THE PERFORMANCE OF TIMBER STRUCTURES, ELEMENTS AND COMPONENTS

Vahik Enjily
Centre for Timber Technology and Construction (CTTC)
Building Research Establishment (BRE)
UK

1. INTRODUCTION

This paper has been prepared for COST Action E24 for consultation and information purposes only. The main aim of this paper is to highlight and summarise typical behaviour and performance of timber structures, elements and components.

This paper includes the following topics:

- Behaviour of medium-rise timber frame buildings in disproportionate (progressive) collapse.
- Whole building racking stiffness of medium-rise timber frame buildings.
- Racking resistance of wall panel elements.
- Performance of trussed girder components under eccentric loading.
- Performance of trussed rafter roofs containing a 3-ply girder (based on original design)

The information given in this paper is not intended to be exhaustive. It is intended to highlight typical performance and modes of failures which should be considered in the reliability analysis of timber structures.

2. BEHAVIOUR OF MEDIUM-RISE TIMBER FRAME BUILDINGS IN DISPROPORTIONATE (PROGRESSIVE) COLLAPSE

2.1 Summary of the TF2000 project

The well documented Timber Frame 2000 project provided a six-storey experimental timber frame building (Figures 1, 2 & 3) for the sole purpose of investigating the performance and economic prospects of medium-rise timber frame buildings in the UK. The majority of associated research projects conducted on this building have been completed and the results published and presented on different international forums.

Figure 1: The Six-storey TF2000 Building at BRE Cardington

Timber frame buildings of up to four storeys are being used quite extensively in the UK, however no comprehensive design rules exist for five storeys or more. As a result BRE and TTL carried out a joint feasibility study [2] on such buildings and subsequently built a six storey experimental building in partnership with the UK industry and DETR. The feasibility study, construction of the experimental building and later testing of the building were all conducted as part of the Timber Frame 2000 project (TF2000) with the objective of providing the technical tools and commercial drivers for the safe and economic construction of medium-rise timber frame buildings in the UK.

Consequent to the feasibility study's findings and conclusions, the TF2000 project started in October 1995, preparing the way for proposed tests on a test building. The TF2000 project entailed a two-phase investigation. Phase I [3], provided the background study and justification to recommend and approve programmes of work (Table 1) for Phase II along with the detailed design of a full-scale medium-rise building to be used as a test-bed. Phase II comprised the construction and testing of the building and the preparation of authoritative guidance for the design of medium-rise timber frame structures.

TEST PROGRAMMES
• Value Engineering & Process Engineering
• Differential Movement.
• Whole building lateral (racking) stiffness.
• Disproportionate Collapse.
• Fire – Compartmentation & Stairs.
• Acoustics
• Guidance Documents.

Table 1: Test Programmes

Figure 2: Completed construction stage after 17 days

Figure 3: Prefabrication (roof segment on site & wall panels off-site)

2.2 Engineering design & specification

The following sets out the basis for the preliminary calculations and the principles of the TF2000 building:

Raft concrete foundation was used for the building. The preliminary structural calculations were based on BS5268: Part 2: 1996[5]. BS5268: Part 6: Section 6.1: 1996[6] was used as a basis for judgements concerning the timber frame wall designs. BS5268: Part 3: 1985[7] was used to indicate the roof structure. Parallel design checks to applicable sections of DD ENV 1995-1-1: 1994[8] and its National Application Document (NAD) was carried out. The following were used for the preliminary calculations:

- BS6399: Part 1: 1984[9] - Code of practice for dead and imposed loads.
- BS6399: Part 2: 1995[10] - Code of practice for wind loads.
- BS6399: Part 3: 1988[11] - Code of practice for imposed roof loads.
- CP3: Chapter V: 1972[12] - Basic wind data for the design of buildings.

Strength Class C16 to BS EN338[13] was used throughout the calculations, in order to ensure that the tests on the structure will demonstrate the performance of such a building when using softwood from any suitable supply source (including the UK.). In consideration of the desirability of minimising differential movement, a decision was taken to designate special target moisture content at the time of erection. This was 12% ± 2% for floors and 18% ± 2 for walls.

The following general specifications for key elements were proposed and agreed (Figure 4):

Figure 4: Brick cladding under construction

- The ground floor is notionally a concrete slab-on-ground. Walls consist of two layers of plasterboard with a vapour control layer and 89 x 38 mm C16 timber studs with mineral wool insulation in between. The sheathing is 9 mm OSB, Type F2 (OSB3). The cavity is 60 mm with single leaf brick cladding tied with stainless steel ties to timber frame.
- Internal load bearing walls consist of C16 timber studs with two layers of plasterboard and 9 mm OSB, Type F2 sheathing to one side, where needed for wind bracing. The internal non-load bearing walls consist of C16 timber studs with one layer of plasterboard to each side. The compartment walls are twin leaf with C16 timber studs and mineral wool insulation in between. OSB Type F2 sheathing is used where wind resistance is required.
- Compartment floors consist of two layers of plasterboard ceiling lining and C16 joists with mineral wool in between. OSB Type F2 is used as a floor deck. Floating floors contain proprietary resilient battens with plasterboard and Type P4 chipboard.
- The roof is the trussed rafter type with hipped ends supporting concrete interlocking tiles.

More information is given in reference 14.

2.3 Robustness and Disproportionate Collapse

Robustness is the principle of providing additional strength against potential and unforseen events. It is not a case of over-design but a primary responsibility of a building engineer to produce a design solution that can resist catastrophic failure that might occur due to small incidents or misuse. The engineer avoids over design through skills in selecting design solutions that are economic and practical to ensure that people in a building are safe within the designated area of influence. Poor robust design can be over cautious and impractical to the scale of the project. While there are UK building regulation requirements for the minimum levels of robustness, additional robustness may be justified in the value of the building: for example the cost associated with redress of damage and subsequent restoration.

