CIB-W18 Timber Structures

– A review of meetings 1–43

Part 5: Special actions

Hans Jørgen Larsen
Jørgen Munch-Andersen (ed.)

Danish Timber Information 2011

CONTENT

TRÆ Information

5.1 SEISMIC BEHAVIOUR

17-15-2 K F Hansen
Seismic design of small wood framed houses

It is to be expected that European manufacturers of one-family houses to a growing extent will seek new markets abroad and so in many cases will face the requirements that the houses must be able to withstand earthquakes. A particularly suitable type for earthquake areas is timber framed houses with a sheathing of for example plywood, particle boards, fibre boards, plasterboards or similar materials.

In this paper first a qualitative description is given of the forces induced in a house during an earthquake. The general rules for the design of earthquake-proof houses are listed and a method for taking up the forces in panelled timber framed houses based on vertical shear walls and horizontal diaphragms is described. The last chapters of the report deal with the general requirements to be met when designing the structural elements and joints of the house and what steps to take to ensure that the statically secondary elements of the house can withstand seismic vibrations.

Supplementary to the general advice and instructions of the report the Appendix contains guidelines on design of panels subjected to forces in their own plane.

Requirements for non-bearing structures and secondary structures
If a building is designed and built as a shear wall structure, the relative movements between the various parts of the building are small and the damage from these relative movements on secondary structures, non-bearing walls, windows, doors etc. will therefore, be relatively small. In spite of this, details by windows, etc. should be made in such a way that it is possible for a wall to deflect without the window getting a similar deflection in order to avoid broken window panes. A gap of at least 1/8" ~ 3 mm between wall and window is recommended. For non-bearing walls it would be prudent to build in a certain gap, too.

Even if the shear wall structure reduces the relative deflections among the bearing structural parts, it can of course not avoid that all parts within the house are exposed to the earth tremors.
The following rules serve to reduce the unfortunate consequences of these tremors.

– Cupboard, like boot, cases, closets etc. ought to be screwed (not, nailed to the walls. Closets should be provided with a positive locking device.
– Brick chimneys and fire places as well as heavy partitions should be avoided.
– Stoves, boilers and similar, heavy objects must be properly secured to floors and walls, and they should be placed as low as possible.
– Any gas installation must have an easily accessible closing valve.

18-15-1 A Ceccotti, A Vignoli
Full-scale structures in glued laminated timber, dynamic tests: theoretical and experimental studies

Abstract
This paper presents the results obtained by application of a particular methodology of dynamic tests being carried out on full-scale assembled timber structure, analysing in particular the theoretical approach and the importance of results obtainable from experiments.

Introduction

Factory control
In Italy quality control on glued-laminated timber is usually conducted by checking single structural elements while they are being manufactured in factories. This involves first of all the quality control of machinery and staff training, followed by the checking of gluing in the finger joints (bending tests) and between one lamina and another (shear tests) according to statistics.

Assembly Field control
Checks in the finished structure are left to the discretion of the "Direttore dei lavori" (Chief Resident Engineer) firstly, and the "collaudatore" (Testing Engineer) , secondly. They have the right to carry out tests that they consider suitable both before assembly as well as after assembly in order to check the quality of the materials themselves and to check the structural diagram provided by the designer.

Without entering upon the various problematics that appear at this stage, from now on we would like to concentrate on the particular aspects of loading tests in structures after assembly, proposing an agile dynamic variant to them.

General

Dynamic tests on structures (free vibrations)
In the field of structures already assembled, be they in steel, reinforced concrete or maonry, dynamic test methods have already started making themselves known.

Just to give a broad outline, these methods consist of exciting the structure's free vibrations, detecting the relative frequencies by means of accelerometers or other instruments capable of detecting vibrations more generically, and comparing results to those predicted according to a mathematic model previously prepared.

