistances of all the walls have been evaluated then the following checks must be made:

At First Floor Level

- $WRL_{FB} < \frac{V_{FX}}{2}$
- $WRL_{FL} < \frac{V_{FY}}{2}$

At Ground Floor Level

- $WRL_{GB} < \frac{V_{GX}}{2}$
- $WRL_{GL} < \frac{V_{GY}}{2}$

The identities may be used either to check for safety against an applied load or to find the maximum allowable applied loads.

Note

(a) The ground floor cases will normally be critical.

(b) Evaluation of the wind pressures $W_x$ and $W_y$ is discussed in the text.

Figure 8.7 A Simplified Method for Evaluating the Racking Capacity of a Building Assuming the Diaphragm to Act as a Beam With No Torsional Rigidity.

-579-
Vx and Vy may be considered independently or as a combined load case.

**Design Method**

Check that \( V_x < WRL_B + WRL_F \)
and \( V_y < WRL + WRL_R \)

If \( V_x/2 < WRL_B \) and \( WRL_F \), then \( R_B = R_F = V_x/2 \)
and \( V_y/2 < WRL_L \) and \( WRL_R \), then \( R_L = R_R = V_y/2 \)

If \( V_x/2 > WRL_B \) or if \( V_x/2 > WRL_F \)
then \( R_B = WRL_B \)
and \( R_F = V_x - R_B \)

Then \( M_x = \left( \frac{R_B - R_F}{R_B + R_F} \right) V_x \)

and the following forces are set up in the side walls

\( R_L = -R_R = \left( \frac{R_B - R_F}{R_B + R_F} \right) \frac{V_x}{L} \)

The direction of forces relative to the origin will be the same as the wall load which is less than half the applied force.

The same argument can be applied if \( V_y/2 > WRL_L \) or \( WRL_R \)

**Figure 8.8** External Wall Forces Based On the Use of a Torsionally Rigid Diaphragm
Figure 8.9 Map of United Kingdom Showing Basic Wind Pressure in kN/m²
Topography

<table>
<thead>
<tr>
<th>Value of $S_{11}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) All cases except those in b and c below</td>
</tr>
<tr>
<td>(b) Very exposed hill slopes and crests and valleys causing funnelling of the wind</td>
</tr>
<tr>
<td>(c) Steep sided enclosed valleys sheltered from wind</td>
</tr>
</tbody>
</table>

Table 8.4 Topography Factor $S_{11}$

Ground Roughness

<table>
<thead>
<tr>
<th>Value of $S_{12}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(e) Open country with no obstructions</td>
</tr>
<tr>
<td>(f) Built up areas or country with many natural windbreaks</td>
</tr>
</tbody>
</table>

Table 8.5 Ground Roughness Factor $S_{12}$

Aspect Ratio

<table>
<thead>
<tr>
<th>Value of $S_{13}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(wider length/narrower length)</td>
</tr>
<tr>
<td>greater than 1.75</td>
</tr>
<tr>
<td>less than 1.75</td>
</tr>
</tbody>
</table>

Table 8.6 Building Shape Factor $S_{13}$
CHAPTER 9

SUMMARY AND CONCLUSIONS

9.1 THE CURRENT INVESTIGATION

The principal objective of this investigation was to establish an accurate yet practical design method for timber frame walls which could be fully substantiated by test data. The capacity of the walls to resist vertical loads and wind forces normal to the face was related to standard design procedure (BS 5268 part 2) and therefore could be ignored. Thus the work concentrated on the racking resistance of the walls, which defines their capacity to resist in plane shear forces, either with or without the vertical loads acting in the plane of the racking forces. The secondary aims are listed below, these are:

(i) to simplify the design method so that the necessity for structural engineering input in standard building construction is reduced in cases where there has been no evidence of structural shortcomings in relation to resistance to applied loads,

(ii) to link the design method to a computer based analysis to enable the procedure to be extrapolated without the need for further expensive full scale tests except as isolated checks on the accuracy of the analysis and design data,

(iii) to consider the progression from the design of walls to the assessment of overall building performance in resisting horizontal wind loads.
The initial studies covered the construction of timber frame walls, the present requirements for their structural acceptability and both the analytical and test information concerning their design. The general conclusion was that the American ASTM E-72 test favoured by many countries, and commonly used in Britain prior to this decade, was unsuitable for modelling the standard restraints found in timber frame construction. The analytical methods recently developed all related to the ASTM test but were either too simplistic to cover the complexities of practical building length walls or were computer based and seemingly without an adequate data base to prove their acceptability.

Both the ASTM test method and the related analyses were rejected. The investigation was then directed to the development of a full scale test method suitable for both panel and wall tests which could be used to generate information from which an empirical design method could be substantiated. The early work on the British test conducted at Princes Risborough was followed up and a racking test related to applied vertical loads was developed. This has now been accepted for use as the British Standards approved method for testing timber frame walls. It was proposed principally to cover standard panels from which materials parameters could be evaluated for subsequent design use. Thus manufacturers could prepare normalised data for their products. However, the test was also used to carry out the investigation of more complex walls with openings from which all subsequent British design information has been derived. This extension to the use of the test method was generally very successful but great care was necessary in the reduction of the results to ensure that adequate factors of safety were maintained throughout the design process. This was complicated by the fact that the racking resistance was based on both stiffness and failure parameters which included partial safety factors to cover the quantity of
tests and which could be different for the two conditions. Furthermore, the test was required to produce data covering the full range of vertical load to which the materials were likely to be subjected.

Once the method of test had been formulated work in the investigation turned to the production of design values for the common materials used in timber frame construction in Britain. The method of reduction of results was adapted to produce a single design value, the basic racking resistance, which although less efficient was more suited to a general design method. Test results on similar materials were grouped and analysed to determine a tabulated value (the datum racking resistance), which defined a specific use of the sheathing material. Datum racking resistance is the standard starting point for design calculations but may be substituted by test data in which case a further degree of safety must be introduced and the applicability of the result is reduced.

The general design method is based closely on standard timber practice, thus the datum racking resistance is the equivalent of a grade stress and modification factors are applied, in addition to the wall length, to find the racking capacity of the wall at a given vertical load. The format for the modification factors follows that of BS 5268 Part 2 (BS1, 1984) such that where possible they are independently based and have a fail safe value of 1.0 for use with the lower bound of conditions that are commonly encountered. In the case of the length, the modification factor is also dependent on vertical load and these two variables must be considered together. However, as with the slenderness ratio modification factor K12 the second factor (in this case that for vertical load) is still applied separately. The modification factors are grouped into three sections covering materials, wall parameters and wall use which can be applied separately to identify the racking resistance of standard panels (a 2.4m square
imperforate wall) and practical walls. The use of the datum racking resistance in wall design implies certain minimum construction standards such as adequate fixing of the panel to headplates, baseplates and abutting panels, but in some cases secondary design calculations may be applied to evaluate non standard cases. It is necessary for the datum racking resistance and the modification factors to include a degree of conservatism because firstly very complex situations are covered by single equations and secondly extrapolation is difficult for walls longer than the 4.8m maximum test length.

In general the safety factors have been rigorously applied to datum racking resistance values and modification factors are based on mean behaviour patterns. Results have generally been based on stiffness performance and checks have been made to ensure that an adequate factor of safety is maintained against failure in practical wall situations. This method requires care in the use of subsequent test data, hence a further reduction factor is applied to such results. Throughout the design, but principally in the wall modification factors of vertical load, length and openings, the test data have been accurately modelled and the procedure is seen to combine precision with simplicity. During the investigation the method was compared with that included in the draft for publication of the Code of Practice on timber frame walls, BS 5268 part 6 (BSI, 1986). This had been prepared by the author but was based on limited test information and therefore had to place more reliance on previous, unsubstantiated test data from other sources. The current analysis of the data is therefore a much more accurate representation of behaviour in individual wall tests. Consequently much of the work has already been incorporated into the Code which has been substantially changed since being issued as a draft for comment. The inclusion of the findings of the current investigation at so late a stage in the preparation of the Code of Practice is a clear
indication that the principal objective of the work has been achieved.

The simplified design method was readily achieved by eliminating a number of the less important modification factors, limiting the variation in modification factors (i.e. reducing tables or equations to two or three alternative values) and simplifying equations used in the calculation of such factors. Clearly this loss in complexity will result in a reduced accuracy and greater conservatism. However, the test data is now presented in a form more equitable with that of the Building Regulations "deemed to satisfy" approach.

As a direct result of the test information and the production of the design procedure it is now possible to calculate the racking load capacity of all timber frame walls. Thus the resistances of all the walls in a building can be summed, taking account of any benefits to be gained from vertical loading, to determine the permissible horizontal wind load on a timber frame structure in any given direction. At this stage the practical design process becomes more subjective. Further safety factors are applied to some walls to take account of the previously limited structural use of either the materials or the element of the building. It is not part of the scope of this investigation to state how such walls should be treated and the authors remarks have been confined to advice on acceptable practice in the current climate where timber frame construction is regarded with scepticism by the British public.

Limiting the structural use of some walls has led to certain types of building failing to achieve the necessary racking resistance within the timber frame itself. This is unlikely to lead to any practical difficulties unless the building has no internal walls. However, it is necessary for design purposes to reduce the wind load reaching the
timber frame. Advice has been given on how this can be achieved in normal structures but a full investigation is outside the scope of the current work. Significantly the latest draft of the Code of Practice incorporates unpublished work from the BRE and the Polytechnic of the South Bank and suggests that up to 50% of applied load could be shielded by the brick outerskin in domestic housing. Although this information may be thought to reduce the significance of the work on timber frame walls and the need for an accurate assessment of wall resistance, it is valuable in that it explains why there have been so few recorded cases of practical damage to buildings when the racking resistance of their timber frame has been calculated to be comparatively weak.