The forthcoming Euro Codes formally recognises robustness as a design process and clearly clarifies that accidental load conditions are associated with design and material factors less onerous than those used in normal design process. The primary Euro Code - EC1: Part 1 addresses all buildings and presents alternative methods of approach to limit or avoid poor robustness design:

- Avoiding, eliminating or reducing the hazards which the structure may sustain.
- Selecting a structural form which has low sensitivity to the hazards considered.
- Selecting a structural form and design that can survive adequately the accidental removal of an individual element or a limited part of a structure, or acceptable localised damage.
- Avoiding as far as possible structural systems which may collapse without warning.
- Tying the structure together.

The UK building regulations addressees robustness under the term disproportionate collapse. To-date the UK building regulations Part A- Structure, requires that ***"The building shall be constructed so that in the event of an accident, the building will not suffer collapse to an extent disproportionate to the cause***". It also requires (to-date) that buildings only above four storeys are to include in the design process a check to ensure robustness of the structure against collapse or allow localised damage when subjected to a defined level of accidental damage. It is being considered however that the disproportionate rules will be applied to all heights of buildings within the near future.

The building regulations accept localised damage as being inevitable under certain accidents but limits this to 15% of the area of the storey or $70m^2$ (which ever is less) within that storey and immediately adjoining storey above.

It is not important to understand what the accident is but rather that defined length or elements of the structure that can be removed in turn and to investigate theoretically what the consequence of the removal does to the remaining structure.

The timber frame fabrication process, both volumetric and panel construction, provides a significant amount of connectivity through site and factory nailing. The "common sense" approach is that timber frame panels offer significant inherent robustness and capacity to span over removed panels – particularly in the short term periods of accidental damage. Hence it is appropriate for timber frame construction to addresses robustness by the concept of notional removal of defined structural panel lengths – one at a time on each storey level and in turn check the stability of the remaining structure.

What are the defined timber frame structural elements to be removed?

- Load bearing walls or a column/post
- Wall length is length between intersecting walls (party walls, return walls, etc.)
- Panels between column/post
- Minimum length of 2.4m

Collapse in disproportionate checking procedures is considered to be failure of structural elements when the following is exceeded:

- Floor/wall vertical deflections greater than L/30 (including total collapse) or if the layout and function of the element under consideration becomes so high that the members are over stressed or that safe egress from the building is not possible.
- Once a 'support member' has been removed, one at a time on each storey in turn, then the area of failure of the storey and immediate adjacent storeys is limited to the lesser of:
  A 15% of the floor area of the storey
  B $70m^2$

Area of storey = the whole building storey floor area (building foot print)

England/Wales and Scotland/Northern Ireland Building Regulations and Standards all refer to the need for disproportionate collapse robustness for buildings above four storeys. The objective of this part of the programme of tests was to enable an evaluation of the actual behaviour of the TF2000 building when selected vertical load bearing wall panels are removed. This evaluation was to verify that the inherent stiffness of cellular platform timber frame construction could provide the necessary robustness so that, in the event of an accident, the building would not suffer collapse to an extent disproportionate to the cause. This is best achieved by designing in such a way that a beam, column or section of wall can be removed without the structure above collapsing (although damage to the building is allowed). To achieve this, beams are incorporated within floor depths over external walls, or the walls themselves are made to act as beams.

The building was loaded using weighed sandbags positioned on each floor above the full flat area where the tests were carried out in similar manner to that set out in the other material codes for checking disproportionate collapse. The following rules were defined for the TF2000 building:

Duration	Vertical Deflection (mm)	
	Floor	Wall above
For internal wall removed:		
30 minutes	13	No significant movement
4 hours	19	No significant movement
20 hours	26	No significant movement
For external wall removed:		
30 minutes	2.5	1.2
4 hours	3.7	2.4
20 hours	4.0	2.6
No cracking of masonry cladding		

Table 2 - Summary of disproportionate collapse results

- The horizontal length of any load-bearing wall to be notionally removed in any given instance is the length between intersecting internal/return walls or special vertical support system.
- A full analytical review of the building was undertaken to predict the effect of removing each of the load bearing wall panels so that the 'worse case' could be chosen for the tests. For maximum vertical load effects and practicality, ground floor panels were chosen.
- Two main areas of the building were subjected to the notional removal of walls, namely: (a) an internal wall and (b) a part of an external wall, between return walls (Figure 5).
- The agreed limit level of serviceability (a function of the practical limit of deflections) for floor spans of 3600mm was chosen to be 300mm (approx. L/12).

Table 2 shows a summary of the results and more information can be found in reference 17.

Figure 5: Disproportionate Collapse Test

2.3.1 Summary of the Principle of Disproportionate Collapse Check

- To ensure that safe egress of people can be achieved when a building has a structural problem (however caused) which is not disproportionate to the cause.
- To ensure that the building can sustain safe access for temporary propping which might be considered essential for public safety.
- Stability of the building under the above two conditions shall be ensured for a minimum 24 hour period.

Cladding -

- Cladding is seldom structural although failure through total collapse would be a risk to public safety.
- The area of masonry based cladding considered acceptable for collapse is 2.25 times the storey height. Guidance on this is available in the masonry code BS 5628.
- The area of timber cladding or other lightweight cladding shall be the same as the timber frame.

Structural loading under the check -

- Full dead load (including any finishes or fixed plants).
- 1/3 improved floor load.
- 1/3 imposed roof load.
- No wind load checks are required.

Structures below timber frame -
Where ground or other lower storeys are constructed from other structural materials, i.e. concrete, masonry or steel frame. The effect of loss of support from the underlying structure on the timber frame shall also be considered. Co-ordination of the removal or deflection of support structure must be allowed for the timber frame check.

Timber design issues under robustness design process -
In line with the principles of the Euro Code principles the design strength of timber can be taken with a duration of load factor of double the working stress values eg for BS5268 part 2 design code K3=2 for stress checking. In addition mean MOE values may be assumed for single/double members. Where steel and timber connections are used, twice basic working stress design values may be adopted.