Application field
These tests are not only applicable to structures which have to support dynamic actions during their life span, but also to every type of structure as they enable one 0n one hand to obtain experimental indications useful in finding a more precise theoretical model of the structure, and on the other in finding principal "elastic" parameters of the material. All this is done through general information, that is not limited to a few sections and which is obtained through tests that are not expensive owing to loading means and times even if, on the contrary, they require sophisticated experimental survey equipment.

The case of wooden structures
The present paper's writers think it is not unsuitable to start experimenting this testing methodology on wooden structures, especially if one considers that this method requires excitations lasting' only for few seconds; it enables one to derive indications that are not affected by time as in the case of static loading.

Moreover, the static loading due to their own nature, in the case of assembled structures of a certain dimension and structural complexity, can't be easily codified and lead to results that are anyway affected by creep phenomena in the wood, which are not negligible right from the tests beginning unlike other building materials such as steel and concrete, for example.

Other dynamic tests
Other dynamic methods like the resonance or ultrasonic tests which have already come into use and which can be used for the stress-grading of sawn pieces of wood are unsuitable for large structures already being assembled whether due to the very-nature of the test (in the case of the resonance test, it can be carried out easily in a laboratory on specimens of reduced dimensions but it can become very problematic on assembled structures or due to the power of the necessary equipment which in the case of ultrasounds should be much higher than that used at present for stress-grading.

The sonic method is easier to use but it enables one to obtain useful indications of single structural elements but not of the whole structure taken as a whole.

Subject of the present paper
The present work illustrates an example of application of the proposed methodology just at it was used during testing carried out on a structure in glued-laminated timber with a span of 40 m.

Conclusions

Also for timber structures, the dynamic testing methods with the excitation of free oscillations, already applied successfully to structures in otter materials, seem worthy of being refined through further theoretical-experimental studies, due to their positive contribution when checking structures already assembled in terms of results validity and in terms of reduction of test.

19-15-1 A Ceccotti, A Vignoli
Connections deformability in timber structures: a theoretical evaluation of its influence on seismic effects

Object and scope

Timber construction plays an important role in many areas throughout the world, and finds a wide application in low-cost residential dwellings and public buildings.
Many of the areas where wooden structures proved to be very feasible are highly seismic regions; indeed, as earthquake experiences have taught, they provide a very good response to seismic waves.

Recently, the European Community Commission has issued a draft of Eurocode 8 "Common Unified Rules for Structures in Seismic Zones". By these rules, the assessment of seismic loads is based on the frequency and intensity of the seismic events, the quantity and distribution of masses, structural geometry; behaviour is assumed to be elastic. The nature of material and the type of structure are taken into account by introducing a modifying factor (in order to reduce seismic action, the so-called behavioural factor "q"), determined by the structure's capacity of dissipating energy through its hysteretical ductility and resisting severe earthquakes by reaching domains of non-elastic behaviour.

In the draft of Eurocode 8, values ranging from 1 to 6 were suggested for the different materials (e.g. 6 for steel), so reducing the seismic design action considerably, down to 16% of the expected theoretical peak for a structure with indefinitely elastic behaviour. Since structural timber shows usually an elasto-linear behaviour before yielding (with a brittle failure), a factor $q = 1$ was proposed for timber structures, which apparently makes it very difficult to obtain cost-effective dimensioning.

That choice does not come from poor performance of timber structures under earthquakes (experience showed just the opposite); rather it was felt by the members of the draft panel that what is presently known about sections and joints behaviour, especially under cyclic loads, is not enough to allow detailed structural analysis as required by Eurocode 8, and by any updated design code.

A recent proposal for new Italian seismic design codes, regarding all building materials, does acknowledge the brittle behaviour of timber sections; still, it points at structures capacity of dissipating energy at connections, and the need for further wide and detailed investigations to this effect.
A behavioural factor $q = 2$ is suggested, to take into account the dissipating capacity at thousands of nailed connections between plywood panels and timber frame elements.