The computer based analysis of timber frame walls has been treated as a secondary part of the investigation and was included to confirm the behaviour of walls during the British test, to enable the extrapolation of test results and to consider the progress of wall design from the test based empirical method presented in this investigation. The development of a new program relies on a wide ranging and accurate database. This was readily available from the full scale wall test results and therefore allowed the accuracy of both the analytical method and material variables to be checked. The analysis used a program prepared in Canada to cover the design of diaphragm walls. This was considered suitable because it dealt with inplane loading on a skeletal structure stiffened by membranes linked to the frame at discrete intervals. A number of modifications were required to the program to make it suitable for modelling the British timber frame wall test particularly with regard to the application of a constant vertical load. The major problem in using the program however related to the data needed to model the materials. Firstly, practical performance values were required for the frame and sheathing elements, not safe working stresses and moduli;
the sheathings caused a particular problem because, except for plywood, the required structural information, was not available. Secondly the nail performance data required for the frame fixings and the sheathing to frame connections covered only plywood and was limited to initial stiffness. Thus it was necessary to quickly generate data to cover all the common sheathing boards which would model performance through to failure. A rigorous analysis of nail performance could alone represent a major investigation and could not be justified within the current work. Thus even greater importance was placed on the full scale data base as a means of back-analysing the material parameters. The initial trials were carried out on the standard 2.4m square plain panels. The ability of the program to model the structure as loaded was shown and it was possible to identify the major factors influencing performance and thereby check the accuracy of the material parameters. These were adjusted to provide a better model and the analysis was extended to cover longer walls and also panels with openings. The results of these trials showed the method of analysis to have great potential although a number of problems were encountered which will need to be overcome before full advantage can be taken of computer based design.

At this stage the aims of the investigation relating to the analysis were considered to have been achieved and it is clear that with further development the program will be able to model all test results and then to extrapolate the design information. This could then be extended to cover other materials requiring only small scale tests to determine the material parameters instead of the expensive standard panel tests presently required.
9.2 RECOMMENDATIONS FOR FURTHER WORK

The development of the racking test method and the test programmes detailed in the preparation of the design method has covered a ten year period which will be effectively concluded by the publication of the Code of Practice on Timber Frame Walls (expected late in 1987). The inclusion of design values for standard materials and the additional safety factors applied to test data will limit the need for standard panel tests to certify new materials. Specialist test investigations may still be necessary, particularly with regard to longer walls, combined sheathings and complex openings where the available data is relatively limited. The benefits gained from such tests would however be very specific and their objective would be to fill in gaps in available information. The tests would undoubtedly be more costly and difficult to set up because of the complexity of the panels to be tested and would lend themselves to analysis by the computer based method once the program and input data files had been adjusted to achieve better accuracy.

The main recommendations for further work concern the development of the computer based analysis. The findings of the present version of the SADT program have been noted in Chapter 8. It is clear that both the method of analysis and the accuracy of the materials parameters must be improved to enable the use of the program to be developed. This will involve a considerable amount of work including many small scale tests to obtain data for the materials and a small number of full scale tests on more complex walls to check the accuracy of the analysis. The computing work may then be extended from single wall units to investigate the performance of buildings. The SADT program developed for walls could be used with the original diaphragm program and incorporated into a three dimensional structural analysis package to predict building behaviour. A great deal of additional test data will then be required.
to cover both building performance, to check the quality of the analysis, and the interaction behaviour of individual components.

In both cases some data are already available, although in Britain much remains unpublished. The major work areas cover the interaction of wall elements the overall stiffening effects of horizontal diaphragms (the box effect) and the shielding effect of the brickwork outer skin walls which reduces the horizontal force applied to the timber frame structure.

9.3 FINALE

To conclude; the work in this investigation allows the designer to carry out repeatable constant tests on timber frame walls and to determine their racking resistance by empirical means. The ability to model the test method and achieve the same results using a computer based analysis has also been shown. The work should now be developed in both the empirical and analytical areas to cover three dimensional structures and thus assess the racking resistance of timber frame buildings.
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APPENDIX A

EXTRACTS FROM THE DRAFT FOR PUBLIC COMMENT OF BS 5268*
SPECIFICALLY PREPARED BY THE AUTHOR

* BS 5268 Structural Use of Timber Part 6. Code of Practice for timber frame walls, Section 1. Dwellings not exceeding three storeys. BSI document 86/13188 DC.
19.2 Methods of determining racking resistance of walls

The racking resistance of walls built up from a number of braced or sheathed wall panels may be derived:

(a) by the assessment method described in clause 20 using the basic racking resistance values given in table 1. The figures given in table 1 for basic racking resistance should be modified by the material modification factors given in clause 20 and the wall modification factors given in clause 21, as appropriate.

Therefore:

Racking resistance of wall

= Basic racking resistance (table 1) x wall length x Material modification factors (K101, K102, K103) x Wall modification factors (K104, K105, K106, K107 and K108)

or

(b) by load testing of 2.4 m square panels to find the basic racking resistance of a particular combination of materials and construction. Such panels should be tested in accordance with Section four and be tested in all respects in a form identical to the construction to be used in the design.

Where it is required to add the contribution of a secondary layer of a category 1, 2 or 3 material (as defined in table 1) then provided that the basic racking resistance of the primary board material, derived by test, does not exceed 2.1 kN/m, then the additional contribution values given in table 1 may be used. If the basic racking resistance of the primary board material, derived from test, is greater than 2.1 kN/m then any additional contribution from secondary board materials should be quantified by testing the primary board material with and without the secondary material.

The basic racking resistance value derived from such testing may be substituted for the values given in table 1 and modified by the wall modification factors described in clause 21. Since the test should incorporate the required nail spacing, etc, the material modification factors given in clause 20 (i.e. K101, K102 and K103) should not be applied to basic racking resistance derived from test.

Racking resistance of wall

= Basic racking resistance (derived from test) x wall length x Wall modification factors (K104, K105, K106, K107 and K108)

or
(c) by load testing of full sized walls. The walls should be tested in the form in which they are to be used and the permissible racking resistance for the wall derived in accordance with Section four. Material and wall modification factors (K101 - K108) should not be applied to wall racking derived in this manner.

or

(d) by detailed analytical methods which are outside the scope of this code. Material modification factors given in clause 20 and wall modification factors given in clause 21 should not be applied to designs carried out independently of this code.

19.3 Racking deflection

The permissible racking deflection should be within limits appropriate to the type of construction, having particular regard to the possibility of damage to surface materials, ceilings, partitions, doors, windows and finishings. Values of basic racking resistance given in table 1 are based upon a maximum deflection of 0.003 x panel height. Table 1 values may be reduced proportionally in respect of a lower deflection limit but they should not be increased.

19.4 The contribution of plasterboard to racking resistance

19.4.1 General. With the specific exception of separating walls, comprising two or more built up layers of plasterboard (see clause 19.5), plasterboard must not be relied upon solely to provide the racking resistance of a dwelling.

However, plasterboard can be assumed to make a contribution to racking resistance provided that the principal resistance is provided by a category 1 or 2 material (as defined in table 1). When considering the walls providing resistance to wind forces in any one direction the plasterboard linings described in 19.4.1 and 19.4.2 may be taken into account provided that the total contribution does not exceed 50 % of the resistance provided by category 1 or 2 materials as defined in table 1.

The contribution of plasterboard to racking resistance assumes that:

(a) The plasterboard is fixed in accordance with the requirements of table 1.

(b) The walls are fully supported throughout the length and connected to supports in such a way as to ensure the transfer of applied shear forces.
19.4.2 **Plasterboard linings to external sheathed walls.** The contribution of plasterboard to external sheathed walls may be calculated by using the additional lining contribution values given in table 1 modified as appropriate by modifications factors K103 to K108. The plasterboard may be fixed on either the opposite face or the same face as the sheathing, providing that it is independently nailed.

19.4.3 **Internal walls:** Subject to the overall requirements for plasterboard lining contribution given in 19.4.1 above, internal walls lined each side with plasterboard may make a contribution to the racking resistance of the dwelling.

The basic racking resistance for the wall should be derived from table 1 using the basic racking resistance for a plasterboard lined wall plus the added contribution of the second layer. The value so obtained should be modified by modification factors K103 to K108 as appropriate.

Door openings in such walls should be regarded as structural discontinuities and the racking resistance derived from the sum of the racking resistances of the plain panels on either side of the openings.

**NOTE.** In calculating the racking resistance of the plain walls the length should be taken as the length of each plain section of wall under consideration.

19.5 **Plasterboard lined separating walls**

In view of the special construction of separating wall panels, no limit is placed upon the contribution of plasterboard in such walls built-up of 2 or more layers of plasterboard to each leaf nailed as described in table 1.

The unlimited contribution of plasterboard in separating walls is subject to the following conditions:

(a) Full panel height diagonal bracing should be fixed to each separating wall panel such that there are never less than two such braces on any separating wall leaf. The diagonal braces should be attached to each stud and nail with a minimum of 3 no x 3.25 mm diameter steel nails with a pointside penetration of at least 35 mm.

or

(b) A panel height sheathing of category 1 material (as defined in table 1) and 1200 mm wide placed at each separating wall abutment with an external wall, on each leaf and fixed as described in table 1.

or

(c) One of the layers of gypsum plasterboard fixed to the timber frame separating wall panels should be of a moisture resisting grade.
20 Assessment method for determining the basic racking resistance of certain material combinations

Basic racking resistances for certain material combinations are given in table 1. The values given in table 1 are basic racking resistances based upon test evidence of fully sheathed wall panels, 2.4 metres square, and for generic materials described in Section two. The values given in table 1 taken account of the appropriate load duration factors given in table 15 of BS 5268: Part 2: 1984 for short and very short term loads and are based upon the zero vertical load condition.

Specific test results derived from tests in accordance with Section four of this code can be substituted for the values given in the table subject to the rules laid down in 19.2 (b). The use of table 1 materials on test evidence of basic racking resistance, however, should not be taken as evidence that a particular material is fit for the purpose for which it is intended. Designers must assure themselves of the required durability for intended use of materials.
<table>
<thead>
<tr>
<th>Primary board material (thickness in mm)</th>
<th>Fixing</th>
<th>Basic racking resistance (kN/m)</th>
<th>Additional contribution of secondary board on timber frame wall (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Category 2 or 3 Materials</td>
</tr>
<tr>
<td><strong>Category 1 Materials</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.5 mm Plywood</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.0 mm Mediumboard</td>
<td>3.00 mm diameter wire nails at least 50 mm long</td>
<td>1.68</td>
<td>0.28</td>
</tr>
<tr>
<td>12.0 mm Chipboard (Type III)</td>
<td>Maximum spacing</td>
<td>150 mm on perimeter</td>
<td>300 mm internal</td>
</tr>
<tr>
<td>6.0 mm Tempered Hardboard</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Category 2 Materials</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.5 mm Bitumen impregnated insulation board</td>
<td>3.00 mm diameter wire nails at least 50 mm long</td>
<td>Maximum spacing</td>
<td>75 mm on perimeter</td>
</tr>
<tr>
<td>Separating wall of min 30 mm plasterboard (in 2 or more layers)</td>
<td>Each layer to be individually fixed with 2.65 mm diameter plasterboard nails at 150 mm spacing. Nails for the outmost layer to be at least 60 mm long</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td><strong>Category 3 Materials</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.5 mm Plasterboard</td>
<td>2.65 mm diameter plasterboard nails at least 60 mm long</td>
<td>Maximum spacing</td>
<td>150 mm.</td>
</tr>
</tbody>
</table>
20.1 Table 1 values are applicable under the following conditions:

(a) Timber members in wall panels should be not less than 38 mm x 72 mm rectangular section with linings fixed to the narrower face, with ends cut square and assembled in accordance with Section five.