Wall panels

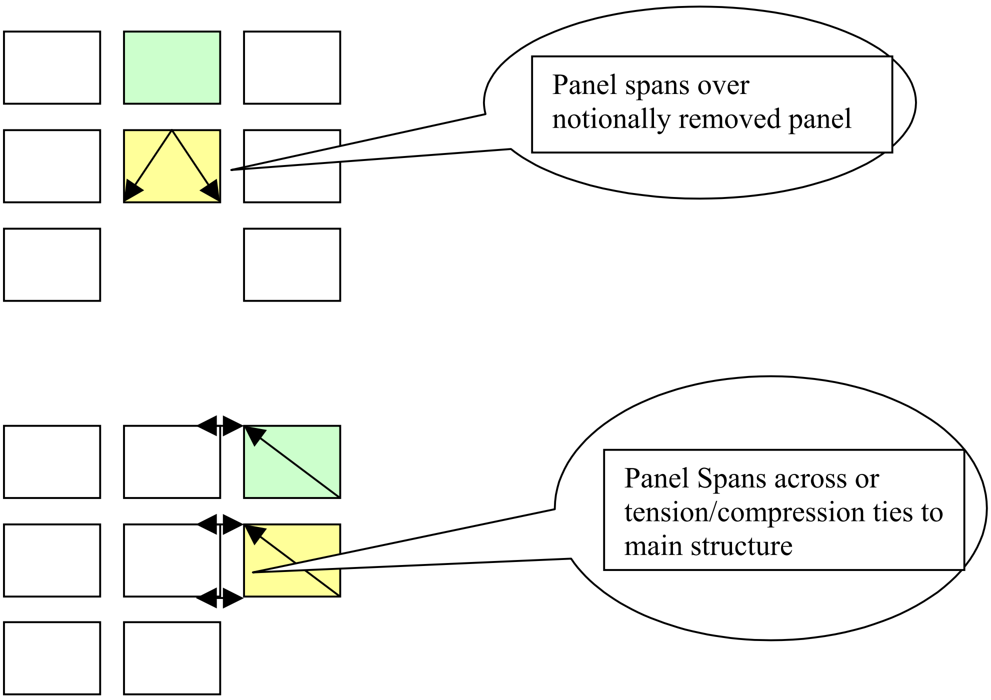

Figure 6: Elevation panels spanning over a gap

Most wall panels have sufficient stiffness created by the plasterboard/timber sheathing to span moderate distances. A structural check on a theoretical deep beam action of a panel under accidental loads will easily demonstrate this assuming of course that there is continuity of the panel’s top and bottom plates (Figure 6).

In some applications additional support to walls and floors can typically be found within the typical detailing of timber frame in the form of continuous or cantilever header joists and/or rim beams (Figure 7).

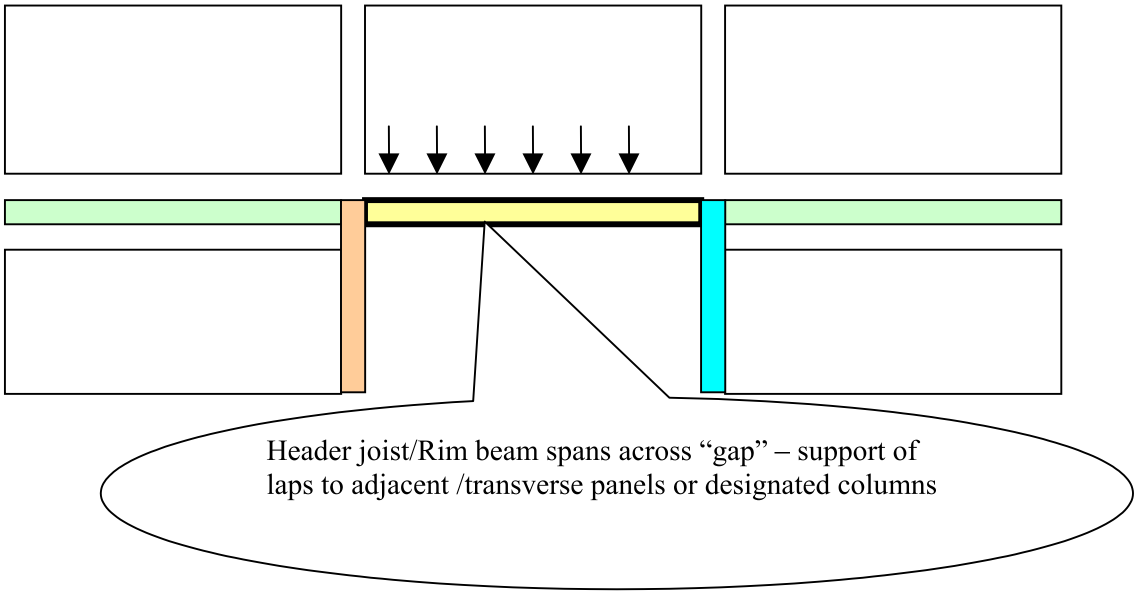

Figure 7: Elevation on joist/rim beam spanning over a gap

Tests at BRE Cardington have shown that floors subject to loss of 'central' support and where built in to the perimeter walls (as in floor cassettes in platform frame) will have significant built-in support from the walls running parallel to the joist span. This was shown to occur in 2:1 aspect ration of width to span.

If the walls provide support to floors then a check on the capacity to hold the floor through specified skew nailing shall be made or hangers provided to support the resultant loads.

Where a discrete column is used then this shall be designed as a key element in the same manner as steel framed buildings and the following design criteria should be adopted:

- The column shall be designed under accidental loads and deflection limits (L/30) to resist 34kN/m^2 over the length of the column.
- No lateral restraint provided by the removed panel – but restraint may be available from others that are not within the area of influence of the notionally removed panel.