However referred to a very simple example, this study is aimed as a preliminary contribution to the quantitative assessment of the effect of connections deformability on peak stress conditions that a timber structure could undergo during an earthquake.

Conclusions and further investigations
The authors are well aware of the following limits in their research: the geometry and shape of the structure; just two input accelerograms; the cyclic behaviour model not fully reflecting actual joint behaviour - cyclic behaviour laws were in fact derived from a model implemented for steel structures. Moreover, hysteretic damping could not be considered directly; rather, the type of analysis involved assuming damping at generic time as depending on the stiffness matrix: such assumption should be confirmed or rejected through tests on full-scale models, along with damping value at time $t = 0$; in our study, this was prudentially assumed to be 5%, and constant for all structures.

However, they think that such method, once extended and refined by means of both improved analysis programs and a greater wealth of experimental data, could help effectively in studying seismic effects on timber structures and allow a more exact assessment of the values required by

20-15-1 A Ceccotti, A Vignoli
Behaviour factor of timber structures in seismic zones

Summary
The ideas pertaining to the definition of the structural behaviour factor for the structures' design in seismic zones are re-examined. A method of evaluating the behaviour factor of plane timber structures composed of rigid (glued) joints or seimirigid joints (i.e. dowels) is presented. Behavioural laws relating to the timber's structural elements and their joints under cyclic loads were obtained experimentally.

Accelerograms, applied to the base of the structures, were pulled out on the basis of their response spectrum as defined by the Italian draft proposal CNR-GNDT. The acceleration values corresponding to the elastic field limit and the point of collapse (i.e. A_y, A_u) were obtained using a direct integration technique with a DRAIN-2D non-linear analysis computer program. The behaviour factor was defined as the ratio between A_u and A_y. The values of this coefficient in the case of semi-rigid joints can even be ten times greater than the values obtained for rigid joints. At the conclusion of the research a critical analysis of the results was carried out, indicating suggestions for further research.

Conclusions and suggestions for further research
The conducted analysis clearly allows the assessment of the variation tendency of some fundamental quantities depending on the most important constructive factors. More specifically, it can be stated that, in the case of

joints with nails or dowels of simple frames, the structural behaviour factor can be ten times greater than the case of glued joints. These results are just an approximate indication of the structural coefficient values for timber structures, because the conducted analysis was limited to a few examples and the joints' behavioural laws were simplified. The methods presented will provide more precise indications for the behaviour factor if more realistic models of the joints' cyclic behaviour are used. Following this procedure, significant indications for the structural behaviour factor can be obtained also for more complex static schemes.

21-15-5 A Ceccotti, A Vignoli
Behaviour factor of timber structures in seismic zones (part two)

This report deals with some recent developments about theoretic studies on behaviour coefficient *q* as mentioned in **Paper 20-15-1** "Behaviour factor of timber structures in seismic zones.

As suggested in the mentioned paper, a more realistic behavioural model for semi-rigid joints under cyclic loading has been assumed by the authors.

A new cycle with slip-joint is reported.

The case of base restrained portal frame with four semi-rigid connections (which showed lower *q* values in comparison with two-hinged portal), has been re-examined.

22-7-5 R Gunnewijk and A M J Leijten
Plasticity requirements for portal frame corners

Introduction

Application of plastic theory in structural analyses for timber structures is not common practice. The main reason is that timber is regarded as an elastic material although it may have some plasticity in bending. The response of a structure does not only depend on the material, joints play a part too. Although strength and stiffness properties are covered by code design rules, the ductility has not attracted much attention. This is one of the reasons why at present timber structures have such a bad position in Eurocode8, structures in seismic regions. Although the present used fastener types can be ductile, strength and ductility not always go together. Joints with many dowel type fasteners are stiff and strong but fail brittle. However the development of locally reinforced joints may change this situation. This paper tries to formulate the ductility or moment rotation requirements of moment joints to insure a maximum of moment redistribution. This approach is not new, with some minor modifications this method is applicable for timber structures. An example is given about the capabilities of steel reinforced joints. The approach is straight forward and easy to handle. Because the design philosophy below is based on simple plastic theory in which geometrical non linear effects are not included, the design rules only hold for braced frames.