(b) Studs should be spaced at centres not exceeding 610 mm.

(c) Board edges should be backed by, and nailed to timber framing at all edges except in the case of the underlayers in separating wall construction where it is normal to fix boards horizontally, in which case the intermediate horizontal joint may be unsupported.

(d) Studs are to be of a species and stress grade falling into Strength Class 3 or better (as defined in Part 2 of this Code).

(e) The additional contribution from a category 1, 2, or 3 material should only be included once in the determination of basic racking resistance, no matter how many additional layers may be fixed to the wall panel.

(f) The values given in table 1 assume that the designer has checked to ensure that the wall panels under consideration are adequately fixed to ensure resistance to sliding and overturning.

20.2 Modification factors for variation in fixing and thickness of materials described in table 1.

20.2.1 Variation in nail diameter. For variations in nail diameter between 2.25 mm and 3.75 mm the values for basic racking resistance given in table I should be multiplied by K101.

where: \[ K_{101} = \frac{\text{Proposed nail diameter}}{3} \]

The recommended size of nail for fixing plasterboard is 2.65 mm diameter. No enhancement of basic racking resistance is permitted for the use of any other size of nail.

20.2.2 Variation in nail spacing. For sheathings other than plasterboard the values for basic racking resistance given in table I may be multiplied by K102 to account for variations in nail spacing.

where: \[ K_{102} = \frac{1}{(0.6A + 0.4)} \]

Where \[ A = \frac{\text{Proposed perimeter spacing}}{\text{Perimeter spacing of nails specified in table 1}} \]

No variation in the table 1 racking resistance for plasterboard should be taken as a result of variation of nail spacing. Plasterboard nailed with smaller nails or at greater centres than specified in table 1 should not be considered to contribute to racking resistance.
Where plasterboard is combined with other sheathing on the same wall no increase in basic racking resistance should be permitted to the combined basic racking resistance value given in table 1 as a result of increasing nail density. However, an enhanced value for the sheathing alone can be substituted if it is more advantageous than the combined board condition.

20.2.3 Variation in board thickness. The values for basic racking resistance given in table 1 may be modified by K103 in respect of variations in thickness of sheathings or linings.

where: \[ K103 = (2.8B - B^2 - 0.8) \]

where \( B \) = Proposed board thickness
\[ \text{Standard board thickness specified in table 1} \]

In no case should \( B \) be less than 0.75 or greater than 1.25.

21 Modification factors for wall length, vertical load, height openings and interaction

21.1 Height of wall panels

For wall panels of height between 2.1 m and 2.7 m the height effect factor K104 should be calculated as follows:

\[ K104 = \frac{2.4}{\text{Wall height in metres}} \]

For walls exceeding 2.4 m in height and where an intermediate horizontal joint is required such joints should be framed and nailed in accordance with the requirements of Section five. The formula for K104 should not be used to extrapolate values for heights of walls less than 2.1 mm or greater than 2.7 mm.

21.2 Length of walls

The permissible racking resistance of a timber framed wall will be influenced by the length effect of the wall. The length effect factor K105 should be taken from table 2 or calculated as follows:

For wall lengths from 0 mm to 2.4 m

\[ K105 = \frac{L}{2.4} \]

For wall lengths over 2.4 m to 4.8 m

\[ K105 = \left(\frac{L}{2.4}\right)^{0.4} \]
For wall lengths in excess of 4.8 m.

\[ K_{105} = 1.32 \]

Table 2 tabulates some values of \( K_{105} \).

<table>
<thead>
<tr>
<th>Length of wall (in metres)</th>
<th>0.6</th>
<th>1.2</th>
<th>1.8</th>
<th>2.4</th>
<th>3.0</th>
<th>4.2</th>
<th>4.8 or greater</th>
</tr>
</thead>
<tbody>
<tr>
<td>( K_{104} )</td>
<td>0.25</td>
<td>0.50</td>
<td>0.75</td>
<td>1.00</td>
<td>1.09</td>
<td>1.25</td>
<td>1.32</td>
</tr>
</tbody>
</table>

Where wall panels are combined to form the lengths of wall given in this clause it is imperative that the following conditions are met.

(a) Tops of individual wall panels should be linked together by a continuous member or construction. For example, a head binder nailed at 600 mm centres ensuring 35 mm pointside penetration of nails into the wall panels.

(b) The faces of end studs of contiguous panels should be fixed to an extent that any vertical shear is transferred. In the absence of more specific information end studs should be nailed with the equivalent of 3.35 mm nails, 75 mm long at 300 mm centres.

(c) The coupled panels should be able to resist overturning forces as described in clause 16.

21.3 Window, door and other fully framed openings in walls

For a wall with framed openings, the permissible racking resistance should be reduced to take account of the effect of framed openings. The opening effect factor \( K_{106} \) should be taken from table 3 or calculated as follows:

\[ K_{106} = (1 - 1.3 \, p)^2 \]

where \( p = \frac{\text{Area of openings}}{\text{Total area of wall}} \)

\( K_{106} = 0 \) where \( p \) is greater than 0.75.
Table 3 tabulates some values of $K_{106}$

<table>
<thead>
<tr>
<th>Table 3. Some values of modification factor $K_{106}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>0</td>
</tr>
<tr>
<td>1.0</td>
</tr>
</tbody>
</table>

These recommendations for openings are valid provided that:

(a) all edges are supported by members having a thickness not less than the thickness of the studs;

(b) a means of transferring horizontal in-plane forces above and below openings is provided. Where no such provision is made the wall lengths on either side of the opening should be designed as separate parts.

Where an opening is less than 300 mm from the corner of a building and the depth of opening is greater than half the panel height then the length of that part of the wall, up to and including the opening, should be disregarded when determining the total length of wall (see clause 21).

Where two framed openings are separated by less than 300 mm and the heights of both openings are greater than half the panel height then the area of opening should be taken as that of the rectangle which enclosed both openings.

NOTE. This method of assessing the effect of wall openings takes account of the worst case of openings in a timber framed wall. Where higher values of racking resistance can be obtained by considering a wall as a number of shorter lengths then this is acceptable.

21.4 Small unframed openings

Notwithstanding the recommendations given in 21.3 the effect of small unframed openings need not be considered and the edges of these openings in the sheathing need not be framed where:

(a) they do not exceed 250 mm in diameter or in length of side;

(b) the clear distance between openings is not less than the greatest dimension of the openings;

(c) the clear distance between the edge of the sheathing and the edge of any opening is not less than the greater dimension of the opening;

(d) not more than one such opening occurs in any one 600 mm width of sheathing or lining.
Smaller un-framed openings are acceptable to a greater extent provided that the total area does not exceed the area of opening that would have been permitted by (a) above. In this case the rules governing the position of openings in (b) and (c) above will apply.

21.5 Variation in vertical load on timber frame wall

The values of basic racking resistance given in table 1 assume zero vertical load on the timber frame wall panels. Where vertical loads are imposed on wall panels the basic racking resistance may be multiplied by the modification factor $K_{107}$. In calculating the value of $K_{107}$ the vertical load on the wall should be calculated using only the dead or permanent loading and any nett effects of wind. In calculating $K_{107}$ the maximum value of uniformly distributed vertical loading on any wall or panel should not be taken as being greater than 10.5 kN/m. $K_{107}$ is calculated as follows:

$$K_{107} = 1 + \left(0.09F - 0.0015 F^2\right) \times \left(\frac{2.4}{L}\right)^{0.4}$$

where, $F =$ uniformly distributed load (kN/m) (not to be taken as greater than 10.5kN/m)

$L =$ length of wall (m)

Some values of $K_{107}$ are given in table 4.

<table>
<thead>
<tr>
<th>Length of wall in m</th>
<th>Vertical load in kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>--------------------</td>
<td>---</td>
</tr>
<tr>
<td>1</td>
<td>1.00</td>
</tr>
<tr>
<td>2</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>1.00</td>
</tr>
<tr>
<td>4</td>
<td>1.00</td>
</tr>
<tr>
<td>5</td>
<td>1.00</td>
</tr>
<tr>
<td>6</td>
<td>1.00</td>
</tr>
<tr>
<td>7</td>
<td>1.00</td>
</tr>
<tr>
<td>8</td>
<td>1.00</td>
</tr>
<tr>
<td>9</td>
<td>1.00</td>
</tr>
<tr>
<td>10</td>
<td>1.00</td>
</tr>
</tbody>
</table>

It is assumed that in using $K_{107}$ that any uplift or overturning moments have been taken into account and any necessary holding down fixing designed. Therefore, in applying $K_{107}$ the vertical load should not be considered to be less than zero.
For the purposes of applying table 4 concentrated vertical loads should be converted into equivalent vertical uniformly distributed loads by the formula:

\[ F = \frac{2aF_p}{L^2} \]

where

- \( F \) is the equivalent u.d.l (kN/m)
- \( F_p \) is the concentrated load (kN)
- \( F_a \) is the distance from \( F_p \) to leeward end of the wall panel under consideration (m)
- \( L \) is the length of wall under consideration (m)

A concentrated load can also be assumed to be developed by connections directly between the wall panel studs and the substructure, or in the case of a corner or internal wall, the wall at right angles.

21.6 Interaction

The basic racking resistance values given in table 1 or as derived from test and modified, as appropriate, by modification factors K101 to K108, give accurate assessments of the racking resistance of plain walls when subjected to test racking loads.