2.3.2 Conclusions

It was recognised that it is not simple to develop mathematical models for engineers to check compliance to disproportionate collapse criteria. More reliability-based design research is needed to advance engineering models for prediction of actual behaviour.

More information can be found in reference 17.

3. LATERAL STABILITY

3.1 Whole Building racking stiffness of medium-rise timber frame buildings

One of the many items of interest in the construction of the TF2000 is the contribution of the various building elements (e.g. plasterboard, brick cladding, etc.) to the overall stiffness of the structure. However, measuring the overall stiffness of a six-storey building is not a simple task, and dynamic testing was selected as an indirect way of determining the required information.

Measurements of the frequencies of the building were made using a laser system (Figure 8) to monitor the ambient response of the structure, i.e. its natural vibration caused by air movement within the BRE Cardington Hangar. These measurements are processed to produce an autospectrum, which is used to identify the frequencies of the modes of vibration. The laser has the advantage that the measurements can be made remotely from the building.

More comprehensive forced vibration tests (Figure 8) were also undertaken at a number of key stages. These are used to determine all of the characteristics of the fundamental modes of vibration, i.e. frequency, mode shape, stiffness & damping. This provides detailed information but required the use of a vibration generator attached to the building.

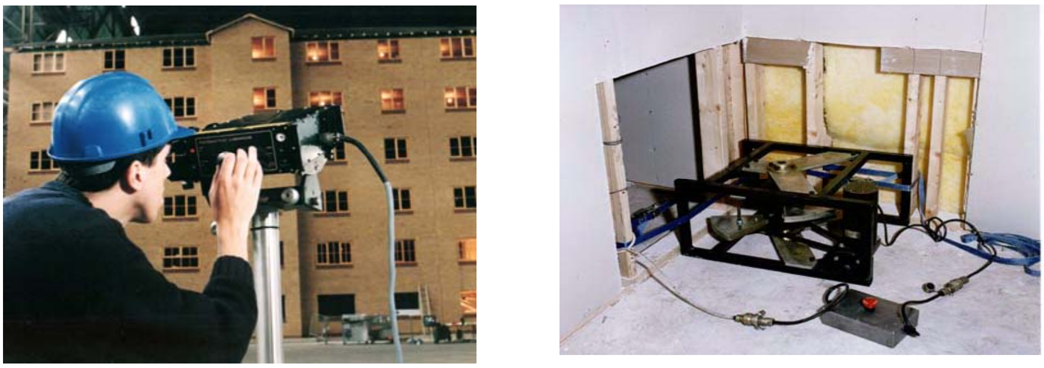

Figure 8: Stiffness measurements of the TF2000 building

The following stages show the overall tests carried out:

Stage I - Timber frame alone (blocks of flats, stairs, roof, roof tiles)
Stage II - Stiffness of timber frame plus all plasterboard wall and ceiling linings
Stage III - Stiffness of completed building including masonry cladding

The following analyses were carried out:

- Lateral stiffness of timber frame only
- Lateral stiffness of timber frame and plasterboard linings
- Lateral stiffness of whole building (with brick cladding)

The results show that the addition of plasterboard linings increased the stiffness by a factor of 3.3, indicating that these items play an important role in the overall stiffness of the structure at that stage. The addition of the brick cladding introduced a further overall stiffness of the structure by a factor of 5.3 (to a total factor of 17.5 over the original timber frame). These results are being analysed for codes and standards and amendments will be made to the codes in due course. More detailed results are given in reference 16.

3.2 Racking resistance of wall panel elements

It must be noted that the above tests on the TF2000 building, given in section 3.1, did not include strength tests. However, the strength test can be simulated by the element tests given below:

The most common type of timber frame walls constructed in the UK usually consists of a timber frame (studs, and top and bottom rails) sandwiched between panel products sheathing and plasterboard linings. BS5268: Part 6 gives all the relevant requirements for the design, testing and construction of timber frame walls.

The performance of timber frame wall panels in racking is not easily amenable to conventional methods of analysis based on material characteristics and principles of structural engineering. It is therefore necessary to assess the performance of wall panels before any attempt is made for analysis. It is essential that such a method is able to take account of the numerous variables relating to loads, materials, fixings and panel configuration. This part of the paper summarises typical types of failures in timber frame walls subjected to racking loads.

Types of failure (Figure 9):

- Lateral shear failure of nails along with their pull-out from the bottom rail at the bottom corner of the panels (on the loading side).
- Embedment of nail heads into the sheathing and pull-through
- Cracking of the bottom timber rail.
- Lateral shear failure of sheathing at nail positions
- Tension and compression failures of sheathing
- Buckling of the sheathing within the studs.