Suggestion

Future tests of joints should not to limit the slip deformation of dowel type fasteners to 15mm as prescribed by IS08375, but relate the maximum slip deformation to the diameter of the fastener, for instance to twice the diameter. Always record the maximum slip deformation although it may exceed the 15mm limit. As for tubes, which can be regarded as dowel type fasteners, diameters up to 100 mm may be needed to meet the rotation capacity requirements.

22-7-7 M Yasumura
Mechanical properties of joints in glued-laminated beams under reversed cyclic loading

Introduction

In the large scale timber construction, arched frames are frequently connected so that the moment and shear forces are transmitted. Although the vertical loads such as the dead load and the snow load are generally more critical in this kind of structures than the lateral loads such as the wind and seismic loads, it is supposed that the mechanical properties of the joints of horizontal members affect a lot on the structural performance especially in case of the earthquake. Thus ten glued-laminated beams with various mechanical joints were subjected to the reversed cyclic loads in order to investigate the strength and the ductility of a joint. The purpose of this study is to provide the information for evaluating the seismic properties of a frame having the joints in horizontal members.

Conclusion

Summarizing the results of this study, the following conclusions are lead.

(1) Double shear bolted joints with the steel plates placed on the side of the beam and those with the steel plate inserted in the beam showed very brittle failure. Stress distribution in the direction perpendicular to the grain should be considered to design this type of joint.
(2) Single shear bolted joints with the steel plates placed on the edge of the beam showed relatively high ductility. However the joints having large numbers of bolts or lag screws showed less ductility.
(3) The joint in which pre-stress was applied with the steel rods did not fail at the deflection of 30 centimeters, and showed high ductility.
(4) The hysteresis loops of the joints were different according to the types of the joint. Initial slips caused by the clearance of the bolt holes were remarkable.
(5) The equivalent viscous damping of the mechanical joints in which neither adhesives nor tensile bolts were used was approximately 5 to 12 % when the deflection was small. This relatively high viscous damping may be caused by the initial slips of the joints.
(6) The equivalent viscous damping of the joint was approximately 2 % at the deflection of 1/100 of a span, and it increased to approximately 5 % in plastic area.
(7) Initial rotational angle of the joint in which neither adhesives nor tensile bolts were used varied from 1/350 to 1/130 and the rotational stiffness of the joint was about a half of those calculated from the experiment of one bolt.
(8) Ductility factors of the joint with the steel plates placed on the edge of the beam were 4.3 and 4.8 respectively, and that of the joint in which the steel plate was inserted and connected with twenty drift pins per joint was 3.2. Other types of joints showed brittle performance except for the specimen in which the pre-stress was applied on the steel rods.

22-15-5 M Yasumura
Seismic behaviour of arched frames in timber construction

Introduction

Arched frames of glued-laminated timber have been used widely in large scale construction. As the vertical loads such as the lead load and the snow load are generally more critical than the lateral loads in the large span structure, few studies have been done on the seismic behavior of the arched frames. In the previous paper, **Paper 22-7-7**, the mechanical properties of the joints in glued laminated beams subjected to the reversed cyclic loads were investigated. In this study, the arched frames of the glued laminated timber having the joints in the principal rafters were subjected to the reversed cyclic lateral loads, and the mechanical properties of the frames were investigated.

Time-history earthquake response of the three-hinged frames vas obtained from the linear model and the bi-linear elastic model, and the influence of joints of the principal rafters on the seismic behavior of the frame was also studied in this paper.

Conclusions

Summarizing the results of this study, the following conclusions are lead.