When such walls form part of completed dwellings experience shows that the method of assessment underestimates the permissible racking resistance of walls since it does not take into account factors, such as the stiffening effect of corners, and the interaction of walls and floors through multiple fixings.

Therefore, it is recommended that in calculating the permissible racking resistance of walls, the basic racking resistance is multiplied by K108, where K108 is 1.1 in all cases.

22 The contribution of masonry veneer to racking resistance

The permissible racking resistance for sheathed timber frame walls, for all combinations of sheathing, lining and vertical load conditions, is permitted to be increased to take account of the racking resistance of masonry cladding provided that the wall ties, with appropriate fasteners, have a minimum design horizontal shear strength of at least 150 N at deformations of 5 mm or more and a characteristic horizontal shear stiffness of not less than 30 N/mm over the deformation range 0 mm to 5 mm when tested in accordance with appendix A.

The additional racking resistance for masonry cladding given in table 5 should only be applied to those parts of the wall comprising a minimum of 2.4 m storey height masonry at least 600 mm wide backed by storey height timber frames, have a specification and constructed in accordance with the requirements of clause 10 of this code.
Section four. Load testing

26 General

As an equally acceptable alternative to a theoretical analysis or a design using figures quoted in section 3 the adequacy of a timber framed wall may be determined by full-scale testing. Tests may be used to determine any of the following.

(a) The racking resistance of a given configuration of materials and thus derive the basic racking resistance for that combination of sheathing(s), fixings and framework. The tests should be carried out on standard 2.4 m square panels representative of the typical structure but excluding features such as openings and should cover a range of vertical load conditions.

(b) The racking resistance of a full scale wall. The test is usually carried out when:

(1) calculations are deemed impracticable because of the complexity of the design;

(2) there is doubt or disagreement as to whether the wall complies with the design requirements or whether the quality of material is of the required standard.

(c) The racking performance of a panel where routine check of mass produced panels is required by prior agreement between the client and the manufacturer.

Section 8 of BS 5268: Part 2: 1984 deals with the testing of timber structures and components generally. For testing timber frame walls under combined wind racking and vertical loads the detailed procedure described in this section should be adopted. Wherever possible more than one panel should be tested to enable an assessment to be made of likely variability.

27 Testing authority

Tests on timber frame wall panels should be designed, supervised and certified by a competent authority to ensure that the tests are in accordance with this section.
28 Information required

A copy of the detailed drawings and specifications for the panel and fixings together with details of design loads (both racking and vertical) when known, conditions of exposure, humidity and temperature in which the panel is to be used, or the moisture content of the timber and sheet materials that have been assumed for design purposes, and any other data or information that may be required for the purposes of the test should be deposited with the testing authority before the tests are commenced.

29 Materials

The materials used in the test panel should be of the minimum basic dimensions allowed by the specification. The quality should be, as far as is practicable, the minimum quality, and in no case better than the average quality allowed by the specification. Such evidence of quality as is required to ensure that this condition is met should be supplied to the supervising engineer who can require the test panel to be replaced or modified if in his opinion the panel as supplied is not truly representative of the specification requirements.

Where testing is being carried out in order to derive basic racking resistance for use in place of table 1 values, the wall panel should be constructed from timber no better than Strength Class 3 (in accordance with Part 2 of this Code), of average density for the species and in no case having a density greater than 540 kg/m³ when measured at 18% moisture content (see table 91 of Part 2 of this Code).

30 Manufacture

Where a prototype or production timber frame wall is to be tested the manufacture and assembly of the wall should comply with the design specification and the methods used should simulate as closely as possible those which would normally be used in production or site assembly.

Where a standard timber frame test panel is used to determine the basic racking resistance of a combination of materials, a 2.4 m² square panel should be assembled simulating as closely as possible the typical panel construction with regard to:

(a) size, spacing and specification of studs and horizontal members.
(b) type, thickness, size and orientation of sheathing.
(c) size and spacing of mechanical fasteners.
(d) method of assembly.
31 Test equipment

The test equipment should be capable of applying separately, horizontal racking loads and vertical loads to the top plate of the test panel. In order to simulate design requirements, the vertical load equipment should be capable of imposing both concentrated and uniformly distributed loads to the top of the panel. It is allowable for uniformly distributed loads to be applied as a series of equal concentrated loads distributed along the panel by means of a suitable spreader beam. When testing a panel to derive basic racking resistance (see 26(a)) the vertical load should be applied as equal point loads over the stud positions, at 600 mm centres. The load intensity is then described as the point load in kilonewtons per stud. The equivalent uniformly distributed load is given as:

\[
\text{UDL (kN/m)} = \frac{\text{stud load (kN)} \times 5}{2.4}
\]

The methods of application of racking loads and vertical loads should be such that no significant restraint is offered to the panel by either load except in the direction of application of the load. The accuracy of load application and load measurement should be within ±5 %.

A rigid datum independent of the test panel and loading frame should be provided for deflection measurements which should be made in the direction shown at points A and B in figure 1. Deflections should be measured to the nearest 0.05 mm.
All dimensions in millimetres

Figure 1. Arrangement of standard panel in test rig (to be read with the notes for guidance)
NOTES FOR GUIDANCE IN SETTING UP THE TEST EQUIPMENT

1. It is essential that the stud loads on a standard panel are applied at nominal 600 mm centres and they should normally coincide with stud centres. It should be noted that, in the failure tests, racking deflections greater than 50 mm can be expected and therefore, dependent on the method of application, the first vertical load point may need to be moved back; however the distance to the second load point should not be reduced below 500 mm.

2. A head binder is often fixed to the top rail of the test panel and is essential in standard tests if either the studs are not at nominal 600 mm centres or the top rail is not continuous. Lateral restraint of the panel may be applied through the head binder.

3. The racking load should be applied to the top rail of the panel. The point of application may be fixed but it should be noted that the panel will lift and twist backwards under load. The load should be applied to the frame only and must not impede movement of the sheathing(s).

4. It is essential that the method of application of the vertical load is such that it will not cause any movement in the base of the test rig.

5. Horizontal displacement should be measured at points A and B on the top and bottom rails of the panel. The racking deflection is then taken as displacement A minus displacement B. The value of B taken in this calculation should never exceed 0.5 mm.

6. The uplift of the first stud should be measured at C as it is useful in judging the relative behaviour of panels and in giving an indication if an error has been made in setting up the panel in the test rig or in carrying out the test procedure.

7. The standard panel should be bolted to the base of the test rig with four bolts, one in each bay of the panel where studs are at nominal 600 mm centres. The first bolt should be fixed between 150 mm and 200 mm from the face of the panel to minimize uplift of the bottom rail. The bolts should act through 50 mm square washers and should be tightened until the washer starts to embed into the bottom rail.

8. The timber packer should be used in the standard test and may be necessary in all tests to ensure that the movements of the sheathing boards are not impeded by the test rig.

9. The base of the test rig must be perfectly flat under the panel and should be sufficiently stiff not to move under application of loading.
32 Test conditions

The test panels should be installed in the test rig and fixed to the base by methods which simulate as closely as possible the fixings that are to be used in service. Where the method of holding down the panel is not known at the time of test the fixings to the base should be such that uplift or horizontal movement of the bottom plate of the panel is minimal during the test. Particular attention should be given to the positioning of panels on the base and the location of bearers at loading points, to ensure that no loads are applied directly to the sheathing or plasterboard lining except through the fixings between the timber frame and the sheathing or plasterboard lining.

Lateral restraint at right angles to the plane of the test panel should be provided equivalent to that likely to be attained in service. Care should be taken to ensure that these restraints do not inadvertently resist movements in the plane of the panel.

Where it is clear that there is unavoidable and significant divergence from service conditions either in load application or restraint, it is essential that this be noted and taken into account when analysing the test results.

33 Criteria for selection of test loads

With regard to structural performance the serviceability of panels subject to racking loads may be limited by either stiffness or strength, both of which are dependent on the applied vertical load.

Where the racking and vertical loads likely to occur in service are known they should be used for tests conducted on a single panel to establish the suitability of the panel for use under the specified load combination.

Where a panel is intended for use under a range of vertical loads a minimum of two similar panels should be tested. One panel should be tested for strength under the assumed maximum vertical load and the other under minimum vertical load. In addition at least two stiffness tests should be carried out, one under the maximum vertical load and another under the minimum vertical load. In the absence of any specified alternative, the minimum vertical load should be taken as zero. Further tests at intermediate vertical loads are helpful in the interpolation and derivation of the permissible design racking loads over the range of vertical loads considered. Each test panel should be subjected to a maximum of three stiffness tests, each at different vertical loads but only one strength test should be carried out under one vertical load.
In the particular case where the basic racking resistance (see 26(a)) of the combination of materials is being assessed the vertical loads should range between zero and 5 kN/stud, or equivalent. The test procedure described in this section is not intended for assessing the racking resistance of panels subject to a net vertical uplift. Where it can be shown that uplift forces are effectively transmitted through the structure independent of the sheathing or bracing the permissible racking load for this condition should be based on racking tests with zero vertical load.

34 Test procedure
34.1 Introduction

Each type of panel should be tested for stiffness and strength. Where it is intended to take account of the additional stiffness and strength due to plasterboard linings the panels should be tested with both structural sheathing and plasterboard lining attached. It should be noted that it will be necessary to know the performance of the structural sheathing alone to meet the requirements stated in 19.1. This should be assessed using table 1 or by separate tests on panels clad only with the structural sheathing.

34.2 Vertical settling loads

Before testing a panel in racking for stiffness or strength a uniformly distributed vertical settling load of 1.5 kN/m run or 0.75 kN/stud should be applied to the top plate of the panel. This load should be maintained for 5 min, then released and the panel allowed to recover for 5 min before the commencement of the racking tests.

The uniformly distributed vertical settling load need not be applied when the first racking test on the panel is carried out under a uniformly distributed vertical load greater than 1.5 kN/m run.