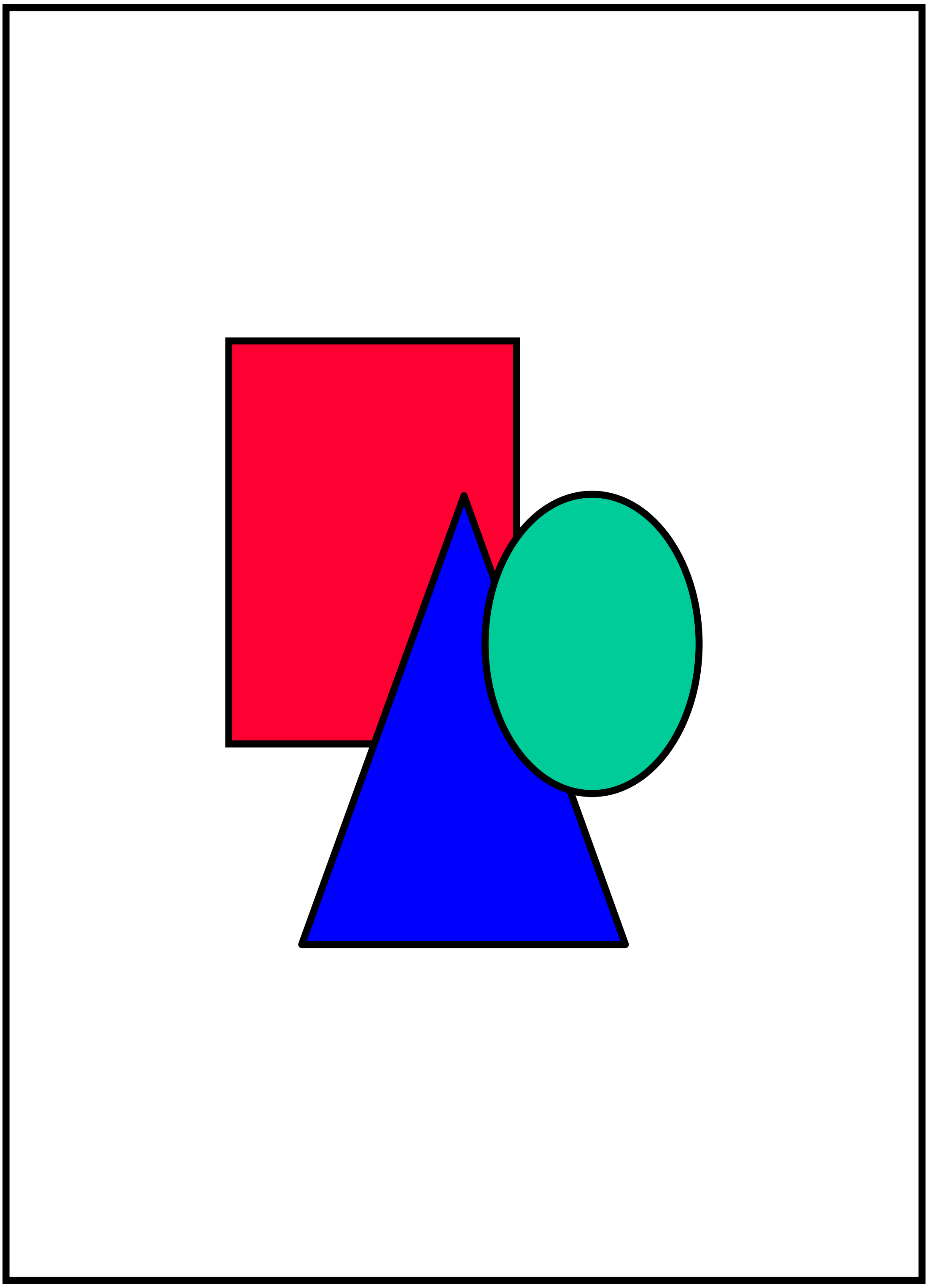

Figure 9: Typical types of failure in racking wall panels

4. PERFORMANCE OF TRUSSED GIRDER ROOFS

4.1 Trussed girder components under eccentric loading

This part of the paper describes the results of an experimental and theoretical investigation into the performance of multi-ply trussed girders under eccentric loading. Failure modes and safety indices are identified on a range of girders by means of full-scale laboratory testing designed to closely simulate practical conditions. Current design deficiencies are highlighted and new design proposals are made, based upon test results and theoretical studies of the stress conditions present in such situations (note that BS5268: Part 3 already has adopted the proposed design rules).

In trussed rafter roof construction it is common practice at roof intersections to use trussed girders to support in-coming trusses, via steel shoes connected to the bottom chord of the girder (Figure 10). This eccentric load can create significant torsional and bending forces on the girder bottom chord which had been ignored in UK design or detailing practice. Anti-split plates or rotational restraint brackets fixed to the girder bottom chord are details which are known to be in use in some countries around the world, but their application seems to be somewhat arbitrary.

The main objective of the research was to investigate the behaviour and safety levels present in multi-ply girders under eccentric loading. All the girders were designed according to then current British design practice, based on BS5268: Part 3 (BSI, 1985) and BS5268: Part 2 (BSI, 1991). The previous design practice did not take into account the eccentric loading effect on girders, and was based on all plies in a multi-ply girder carrying equal proportions of the load.

The plies of the 3-ply girders were nailed together while the plies for the 4-ply girders were bolted together. These fixings were therefore designed to transfer 2/3 and 3/4 of the total load, for 3- and 4-ply girders respectively. The total design load for the 3-ply girder was 73.3 kN while for the 4-ply girder it was 93.6 kN.

A test programme was devised to determine types of failure and their causes, and any shortcomings in the design, construction and manufacture of such girders. All the tests carried out to-date and their results are described in more detail in the appropriate BRE Client Reports (Enjily and Tarr 1993 & 1994). A brief description of the test method, tests and summary of major test results is given below:

The eccentric loads were applied to the bottom chord of the girders through standard 1.2 mm thick metal girder shoes Type TW961, 38 mm wide and fixed to the bottom chord by means of 3.0 x 75 mm square twisted sheradised, plain head nails. Loading was by means of 3.9 m long steel loading arms, at 600 mm centres, which contained softwood timber members of 35x100 mm placed underneath and parallel to the loading arms. The timber members of the loading arms were seated in and nailed to the standard girder shoes fixed to the bottom chord of the girders. These timber members were used to simulate the incoming trusses in practice, which provide some lateral stability to the bottom chord of girders. The ends of these timber members were placed at 0-6 mm from the back of the shoes in order to simulate typical positioning tolerances on site.

The application of the load was by means of 12 hydraulic jacks, at 600 mm centres. The jacks were pinned at the steel loading arms and their supports. A timber brace was attached perpendicular to the steel loading arms (parallel and near to the girders tested) for load sharing purposes. Intermediate support to the timber members of the loading arms was also provided 2.3 m from the girders; this was to simulate the stiffness of the incoming trussed rafter roof and to prevent excessive bending of the timber members of the loading arms.

The loads applied to the girders were measured by means of two pairs of load cells, each pair being positioned at the supports. Potentiometers, connected to a data acquisition system, were used to measure the vertical deflections, rotations, and out-of-plane movement of the top and bottom chords, and the opening up of the plies due to torsion, as well as deflections and rotations of the shoes.

4.1.1 Tests

Overall, 16 girders were tested, three of which were 4-ply girders and the remainder being 3-ply girders. Four of the 3-ply girders were used in mock-up tests, each time improving the method of laboratory simulation to that in practice. All the instruments and the data acquisition systems were also checked for accuracy during these mock-up tests. The following tests are annotated A and B for 3-ply and 4-ply girders respectively: The average moisture content of girders during testing was 12%**.**

Tests A1 and B1 : These tests were treated as datum tests and represent typical examples of design solutions currently used in the UK. The girders in these tests were loaded eccentrically, with no rotational restraints at the bottom chord of girders. Standard girder shoes were used at the bottom chords. The girders had 35% and 45% plate bite sizes at the bottom chord nodal points, for the 3-ply (Test A1) and 4-ply (Tests B1) girders respectively.

Tests A2 and B2: The test method was identical to that used in Tests A1 and B1 respectively. On this occasion the girders' bottom chords were restrained by noggings at the back of the girders. The girders were identical to those in Tests A1 and B1 respectively. Standard girder shoes were used at the bottom chord of the girders.

Tests A3 and B3: These tests, were concentric tests. The concentric load was applied to the bottom chord. The girders were identical to those used in Tests A1 and B1 respectively.

Tests A4 and A5: The test method was identical to that used in Test A1. The girders had a 75% plate bite size at their bottom chord nodal points. Standard girder shoes were used at the bottom chord of the girders.

The only difference between the girders in Tests A4 and A5 was that the girder in Test A5 had a quarter point plate width of 76 mm rather than 63 mm as in Test A4. None of the girders had rotational restraints at their bottom chords.

Test A6: The method of test was identical to that used in Test A1. The girder in this test had restraints of plywood decking (noggings and plywood nailed together and then nailed to the bottom chord of the girder. The girder had 75% plate bite sizes at the bottom chord nodal points. Standard girder shoes were used at the bottom chord.

Test A7: The method of test was identical to that used in Test A1. The girder had a newly designed type of shoe restraining the bottom chord. The girder had a bottom chord plate bite size of 35%.

Test A8: The method of test was identical to that of Test A1. The girder had a bottom chord plate bite size of 75%. The girder had standard type shoes plus a new type of strap restraining the bottom chord.

Test A9: The method of test was identical to that of Test A1. The girder had a newly designed type of shoe restraining the bottom chord. The girder had a bottom chord plate bite size of 75%.

4.1.2 Summary of the results

- The main types of failure were:
  - Tests A1 to A5 & B1 to B3:
  - Tension perpendicular to the grain of the timber in the vicinity of the bottom boundary of the bottom chord nodal plates (Figure 10).
  - Test A6:
  - Rafter buckling failure.
  - Tests A7 to A9:
  - More of a shear failure than tension perpendicular to the grain failure in the timber at the bottom chord nodal points.

- The opening up of plies, both in the top and bottom chords, proved to be insignificant due to the connections fixing the plies together and the connections of the shoes to the bottom chord.
- The out-of-plane movement of the top chords proved to be insignificant for the tests with factors of safety of less than 2.0. However, for tests with factors of safety of more than 2, buckling of the top chords was taking place without actual failure of the top chords occurring (except in Test A6 where failure did occur).
- The failures were more due to torsion than bending of the bottom chord of the girders. Therefore, torsion and tension (perpendicular to grain) grade stresses of timber should be investigated so that appropriate permissible stresses of timber can be used in the design.
- Investigation should be carried out as to what proportion of the eccentric loading is resisted by each ply of the girders.
- The size and position of the punched metal plates at the nodal points play an important role in the load-carrying capacity of girders. The load-carrying capacity of girders, under eccentric loading, may be partially increased by extending punched metal plates towards the bottom edge of the bottom chord. Therefore, consideration should be given to extending the punched metal plate connectors towards the bottom edge of the bottom chords to enhance the load-carrying capacity of girders by delaying as long as possible the type of failure obtained in the tests until an adequate design method has been developed.
- Consideration should be given to preventing (as much as possible) the rotation of the bottom chord of girders in practice (by the provision of adequate relevant bracing in the roof structure) until an adequate design method is developed.
- Shoes proved to be adequate regarding their overall strength and serviceability criteria <u>for the actual failure loads obtained</u>. The shoes, and their connections to both the girders and the incoming trusses are one of the important factors to prevent rotation of the bottom chord of girders. Therefore, adequate nailing of the shoes both to the girders and the incoming trusses should be carried out in practice. In addition, <u>certified</u> standard girder shoes (wrapped-round) must always be used to preclude the opening up of plies and to provide for the adequate transfer of the eccentric loads to the bottom chord, as proved by the tests.
- The type of girder used in practice is obviously important. The bottom chord of Fink type girders are not anticipated to perform as well in torsion as Howe type girders because of their layout; More nodal points at the bottom chord would result in a better torsional stiffness (better load-carrying capacity of girder) of the bottom chord. Therefore, various types and spans of girders need to be tested, incorporating all the areas highlighted in this report as well as any necessary criteria for the development of the theoretical design method and its verification.
- The stiffness of the incoming trussed rafter roofs and their connections to girders are two of the important factors to prevent rotation in the bottom chord of the girders in practice. From the results it can be deduced that the existing girders in service may have a factor of safety of more than 1.0 due to the stiffness of the incoming roofs <u>(provided that the standard of workmanship for the construction of the roof is at least as good as that used in the laboratory)</u>. However, it must be said that the precise value of this factor of safety cannot be determined unless a full-scale test is carried out. It is not believed that a factor of safety 2.25 (required by the Code[2]) or more can be achieved because of the type of failure which depends on the tension capacity perpendicular to grain of timber. Therefore, the stiffness of the incoming roofs (especially the bending stiffness of their ties) and the bracing of the girders are important factors to be considered by designers so that to prevent, if only partially, the rotation of the bottom chord of girders.
- Adequate longitudinal bracing (parallel and close to the girders near the point of application of the eccentric load) should be considered in the incoming roofs for additional load sharing purposes.
- The gap between the incoming trusses and the back of the shoes should be kept as small as possible (not more than 6 mm) to prevent excessive eccentricity.
- Inevitably, some stepping of plies in multiply girders takes place during and after manufacture, causing the plies not to bear fully on the supports. Any bearing failure at the supports would not be critical provided that the distance between the lower edge of plate (located over a point of support) and the lower edge of the member in contact with the support, is at least that specified in the Code[2]. Therefore, girders should ideally bear fully on their supports by considering constructional provision to overcome the stepping effect.