34.3 Stiffness test

The vertical load specified for the test should be applied to the top plate of the panel and maintained constant throughout the test. A racking pre-load, sufficient to cause a racking deflection of 0.0005 x panel height, should then be applied to the top plate of the panel and removed immediately. Five minutes after the removal of this load all deflection gauges should be read to provide the datum from which all subsequent deflections, under the applied vertical load, are measured.
Immediately after the datum readings are taken, the racking load should be reapplied and increased until the racking deflection is equal to 0.002 x panel height. The racking deflection is taken as the displacement at A minus the displacement at B (see figure 1). The racking load should be increased continuously or in approximately equal increments with equal time intervals between each increment. When continuous loading is used the measurement of load and deflection should also be continuous. When incremental loads are used there should be at least four increments to reach maximum load and the load and deflection measurements should be taken after each increment. The application of racking load, for both continuous and incremental loading, should be such that the maximum load is reached not less than 4 min and not more than 10 min after the commencement of the loading cycle. The maximum load should be maintained only for sufficient time to allow the load and deflection measurements to be taken and then removed, taking care to maintain the specified vertical load constant. Deflection measurements should be taken immediately after the removal of the load and again 10 min later. This loading cycle, excluding the racking pre-load, should be performed four times although it is not necessary to remove the racking load at the end of the fourth cycle if the strength test is to be carried out immediately at the same vertical load.

When a panel is to be tested under more than one combination of vertical and racking loads the minimum vertical load condition should be tested first and subsequent tests should be in order of increasing vertical load. The full stiffness test should be carried out at each vertical load including the application of the racking pre-load to establish a new deflection datum for each vertical load. The panel should be examined at the end of each stiffness test and any structural damage recorded.

34.4 Strength test

The vertical load specified for the strength test should be applied and maintained constant throughout the test. The racking load should then be applied and gradually increased, either continuously or in approximately equal increments with equal time intervals between each increment, until failure occurs. The maximum racking load attained during the strength test should be recorded.

Failure is defined as the fracture of any part of the frame or sheathing material, failure of the fastenings or when the racking deflection continues to increase without any further increase in racking load. Because of the possibility of a sudden redistribution of forces in the panel causing a temporary reduction in the racking load, care is necessary in identifying the maximum racking load.

The application of the racking load, for both continuous and incremental loading, should be such that the increase in racking deflection is not more than 15 mm in any period of 5 min.

At the conclusion of the strength test the moisture content of the timber should be recorded together with a description of the structural damage suffered by the panel.
35 Determination of design racking resistance values

35.1 General

The results of stiffness tests, and where appropriate the strength tests, should be plotted for each panel in the form of racking deflection (displacement at A minus displacement at B as shown in figure 1) against racking load for each loading cycle. These results should be used to assess the stiffness and strength characteristics of each panel as follows.

35.2 Test racking stiffness load

The minimum racking load required to produce a racking deflection of 0.002 x panel height should be determined for each panel for each vertical load applied in the tests. The average of the corresponding values for each panel should be calculated and multiplied by 1.25 and the appropriate factor $K_{108}$, from table 6, to give the test racking stiffness load for each vertical load condition tested.

<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>$K_{108}$</td>
<td>0.80</td>
<td>0.87</td>
<td>0.93</td>
<td>0.97</td>
<td>1.00</td>
</tr>
</tbody>
</table>

In addition the residual racking deflection measured 10 min after the removal of the racking load at the end of the first and third loading cycles, should satisfy the following conditions.

(a) The residual racking deflection after the first load cycle should not be greater than 0.0005 x panel height.

(b) The residual racking deflection after the third loading cycle should not be greater than 0.001 x panel height.

(c) The magnitude of the increase in the residual racking deflection due to each loading cycle should reduce for successive cycles unless the increase is equal to or smaller than 0.0001 x panel height, when it may be ignored.

If the panel fails to comply with requirements (a) or (b) the measured minimum racking load required to produce a racking deflection of 0.002 x panel height in the particular stiffness test where the non-compliance occurred should be multiplied by the factor $K_{109}$. 

ZZ6270(D72) -A21- 87/11343
Where \( K_{109} = \frac{\text{Maximum permissible residual racking deflection}}{\text{Measured residual racking deflection}} \)

This reduced value of minimum racking load should be included in the average of corresponding results from similar panels, when they have been tested, and hence used to calculate the test racking stiffness load.

35.3 Test racking strength load

The test racking strength load should be taken as the minimum racking load required to cause failure in similar panels tested under the same vertical load multiplied by the acceptance factor \( K_{108} \), given in table 6.

35.4 Test racking design load

The test racking design load for the particular vertical load under which a panel was tested should be taken as the lesser of:

(a) the test racking stiffness load, determined in accordance with 35.2; or

(b) the test racking strength load, determined in accordance with 35.3, divided by the appropriate safety factor given in table 7.

When a particular panel type has been tested under more than one vertical load the test racking stiffness loads and the test racking strength loads should be linearly interpolated for intermediate vertical loads. For any particular vertical load the test racking design load should be taken as the smaller of these interpolated test racking stiffness or the interpolated test racking strength values divided by the appropriate safety factor given in table 7.

Test results and test racking design loads should not be extrapolated outside the range of vertical loads applied during the test.
Table 7. Factors of safety for test racking strength load

<table>
<thead>
<tr>
<th>Sheathing, lining or combination</th>
<th>Factor of safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Any sheet material other than plasterboard described in section 2 of this code.</td>
<td>1.6</td>
</tr>
<tr>
<td>(b) Plasterboard or any other sheet material not described in section 2 of this code</td>
<td>2.4</td>
</tr>
<tr>
<td>(c) Combination of two materials as described in (a) above</td>
<td>1.6</td>
</tr>
<tr>
<td>(d)* Combination of two materials where either one or both is as described in (b) above</td>
<td>2.4</td>
</tr>
</tbody>
</table>

* When the combination is of one material described in (a) and one as described in (b), the factor of safety of 2.4 need only be applied to the additional racking strength load obtained by using the material as described in (b).

35.5 Basic racking resistance

The basic racking resistance, expressed as a load per metre run of panel, for a combination of materials is determined from the test racking design loads as derived from a standard test panel over a range of vertical loads that include zero and 5 kN/stud (or equivalent thereof). The basic racking resistance should be taken as the value of test racking design load, as calculated in 35.4, for the zero vertical load condition for the standard test panel (see 26(a)) that when used in conjunction with appropriate vertical load modification factors gives safe design values when compared with the test design figures throughout the range of vertical loads between zero and 5 kN/stud. Table 8 gives modification factor K110 appropriate for use with concentrated stud loads at nominal 600 mm centres and compatible with the uniformly distributed vertical load factor given in table 4.

Table 8. Vertical load modification factor K110 for point loads at nominal 600 mm centres on a standard test panel

<table>
<thead>
<tr>
<th>Vertical load in 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>Modification Factor K110</td>
<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>
</tr>
</tbody>
</table>
35.6 Acceptance of panels for specified design loads

A panel is acceptable as satisfactory to sustain a specified design racking load if the test racking design load determined in accordance with 35.4 is equal to or greater than the specified design racking load.

36 Report of the test

In addition to the test results the report should contain a detailed description of the panels tested including the type and quality of materials used in their construction together with a note of any defects. The report should describe the fixings by which the panels were held down in the test rig, the methods of loading and of measuring deflections together with load/deflection graphs and any other relevant information. The type and position of any failure should be recorded together with the moisture content of timber frame at the time of failure. The nature and size of defects in the timber or sheathing should be recorded together with any defects in the design construction that could lead in time to significant reductions in stiffness or strength.

The report should also contain a clear statement of the test racking design loads determined from the tests, together with the relevant vertical loads.

37 Use of test panels

Panels that have been subjected to strength tests should not be used for structural purposes. Panels that have been subjected to stiffness tests only and for which a design racking load has been derived, may be considered satisfactory for further use subject to agreement between the testing authority, client, designer and fabricator.
APPENDIX B

TEST REPORTS AND PUBLISHED WORK PREPARED BY THE AUTHOR FROM WORK CARRIED OUT ON THE RACKING TEST RIG.

Contents

<table>
<thead>
<tr>
<th></th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>B.1 Test reports</td>
<td>B2</td>
</tr>
<tr>
<td>B.2 Published work</td>
<td>B7</td>
</tr>
</tbody>
</table>
B.1 Test Reports


(3) For Masonite Ltd, Shoreham: Small scale tests on Masonite sheathing boards to determine their suitability for racking panels, Sept. 1976.

(4) For FIDOR, London: Small scale tests on fibre building boards to correlate their strengths with alternative mechanical fixings and the effect of wetting, Nov. 1976.


(7) For Cape Insulation Ltd., Deeside: Report on racking tests carried out on bitumen impregnated insulation board sheathed timber framed wall panels, March 1977.


(9) For Guildway Ltd., Guildford: Interim report on testing work on Karlit panelboard sheathed timber framed wall panels, April 1977.


(13) For Cape Insulation Ltd., Deeside: Report on racking tests on B.1.1.B. sheathed timber framed wall panels subjected to large moisture changes, Jan. 1978.


(17) For Finnforests Ltd., London: Report on racking tests on 5mm and 6mm Finflake sheathed timber framed wall panels, November 1980.


(19) For Guildway Ltd., Guildford: Report on racking tests on 2.46m x 2.4m Stenni board sheathed timber framed wall panels, June 1981.
(20) For Guildway Ltd., Guildford: Report on racking test on 2.4m x 2.4m Stenni board sheathed timber framed wall panel with internal plasterboard sheathing, Nov. 1981.


(24) For Karlit Sales Co. Ltd., Leatherhead: A Comparison of results on racking tests carried out on SPF and Hem-fir timber framed panels sheathed with 9mm Karlit mediumboard sheathings, May 1983.

(25) For Finnforests Ltd., London: Report on racking tests carried out on timber framed wall panels sheathed with 9mm and 12mm Finflake waferboard, July 1983.

(26) For Allan Blunn Ltd., London: Report on racking tests carried out on timber framed wall panels sheathed with 3.5mm building board, Aug. 1983.


(29) For Llewellyn Homes Ltd., Eastbourne: Report on racking tests on standard production timber framed wall
panels, June 1981.

(30) For Duripanel (UK) Ltd., Newbury: Report on racking tests carried out on timber framed wall panels sheathed with 8mm Duripanel cement bonded particle board, Aug. 1983.


(38) For Montague L. Meyer Ltd., Weybridge: Report on racking tests carried out on timber framed wall panels sheathed with 9.5mm and 12.5mm plywoods, Feb. 1984.

(40) For Pilkingtons Ltd., Northwich: Report on racking tests carried out on timber framed wall panels sheathed with 6mm GRC Northwich building board, Jan. 1984.

(41) For COFI, Putney: Report on racking tests carried out on timber framed wall panels horizontally sheathed with 9.5mm douglas fir faced plywood, May 1984.