Figure 10: Typical tension perpendicular failure at the nodal points of girders

4.1.3 Conclusions and recommendations

A general lack of adequate guidance on the design, and construction of girders loaded eccentrically was highlighted worldwide during the course of this project.

Tension perpendicular to the grain failure of timber in the vicinity of the bottom edge of bottom chord nodal plates (Figure 10) must be considered in the design process. This type of failure results mainly from the bending and torsion forces at the nodal plates; local stresses in the outer plies of girders being increased significantly by torsional eccentricities.

The importance of the prevention or delay of tension perpendicular to the grain failure of the timber, and prevention of rotation of the bottom chord at its nodal points has been identified.

The safety indices of 3-ply girder designs have been improved from approximately 1.0 to 2.5 during the course of this research by (a)- increasing the bottom chord plate bite size from 35% to 75%. (b)- by using a newly designed type of girder shoe which reduces the bottom chord rotation by approximately 66%.

Work is continuing in those areas in need of further investigation to improve the design and detailing practice of girders still further. In the meantime, design practice in the UK has changed to reflect the results obtained so far, based on the following tentative recommendations:

- Guidance on the design and construction of eccentrically loaded girders should be included in codes.
- Forces in the leading plies of eccentrically loaded girders are greatly increased by torsional rotations of the bottom chord. These enhanced forces must be catered for in local timber design checks at node points. An indication of the size of these stress concentrations is given in Reference 32.

  These already enhanced forces in the individual members must be increased still further by 25% when checking the anchorage capacity or the tension capacity of the truss plates on the leading plies of girders.

- A tension perpendicular to grain check should be carried out at all joints using rules similar to those outlined in this paper. Where the component is loaded eccentrically these tension stresses must be magnified considerably as indicated in Reference 32.
- Positive restraint to eccentrically loaded components can greatly improve their serviceability and strength. New proprietary metal hanger designs need to be developed which offer engineers more options.
- Since the effectiveness of torsional restraints is primarily dictated by their rotational stiffness, performance criteria need to be established for such restraints, based upon standard test methods.
- Until further research is carried out and/or design rules have evolved, trussed girder designers should (if in any doubt) consider the provision of minimum plate bites of 75% and 50% on eccentrically and concentrically loaded girders respectively. Consideration should also be given in the meantime to the provision of torsional restraint to girders via diaphragms or tension strapping in addition to conventional metal shoes.
- Care should be taken to ensure gaps between the incoming truss and the leading face of the girder are minimised during erection, and that any unavoidable gaps are filled by packing.

Practical implications

Most timber roofs in the UK, and we would suspect most other parts of the world, have a corner turn, an intersecting roof, a hipped end or similar, where incoming trusses meet and are supported on a girder truss spanning in the other direction. Usually, the incoming truss reactions at these points are transferred eccentrically to the girder bottom chord via proprietary metal shoes. From a limited study of girder design and detailing practice worldwide, it seems that almost universally these shoes provide little if any horizontal restraint to torsional rotation of the girder bottom chord.

The full-scale tests carried out under this research so far indicate that torsional strain can create localised load concentrations in the outer plies in the order of 2 and 3 times that assumed from a concentric analysis for 3- and 4-ply girders respectively. Stress concentrations in the outer truss plates themselves can be 25% higher still. This in turn can result in a reduction in the overall factor of safety in such girders down to as low as 1.0; the limiting failure condition often being tension failure in the outer truss plates themselves, or tension perpendicular to grain in the timber immediately beneath them.

Tension perpendicular to grain checks due to torsion are rarely, if ever, carried out at joints due to lack of adequate design rules or data. Those design rules that do exist (Ehlbeck, Gorlacher and Werner 1989, Noren 1981) relate to concentrically loaded joints and therefore do not apply directly. The notional restraints offered to girder bottom chord by some proprietary hanger designs are also of little comfort, since even full restraint (which demands considerable restraint forces) still leaves stress enhancements in the individual plies of the order of 1.5-2.0.

Eccentrically loaded girders must be designed for enhanced forces in the leading plies. Application of these enhanced forces must be coupled with a tension perpendicular to grain stress check at all appropriate node points. The empirical formulae given in Reference 32 may be used as a guide in the formulation of appropriate design rules. Designed torsional restraints can have marked improvements on the performance and safety of eccentrically loaded girders, and new forms of shoe specifically designed for this purpose can often prove cost-effective. Manufacturers and designers alike should consider developing new girder truss shoe designs to suit this purpose.

Whilst this research has concentrated on eccentrically loaded trussed girders jointed with truss plates, its implications are likely to apply equally to all eccentrically loaded components jointed by any form of timber fastener.