(43) For COFI, Putney: Report on racking tests carried out on timber framed wall panels horizontally sheathed with 7.5mm douglas fir faced plywood, July 1984.


(45) For Wimpey Construction UK Ltd., London: Report on racking tests carried out on timber frame wall panels with a glued 9.5mm moisture resistant plasterboard, Aug. 1984.


(49) For British Standards Institute, London: Design values for timber frame walls based on the results of racking tests carried out at the University of Surrey, Dec. 1986.

B2 Published Work


APPENDIX C

PHOTOGRAPHS OF THE RACKING TEST RIG AND ANCILLIARY EQUIPMENT.

Contents

Figures C1 to C6

Page

C2
to

C7
Figure C1

A general view of the racking test rig showing a 4.3m wall with a large percentage area of openings being racked under a 5kN/stud vertical load. In the foreground are the amplifiers for the LVDTs, the data logger monitoring load cells and LVDTs and the control panel for the hydraulic jacking system. On this panel circuit 1 controls the racking jacks, circuit 2 controls the vertical load jack over the leading stud and circuit 3 controls the remaining vertical load jacks each covering two stud positions. The independent frame carrying all deflection gauges is also clearly visible in the photograph.
Figure C2 - The jacking systems
The upper photograph shows the racking jack loading the leading panel edge through a load cell and a ball, socket and roller fitting allowing freedom in panel rotation. The vertical load jack over the leading stud is off centre to allow horizontal deflections of up to 60mm and loads through a roller train. Also shown is the LVDT monitoring racking deflection. The lower photograph shows a typical vertical load jack acting through a roller train and a distribution beam on to two stud position. The tie rods preventing lateral movement of the panel may also be noted on both sides of the photograph.
Figure C3 - The jacking system

The upper photograph shows the racking load jack at the rear of the wall which tensions two wires through a load cell and distribution beam fitted with a ball and socket connection. The lower photograph shows the sledge fixed to the racking jack by the tension wires which applies the load to the top plate which is bolted to the panel. Also visible is the central vertical brace to the test rig which can be tensioned to align the rig.
Figure C4 - Deflection measurements
The upper photograph shows the LVDT monitoring uplift of the leading stud of the wall. The lower photograph shows the LVDT monitoring sliding at the rear of the wall. Both photographs include the hardwood bottom plate on which the panels are mounted, the holding down bolts and the independent dial gauge frame.
Figure C5 - General details
The upper photograph shows the holding down bolts, with 50mm square washers, and the bolt fixing between panels. The lower photograph shows the LVDTs measuring sheathing rotation.
Figure C6 - Relative movement between sheathing and frame
The photographs show the Demec gauge equipment used to monitor the relative movement of the sheathing and frames during the racking test. The upper Demec point is glued to the plywood whilst the lower point is fixed to a double headed nail in the frame. A 25mm hole is cut in the board to give clearance to the double headed nail.
APPENDIX D

COMPARISON OF THE RESULTS OF THE COMPUTER BASED ANALYSIS WITH THOSE OF STANDARD PANEL TESTS.

Contents

<table>
<thead>
<tr>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1  Singly Sheathed Panels</td>
<td>D-2</td>
</tr>
<tr>
<td>D2  Combined Sheathings</td>
<td>D-10</td>
</tr>
<tr>
<td>Figures D1 - D13 and Tables D1 - D7</td>
<td>D-15 to D-27</td>
</tr>
</tbody>
</table>

-D1-
The results of the computer based analyses of standard panels may be compared directly with the results of the full scale tests carried out in the Summer of 1984 for the Department of the Environment (see Appendix B) because all the materials used in the nail tests to determine connection performance in the analysis were identical to those used in the test panels. Both the load to failure versus deflection graphs at zero and 5kN/stud vertical load and the reduced design values can be compared and in the following pages the results are examined independently for the main combinations of materials. The vertical loads noted in the titles to the graphs refer to the uplift loads which include the 0.2kN frame nails component. In a number of cases the ASTM test response is also plotted although no full scale test data is available for comparison. The tables in this section show the factor governing design (either stiffness (S) or failure (F)) and the difference in results expressed as a percentage of the full scale test performance. In some cases the test design values at 2.5 kN/stud are based on an interpolation of failure loads at zero and 5kN stud. Test evidence indicates that this value would have been exceeded if it had been possible to conduct failure tests under a 2.5 kN/stud load. In making comparisons it is notable that in the full scale tests load/deflection curves and failure results are based on one test and stiffness results on two such that their variability may be as much as ±25% about the mean. In practice, however the variability is less, probably no more than ±10%. Thus, unless specific trends can be noted, it is not possible to distinguish between an error in modelling and test variability if the difference in results is less than 10%.
The analysis models the racking load results very accurately, particularly at zero vertical load where there is agreement for both stiffness and strength. At 5 kN/stud the plots are similar up to 3mm deflection, the computer model then exhibits a smoother approach to failure, having a reduced 5mm stiffness load and an identical maximum load (although the failure deflection for the test was higher, being approximately 50mm). Consequently there is little difference in design performance although the 2.5 kN/stud load case cannot be properly checked as the failure load has been interpolated.

The uplifts predicted by the analysis are low in comparison with test results, however, the response is understandable with little uplift occurring until the vertical load has been overcome. The prediction and test plots run parallel at higher racking deflections which suggests that there is an initial slip in the full scale tests which is not modelled by the analysis. The ASTM type restraint tests are of interest in that the load deflection curve is similar to both the 2.5 and 5 kN/stud vertical load tests up to a deflection of 3mm. The plots then diverge and at failure the ASTM result is 2.27 times that at zero vertical load and 1.69 times that at 5 kN/stud. The design loads are based on stiffness and thus the enhancement achieved in design is less at 1.96 and 1.17 respectively. The uplift is minimal in the ASTM test because of the nature of the restraint. Vertical load is progressively increased in the ASTM test to hold down the leading stud but it is notable that at low racking loads the panel stiffness is not less than that at 5 kN/stud indicating the importance of the leading stud load in the resistance to racking forces.

It is evident from the results of the analysis in stiffness and uplift that behaviour at any vertical load is
essentially similar until the initial effect of the vertical load has been overcome, then the racking load response reduces from the unique curve, which could be represented by the ASTM test, and tends towards the failure load.

(b) Plywood/Red-Whitewood/Nails At 150/300mm Centres (Figure D2 and Table D2)

This test was carried out to check the effect of framing on racking performance. The model has increased in stiffness but not to the full extent implied by the tests. The failure loads have also increased and in general there is less than a 5% difference between the model and the test panels (noting that the test failure at 5 kN/stud vertical load was 20.3 kN at a deflection of approximately 50mm). The load/deflection plots clearly show the difficulties experienced by the load step method of analysis in tracing the very ductile failures, typical of plywood. The increased load carried by the model at mid range deflections is a result of the inability of the Kimber curve to model the reduction of nail load after the maximum load has been achieved. It is noticeable that the relative behaviour of the model and the test panels is different for the redwood/whitewood frames but it was clear during testing that the vertical load behaviour of these panels was abnormal in terms of stiffness, as demonstrated in Figure D.2. The uplift pattern is identical to the previous panels; consequently the plot has not been included.

(c) Mediumboard/SPF/Nails At 150/300mm Centres (Figures D3a and b, Table D3)

The mediumboard comparisons using test M6 indicate a limitation of the computer analysis that was not clearly evident in the plywood test. Here the model is unable to develop the full variation in stiffness measured between the zero and 5 kN/stud tests, it is also unable to show the weak, brittle nature of the panels. This may be due to
inaccuracies in the nail performance curve where a 12% increase was applied to the single nail results based on tests on plywood alone. It is quite possible that the "line of nails" behaviour with mediumboard sheathings will be both stronger and more brittle. As previously noted further tests are required in this area. The inaccuracy in the failure load prediction has resulted in the model overestimating performance by 15% at zero vertical load with a progressive reduction as vertical load increases. A second set of data has been plotted for a weaker mediumboard/frame combination which clearly will not match the model prediction, however, the load/deflection behaviour is more like that of the model. The difference is approximately 20% at zero vertical load reducing to 14% at 5 kN/stud. A further set of tests, M5, (not shown here) used a similar mediumboard with hem-fir frames and 50mm-long 3.25mm clout head nails. The design values of 5.71, 8.23 and 9.66 kN were all based on stiffness and compare well with the analysis, showing less variation with vertical load than the S.P.F. framed panels.

The uplift behaviour is similar to that of the plywood except that the test difference is greater at zero vertical load. The model predicts similar uplifts for both sheathings but in practice the mediumboard uplift is greater than that of the plywood due to the brittle failure of the board when resisting break out of the nails along its bottom edge.

The ASTM plot differs slightly from that of plywood in that it shows an almost immediate improvement in stiffness. At failure the enhancement factors achieved by the ASTM condition over the zero and 5 kN/stud vertical load cases are 1.89 and 1.23 respectively, lower than those for plywood. The improvement factors for design values are 1.83 and 1.19; they are significantly closer to those of plywood and indicate stiffness to be the governing factor.
(d) B11B/SPF/Nails At 75/150mm Centres
(Figures D4 a and b, Table D4)

The computer analysis has difficulty in modelling the B11B test results, firstly it fails to achieve the variation in stiffness with vertical load and secondly it overestimates failure. As with the mediumboard it is likely that the estimate of in-line nail performance is wrong and this could be compounded by the close spacing of the nails. It is probable that the failure performance has been overestimated because the Kimber curve cannot show, with sufficient accuracy, the drop off in nail load; reduction of the bang value for nails to 6.0mm (as used for plasterboard) could reduce performance by up to 20%.

The lack of differentiation between initial panel stiffnesses at varying vertical loads, previously noted, is clearly illustrated in the analytical results whereas, in practice the performance curves would be different even at small racking deflections.

The uplift results were again inaccurately modelled. The improvement in accuracy at higher racking deflections is due to the increased load capacity predicted by the analysis. The uplifts in general are less than those for plywood and mediumboard due to the lower loads carried by the panel at any given value of racking deflection. In practice the uplifts are low because the board has little resistance to nail movements which allows board rotation without significant uplift.

The ASTM results were different in comparison with the stronger boards. At failure the improvement factors compared with the zero and 5 kN/stud test failure loads were similar at 1.9 and 1.27 respectively but the design loads were substantially reduced showing improvement factors of 1.42 and 1.02. It is evident that in weaker sheathings stiffness becomes increasingly dominant and that the improvement in stiffness with vertical load decreases. This may be a result, not only of weakness in resistance to
nail movement, but also of low shear rigidity. It was noted during the parametric study that an improvement in shear modulus principally increased the early stiffness of panels at higher vertical loads and enhanced the modelling of the test data. However, it must be noted that the shear modulus of B11B was an estimate based on limited information.

(e) Plasterboard/SPF/Nails at 150/300mm Centres (Figures D5 a and b, Table D5)

The computer analysis provides a poor model of the racking resistance of plasterboard because the calculated failure performances are low in comparison to tests and failure load is paramount in the calculation of the design load of plasterboard. In the single specimen cover nail tests the nail performance was as expected with a very brittle failure occurring soon after 6.0mm deflection in the "perpendicular to grain" test. The results were very consistent and a failure load of 0.5 kN was achieved by the "parallel to grain" nail where the response was less brittle. It is unlikely that the use of a higher value for "bang" on the "parallel to grain" nail would have affected the analysis because the nails experience only small deflections in this direction. It is more likely that the nail tests have not given a true indication of the capacity of the nail in the racking panel because insufficient restraint has been given to the board to prevent it bending. As plasterboard is very brittle in bending any small rotation of the board could cause its premature failure which would substantially effect nail performance. Thus the enhancement achieved by using a line of nails in resisting "parallel to grain" forces is likely to be much higher than noted for the plywood (and applied to the plasterboard nail). Furthermore bending caused by out of line forces may also have affected the "perpendicular to grain" performance, where in practise the board is better supported by the frame members acting in the line of the applied load. It is unlikely that the rough estimate of
the shear modulus of the board had any affect on the predicted performance for two reasons. Firstly the initial stiffness of the model matched that of the tests and secondly the same analysis conducted on a 9.5mm board using the same nail parameters gave almost identical results to the thicker board; thus the shear resistance of the board was high in comparison with the shear resistance due to the nails. Because no small scale nail tests were carried out on the 9.5mm board it is not possible to make further comment on the predictions of the analysis.

The uplift plots indicate the analysis to underestimate uplift both initially and at failure. It is probable that in the latter case this is due to the low racking load sustained by the panel. However, it is noticeable that the uplift recorded under a 5 kN/stud load in the test is not dissimilar to the uplift predicted at 2.5 kN/stud. This result is also true of the previous tests and may indicate the vertical load to be carried only in part through the frame studs.

In design, failure governs both the test results and those of the analysis. As a result the test performance is always underestimated by the analysis. However, it is clear that the pattern of the results is similar to that of the previous boards whereby the analytical results reduce in comparison with test values as vertical load increases.

The dominance of the failure performance has also affected the ASTM test result. Here the enhancement achieved by the stronger holding down method is the same for both design and failure loads. The improvement factors are 2.04 and 1.29 compared with zero and 5 kN/stud results respectively.
(f) Plywood/SPF/Nails At 75/150mm Centres
(Figures D6 a and b, Table D6)

The correlation between the analytical results and those of the full scale tests is not as good as that achieved by the same materials with the nails spaced at 150/300mm centres. The failure loads obtained by the analysis are all high and the range in stiffness, as normal, is low, however, here the stiffness at 5 kN/stud is accurately predicted which results in the zero kN/stud stiffness being overestimated. It can be seen (Table D.6) that in the full scale tests all design values are based on failure results whereas this is true at zero vertical load only for the analysis. It is notable that if stiffness had governed then design values would have been closer. The percentage difference would have been reduced by 8% in all cases.

The uplift behaviour of the computer models is similar in comparison with previous results. The maximum uplift is slightly underestimated but the error would probably be greater if the racking load had not been overestimated.

The ASTM results are very similar to those obtained for the standard nailing of plywood. However, it is noticeable with the stronger panels that the vertical load enhancement reduces and thus in relation to the zero vertical load design values the enhancements at 5 kN/stud and under ASTM conditions are 1.42 and 1.72 respectively compared with 1.68 and 1.96 for the standard nail spacing analysis.
The results of the tests on combined sheathings have been analysed separately from the other standard panel tests for the following reasons:

(i) the model is less precise in its interpretation of the panels because it does not include two separate sheathings,

(ii) the materials used do not always allow direct comparison,

(iii) the results are used to show the effect of the cover nail tests on the panel analysis.

Three analyses were carried out on plasterboard combined with plywood, mediumboard and B11B. In the first case the results could be directly compared with test data but in the second case the full scale tests had been carried out four years earlier using different materials known to give a weaker racking performance. The B11B model could not be compared directly as it required an identical nailing pattern to be used for both boards (in the analysis this was 150/300mm) whereas in the tests both boards were fixed using their standard nailing patterns (75/150mm for B11B). Noting these discrepancies, the results are compared in Figures D7 to D9 and in Table D7. The nail tests for the three combinations are given in Figure D10 to D13.

Comparing the plywood results (Figure D7) it can be seen that the analysis overestimates failure and does not achieve adequate stiffness at high vertical loads which results in stiffness governing the design. Consequently design predictions using the analysis exceed the test values at low vertical loads but not at 5 kN/stud where the test panel is initially very stiff.
The mediumboard results, which use different Kimber curves for the cover nails, differ from the plywood results in that the initial stiffness of all curves, in particular that at 5 kN/stud, are much higher. The failure loads, however, are similar. The full scale test results are weaker than predicted by the analysis due to the change in material but the load/deflection behaviour is very much closer in appearance. The plywood full scale tests have also been plotted on this graph and show a close resemblance to the analytical model in the initial 10mm of racking deflection. The reasons for the difference can be explained by the nail test results (Figure D10 and D11); The following points are notable.

(i) For mediumboard the "parallel to grain" nail performance is initially much stiffer although the maximum load is identical in comparison with that of the plywood.

(ii) For the mediumboard the "perpendicular to grain" performance is very similar to that of plywood except that the failure load is 7% higher.

(iii) The ;bang; values for both boards have been based on the overall performance of the nails giving values of 8 and 9mm for the mediumboard and plywood respectively. However, in both cases there is an initial loss of load after 6mm which might prove critical in practice; ;bang; should therefore be reduced to 6mm in both cases.

Point (i) indicates that better modelling of initial panel stiffness at all vertical loads will be achieved if "parallel to grain" nail test stiffness is increased. Because such tests were carried out with single nails, it is probable that the interaction of nails in rows,
preventing out of plane board movements, would achieve this aim although the improvement would need to be greater than the 12% already included, based on the results of nail tests in plywood alone (see section 7.4.3). Point (ii) has little effect on results except to slightly enhance the failure load for the mediumboard at zero vertical load. Point (iii) has the effect that the failure load, and racking deflections at failure, for both sheathing combinations will be higher than in practice. This result gives further evidence that the value of "bang" is very important to the model performance because the Kimber curve cannot model the nail behaviour after the maximum load has been reached. However, it may therefore be necessary to determine the final solution for "bang" by trial and error based on the full scale results.

The B11B results should not be directly compared due to the change in nailing, however, small scale tests have since been carried out using one nail in the plasterboard and two in the B11B to simulate the different spacings used in full scale panels. The results are shown in figure D13. The improvements are likely to be greater than will be experienced in practice as the two nails in the B11B should be stronger than twice the single nail value due to their interaction. A better test would be to use the modified test with three nails in the plasterboard together with six nails in the insulation board. However the results indicate that the nail curve should be enhanced in the "parallel" and "perpendicular to grain" cases by 25% and 50% respectively. The value of "bang" is correct for both nail spacing situations. These alterations should substantially improve the analytical model although the required increase in failure loads, approximately 20%, infers that a 50% increase in "perpendicular to grain" performance will be excessive.

No conclusions have been drawn in this investigation concerning the individual behaviour of the
two sheathings on the panel since no direct relationship was detected in the standard panel tests (section 6.4.3) between the main sheathing and the combined sheathing cases and in the analysis the plasterboard results, have already been shown to be unreliable. However, an analysis has been made of the nail test results whereby the performance of the individual boards have been summed and compared with the results of the combined case. The results are shown in Table D8 and may be used to explain some of the discrepancies noted in the results of the combined sheathing analysis. In general, the sum of the parts lies within 10% of the combined performance. The main exception is in the initial stiffness of the "parallel to grain" nail tests in plywood. It is notable that the nail curve used in the design is very much less stiff than would have been predicted from the behaviour of the parts and the results of the computer analysis show the model also to be lacking in initial stiffness. It may therefore be concluded that the nail tests on the combined sheathings were inaccurate.

The summation of the individual nail loads to give those of the combined sheathing case could be used effectively to predict the performance of other sheathing combinations without the need for further tests, for instance double sheathings of the same board or B11B plus mediumboard, which is commonly used in Scandinavia.