4.1.4 Areas in need of further investigation -

- More theoretical studies should be carried out on girders loaded eccentrically leading to refinements in the design methods proposed in Reference 32.
- Investigation should be continued as to what proportion of the eccentric loading is resisted by the individual plies of girders.
- The distribution of torsion, tension, shear and bending stresses at the bottom chord nodes of girders needs investigating. In addition, the aspect ratio of the bottom chord of girders should be investigated for torsional stiffness.
- Investigations should be continued on the influence of the plate bite size on girder performance.
- Torsion and tension (perpendicular to grain) grade stresses of timber should be investigated so that appropriate permissible stresses for the timber can be used in the design.
- Various configurations and spans of 3-ply girders still need to be tested, as well as single-ply, double-ply and 4-ply girders loaded eccentrically.
- A performance specification for rotational restraints needs to be determined based on an established test method.
- Girders carrying other incoming girders should be investigated where high local concentrated loads are applied.
- A full-scale roof test is needed to indicate safety factors for existing girders in service.
- Similar research to this is required on other types of eccentrically loaded components, jointed with other types of timber fastener.
- More reliability-based design research is needed to advance engineering models for prediction of actual behaviour.

More detailed information and results are given in BRE Client Reports (Enjily and Tarr, 1993 & 1994) and Reference 32.

4.2 Full-scale trussed rafter roof containing a 3-ply girder based on the original design

This part of the paper pertains to an investigation into the in-built factor of safety of existing roofs in service containing 3-ply trussed girders (based on the original design) under eccentric loading. In trussed rafter roof construction it is common practice at roof intersections to use trussed girders to support in-coming trusses, via steel shoes connected to the bottom chord of the girder. This eccentric load can create significant torsional and bending forces on the girder bottom chord which was ignored in UK design or detailing practice in the past before 1991.

During the investigation, design deficiencies were highlighted by means of a series of component girder tests under eccentric loads, simulating the laboratory test work as close as possible to that in practice, results of which are reported in section 4.1. A major design deficiency was that of the low factor of safety in the component tests. However, girders in service are anticipated to have a higher factor of safety than that obtained in the component tests due to the stiffening effect of other roof members. It was necessary to determine the extent of increase in the factor of safety of such girders in a roof structure for the sake of all the existing 3-ply girders in service prior to the onset of the investigation. Thus a full-scale test was programmed to ascertain an indication of the in-built factor of safety of existing roofs in service containing 3-ply girders. The results of this investigation was included in BS5268: Part 3 for trussed rafter roofs.

This part of the paper summarises and concentrates on the work carried out on a full-scale test of a 'L' shaped building with a Fink type trussed rafter roof and a 3-ply Howe type girder. A brief summary of the layout of the full-scale building (roof), the programme of work, the method of full-scale test, the method of related component tests, the types of failure and their causes are given. Finally, an indication of the in-built factor of safety of a 3-ply girder in a typical full-scale roof is determined in addition to the assessment of the factor of safety of roofs with suspended ceilings (ie without plasterboard ceiling).

4.2.1 The design and construction details

The full-scale roof test and all the component tests were carried out at BRE, in Hall A (Heavy Structures Laboratory, Building 14) and the Timber Engineering Laboratory (Building 20.2) respectively. The building design, fabrication, erection and all the materials were provided by TRA.

The building was a single storey typical 'L' shaped bungalow. All the timber frame walls were designed and prefabricated with studs of SC3 (C16) timber and OSB sheathing of 9.0 mm thick. The girder in the roof was designed and detailed based on the methods used prior to the onset of the investigation into girder trusses. The roof trusses were designed based on BS5268: Part 3: 1985. The full-scale roof was 'L' shaped with 30^{o} pitch Fink trusses and contained a Howe type 3-ply girder at position where the roof changed direction. Extra studs were used underneath the girder (at each support) as supports because of the large concentrated loads at these points. All the prefabricated roof trusses had M75 (TR26) timber members. The Roof covering consisted of felt, battens and interlocking concrete tiles. Bracing of the roof was typical to that in practice in accordance with BS5268: Part 3: 1985. The spans of the incoming trusses and the trusses parallel to the girder were 10.06 m and 7.786 m respectively. The roof had normal plasterboard ceiling (12.5 x 1200 x 1200 mm sheets) with the required noggings of 38 x 45 mm. The plasterboard ceiling joints were not taped or finished with a skim coat of plaster or with 'Artex' as this would have been non-beneficial due to its destruction during the loading of the ceiling ties. The plasterboard orientation was that of the normal practice which changed direction (panel orientation) at the vicinity of the girder. Large openings were provided in the gable walls for access to the attic and inside of the building.

4.2.2 The outcome

- The ultimate load of the roof (girder) was 127.3 kN giving a factor of safety of 1.63 for the design load of 78.1 kN.
- The ultimate load of the roof with suspended ceiling (ceiling load of not more than 0.25 kN/m^2) is assessed to be 104.386 kN giving a factor of safety of 1.53 for the design load of 68.462 kN.
- Factors of safety of 1.63 and 1.53 are obtained for roofs with and without plasterboard ceilings respectively. It has been concluded that although these factors fail the acceptance criteria specified in the BS5268: Part2, the shortfall should be accepted for the following reasons:
  (a). Science evolves, and practitioners cannot be blamed for past omission that was caused by a problem neither originally understood or even apparent to the industry nor any research data available at the time. However, should subsequent development work highlight a potential problem (as it has done) which is then ignored, the blame can be fairly attributed to the practitioners made aware of that potential problem by the code or any other guidance.
  (b). Before the ultimate failure, the roof will definitely exhibit very pronounced signs of distress which with common sense should be reported, checked and not be used;
  (c). The ultimate load obtained is more than the required design load; this design load has a low probability of exceeding the 53% reserve. However, this does not imply that the practitioners should adopt this senorio for the future roofs for the reason mentioned in 1 above.

5. CONCLUSIONS

It is clear that when components and elements are considered in buildings, the three-dimensional effect plays a significant role in the performance and safety factor of timber structures. This three-dimensional effect increases the factor of safety of the elements and components in general, which currently there is no means of it being considered in the design of timber structures.

It must also be said that the majority of the above modes of failure do not always take place in buildings due to the three-dimensional effect. It would be beneficial if a reliability analysis could be carried out using a three dimensional modelling of timber structures. The structures that should be considered are multi-storey timber frame buildings and bridges. In addition, elements such as roofs, floors and walls, along with their components, are also in need of reliability analysis.

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