Changes in nail type between materials could be covered by this method and it may also be possible to consider nail spacing. Thus in the case of plasterboard plus B11B the required nail curve for the combined sheathings could be predicted by summing the plasterboard nail performance values with two times the B11B values (noting the standard B11B spacing to be half that of plasterboard). The results of this prediction are compared with test values in Table D9 and a satisfactory correlation is noted which also confirms the likelihood that too high a "perpendicular to grain" nail result was used in the analysis.
In this examination, loads within a 5mm deflection range have been monitored. Previous results in this section have shown that modelling would have been improved if the "bang" value had been reduced to be closer to that of plasterboard. It is clear that for normal combinations of boards the summation of the nail curves will show a drop in load when the more brittle board reaches its "bang" deflection and that this value should then be used for the combination. Thus if the "bang" value for plasterboard is accurately fixed, failure behaviour of the combined sheathing panel should be modelled with a reasonable degree of accuracy, but noting that the analysis could not be expected to interpret the effect of the sudden redistribution of load which would be likely to weaken the panel.
Figure D1 Standard Panel Performance Comparison: Plywood Nailed At Standard Centres
Figure D2 Standard Panel Performance Comparison:
Plywood Nailed At Standard Centres to Red-White Wood
Figure D3 Standard Panel Performance Comparison: Mediumboard Nailed At Standard Centres

(a) Racking Load Versus Racking Deflection

(b) Uplift of Leading Stud Versus Racking Deflection
Figure D4 Standard Panel Performance Comparison: B11B Nailed At Standard Centres
Figure D5 Standard Panel Performance Comparison: Plasterboard Nailed At Standard Centres

(a) Racking Load Versus Racking Deflection

(b) Uplift of Leading Stud Versus Racking Deflection
Figure D6 Standard Panel Performance Comparison:
Plywood Nailed At Close Centres

---D20---
<table>
<thead>
<tr>
<th>Vertical Load (kN/stud)</th>
<th>Design Racking Load (kN)</th>
<th>Percentage Difference in Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Computer Analysis</td>
<td>P7 (Table 6.1a)</td>
</tr>
</tbody>
</table>
| 0                      | 5.67 F            | 5.55 S               | 5.92 S | -1%  
| 2.5                    | 8.25 F            | 7.95* F              | 8.23* F | +2%  
| 5                      | 9.48 S            | 9.75 F               | 9.96 S | -4%  

* Interpolated value

Table D.1 2.4m Standard Panel Design Values: 9.5mm Plywood/SPF/Nails at 150/300mm Centres

<table>
<thead>
<tr>
<th>Vertical Load (kN/stud)</th>
<th>Design Racking Load (kN)</th>
<th>Percentage Difference in Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Computer Analysis</td>
<td>Full Scale Tests</td>
</tr>
</tbody>
</table>
| 0                      | 6.52 S            | 6.36 F               | +3%  
| 2.5                    | 8.53 S            | 8.26* F              | +3%  
| 5                      | 9.57 S            | 10.02 S              | -4%  

* Interpolated value

Table D.2 2.4m Standard Panel Design Values: 9.5mm Ply/Red-Whitewood/Nails At 150/300mm Centres

<table>
<thead>
<tr>
<th>Vertical Load (kN/stud)</th>
<th>Design Racking Load (kN)</th>
<th>Percentage Difference in Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Computer Analysis</td>
<td>Strong Mediumboard</td>
</tr>
</tbody>
</table>
| 0                      | 6.19 F            | 5.39 F               | 4.51 S | +15%  
| 2.5                    | 8.06 F            | 7.60* F              | 6.59 S | +5%  
| 5                      | 9.52 F            | 9.97 F               | 8.32 S | -4%  

* Interpolated value

Table D.3 2.4m Standard Panel Design Values: 9.0mm Mediumboard/SPF/Nails At 150/300mm Centres

<table>
<thead>
<tr>
<th>Vertical Load (kN/stud)</th>
<th>Design Racking Load (kN)</th>
<th>Percentage Difference in Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Computer Analysis</td>
<td>Full Scale Tests</td>
</tr>
</tbody>
</table>
| 0                      | 3.31 S            | 2.95 F               | +12%  
| 2.5                    | 4.40 S            | 3.99* F              | +10%  
| 5                      | 4.62 S            | 5.04 F               | -8%  

* Interpolated value

Table D.4 2.4m Standard Panel Design Values: 12.5mm B11B/SPF/Nails At 75/150mm Centres

-D21-
<table>
<thead>
<tr>
<th>Vertical Load (kN/stud)</th>
<th>Design Racking Load (kN)</th>
<th>Percentage Difference In Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Computer Analysis</td>
<td>Full Scale Tests G4 (Table 6.5)</td>
</tr>
<tr>
<td>0</td>
<td>2.25 F</td>
<td>2.82 F</td>
</tr>
<tr>
<td>2.5</td>
<td>3.28 F</td>
<td>4.04* F</td>
</tr>
<tr>
<td>5</td>
<td>3.56 F</td>
<td>5.26 F</td>
</tr>
</tbody>
</table>

* interpolated value

Table D.5 2.4m Standard Panel Design Values: 12.5mm Plasterboard/SPF/Nails At 150/300mm Centres

<table>
<thead>
<tr>
<th>Vertical Load (kN/stud)</th>
<th>Design Racking Load (kN)</th>
<th>Percentage Difference In Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Computer Analysis</td>
<td>Full Scale Tests P15 (Table 6.1b)</td>
</tr>
<tr>
<td>0</td>
<td>9.56 F</td>
<td>7.81 F</td>
</tr>
<tr>
<td>2.5</td>
<td>12.18 S</td>
<td>10.00 F</td>
</tr>
<tr>
<td>5</td>
<td>13.60 S</td>
<td>12.20 F</td>
</tr>
</tbody>
</table>

Table D.6 2.4m Standard Panel Design Values: 9.5mm Plywood/SPF/Nails At 75/150mm Centres

<table>
<thead>
<tr>
<th>Sheathing Type</th>
<th>Vertical Load (kN/Stud)</th>
<th>Design Racking Load (kN)</th>
<th>Percentage Difference In Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Computer Analysis</td>
<td>Full Scale Test</td>
</tr>
<tr>
<td>Ply and Plasterboard</td>
<td>0</td>
<td>6.45 F</td>
<td>5.99 F</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>9.73 S</td>
<td>8.65* F</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>10.60 S</td>
<td>11.32 F</td>
</tr>
<tr>
<td>Mediumboard and Plasterboard</td>
<td>0</td>
<td>7.03 F</td>
<td>5.56 F</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>9.84 F</td>
<td>8.30* F</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>11.95 F</td>
<td>11.04 F</td>
</tr>
<tr>
<td>B11B and Plasterboard</td>
<td>0</td>
<td>4.10 F</td>
<td>4.98 F 1.</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>5.74 F</td>
<td>6.98 F 1.</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>7.03 F</td>
<td>8.99 F 1.</td>
</tr>
</tbody>
</table>

* interpolated result

1. Nails in B11B at 75/150mm centres (otherwise all nails at 150/300mm centres).

Table D.7 Comparison Of Results For Combined Sheathing Tests
Figure D.7 Standard Panel Performance Comparison:
Plywood Plus Plasterboard Sheathings

Figure D.8 Standard Panel Performance Comparison:
Mediumboard Plus Plasterboard Sheathings
Figure D.9 Standard Panel Performance Comparison: B11B Plus Plasterboard Sheathing
Figure D.10 Sheathing Nail Tests on Combined Ply and Plasterboard, Nails at 150/300 Centres

Shear parallel to grain of stud
\[ y = ax^e \cdot bx^c \]
- \( a = 1655 \)
- \( b = 0.63 \)
- \( c = 0.5 \)
- bang = 12mm
+ = mean data line

Shear perpendicular to grain of stud
\[ y = ax^e \cdot bx^c \]
- \( a = 4005 \)
- \( b = 1.53 \)
- \( c = 0.35 \)
- bang = 9mm
+ = mean data line

Figure D.11 Sheathing Nail Tests on Combined Mediumboard and Plasterboard, Nails at 150/300 Centres

Shear parallel to grain of stud
\[ y = ax^e \cdot bx^c \]
- \( a = 13590 \)
- \( b = 2.31 \)
- \( c = 0.25 \)
- bang = 9mm
+ = mean data line

Shear perpendicular to grain of stud
\[ y = ax^e \cdot bx^c \]
- \( a = 1847 \)
- \( b = 0.816 \)
- \( c = 0.50 \)
- bang = 8mm
+ = mean data line
Shear parallel to grain of stud

\[ y = ax^{-b}x^c \]

- \( a = 2235 \)
- \( b = 1.53 \)
- \( c = 0.35 \)

bang = 6mm

+ = mean data line

**Figure D.12** Sheathing Nail Tests on Combined BILB and Plasterboard, Nails at 150/300 Centres

---

Shear perpendicular to grain of stud

\[ y = ax^{-b}x^c \]

- \( a = 2235 \)
- \( b = 1.53 \)
- \( c = 0.35 \)

bang = 6mm

+ = mean data line

Note

156 and 157 are BILB above

---

Shear parallel to grain of stud

**Figure D.13** Sheathing Nail Tests on Combined BILB and Plasterboard Nails at 75/150 and 150/300 Centres
### Table D.8 Nail Tests on Combined Sheathings: Summation of Individual Performance Compared With the Measured Value For the Combination

<table>
<thead>
<tr>
<th>Nail Orientation</th>
<th>Sheathings</th>
<th>Nail Load (kN)</th>
<th>For Nail Deflection (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Parallel to Grain</td>
<td>Ply Plasterboard</td>
<td>0.64</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>Total Ply &amp; Plaster</td>
<td>1.17</td>
<td>1.53</td>
</tr>
<tr>
<td>Perpendicular to Grain</td>
<td>Ply Plasterboard</td>
<td>0.64</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>Total Ply &amp; Plaster</td>
<td>0.87</td>
<td>1.10</td>
</tr>
<tr>
<td>Parallel to Grain</td>
<td>Mediumboard Plasterboard</td>
<td>0.91</td>
<td>1.17</td>
</tr>
<tr>
<td></td>
<td>Total MDF &amp; Plaster</td>
<td>1.24</td>
<td>1.60</td>
</tr>
<tr>
<td>Perpendicular to Grain</td>
<td>Mediumboard Plasterboard</td>
<td>0.66</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td>Total MDF &amp; Plaster</td>
<td>0.87</td>
<td>1.10</td>
</tr>
<tr>
<td>Parallel to Grain</td>
<td>B11B Plasterboard</td>
<td>0.22</td>
<td>0.32</td>
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<tr>
<td></td>
<td>Total B11B &amp; Plaster</td>
<td>0.55</td>
<td>0.75</td>
</tr>
<tr>
<td>Perpendicular to Grain</td>
<td>B11B Plasterboard</td>
<td>0.19</td>
<td>0.27</td>
</tr>
<tr>
<td></td>
<td>Total B11B &amp; Plaster</td>
<td>0.42</td>
<td>0.59</td>
</tr>
</tbody>
</table>

### Table D.9 Results of Nail Tests For B11B/Plasterboard Sheathing Combination With Nails At Standard Spacings

<table>
<thead>
<tr>
<th>Nail Orientation</th>
<th>Design Result</th>
<th>Nail Load (kN)</th>
<th>For Nail Deflections (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Parallel to Grain</td>
<td>Predicted</td>
<td>0.77</td>
<td>1.07</td>
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<tr>
<td></td>
<td>Measured</td>
<td>0.75</td>
<td>1.10</td>
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<tr>
<td>Perpendicular to Grain</td>
<td>Predicted</td>
<td>0.61</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>Measured</td>
<td>0.65</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Nail spacings: Plasterboard 150/300mm  
B11B 75/150mm