(1) Three specimens among four failed with the radial stress at the curved part of the glued-laminated wood, and one failed with the yield of a steel plate connecting the principal rafters. Both kinds of failures were not expected when the specimens were designed, and the difference of the safety factors among the steel, wood and mechanical joints should be considered in case of the design procedure.
(2) In three-hinged frames having the joints in the principal rafters, particular effect of the initial slips of the joint was observed. This effect might be caused by the difference of the moment distribution in the right and left joints, and this effect was small in two-hinged frame.
(3) The equivalent viscous damping of the three-hinged frames having the joints in the principal rafters was 2 to 3% when the deformation was small, and it coincided with the equivalent viscous damping obtained from the experimental results of the joints.
(4) The equivalent viscous damping of the three-hinged frame having no joints in the principal rafters was approximately 1%, and showed low damping capacity.
(5) The equivalent viscous damping of the frame after the failure of the glued-laminated timber by the radial stress at the curved part increased to approximately 4%.
(6) The maximum load of each frame was 74.6 to 78.0 kN regardless of the types of joint. The maximum radial stress calculated from the maximum load was approximately 96.6 kN/cm^2.

(7) The horizontal displacement for the maximum load in three-hinged frames and a two-hinged frame having the joints in the principal rafters were 1.8 to 2.4 times as large as that of the frame having no joints.
(8) The maximum acceleration response of the three-hinged frame excited by the earthquake accellerogram of which maximum acceleration was adjusted to 450gal varied from 468 to 1021gal in the frame having no joints and 427 to 888gal in the frame having the joints in the principal rafters.
(9) The maximum displacement response was the largest when the natural period was 1.23 seconds (k=3.3 kN/cm) in Hachinohe NS accellerogram excitation, and the maximum drift was 1/37 in the frame having no joints and 1/33 in the frame having the joints in the principal rafters of the height of eaves.
(10) The maximum shear response of the frame having the joints was reduced to about a half of that of the frame having no joints. This fact concludes that the seismic load can be reduced in a three-hinged frame having certain types of joint in the principal rafters. However, further study should be done regarding the energy absorption in the hysteresis loops.

23-15-4 M Yasumura
Seismic behaviour of braced frames in timber construction

Introduction

Trussed frames with the diagonal braces are the most simple and efficient structural element to resist against the lateral forces such as the wind load and the earthquake load. This structure is widely used in glued-laminated timber construction as well as in conventional wooden construction. As the failure of the braced frames generally depends on the buckling of the braces or the failure of the end joints, it shows smaller ductility and deformability than the framed structures in case of the earthquake. In timber structure, wooden members show generally brittle failures, so it is necessary to secure the ductile property in mechanical joints. In this study, the braced frames of the glued-laminated timber having different types of end joints were subjected to the reversed cyclic lateral loads, and the mechanical properties of the frames were investigated.

Load-displacement hysteresis loops were modelled with the bilinear slip model with reference to the experimental results, and the time-history earthquake response analysis was carried out to investigate the influence of the yield design load level on the response spectrum.

Conclusions

Summarizing the results of this study, the following conclusions are lead.
(1) All the specimens failed with the destruction of the end joints of the braced members due to the shear failure or fracture of wood. In any case the joints failed finally in tension. Although steel pin joints were applied, the rotation of plates was observed in compression braces.
(2) Load-displacement curves showed the non-linear hysteresis loops with slips due to the embedding of a bolt into wood. The unloading stiffness was approximately twice to three times as large as the initial stiffness. The degrading after the yielding of a bolt was observed in some specimens, but the yield point was not clear. This indicates that the braced frames are not basically ductile structure .
(3) The equivalent viscous damping of each specimen showed very similar values and was approximately 15% when the horizontal displacement was less than 1/200 of the specimen height, and it decreased to approximately 10% when the horizontal displacement was approximately 1/100 of the specimen height.
(4) The maximum load of the specimen having the thickness-to- bolt-diameter-ratio of eight and twelve was 1.25 to 1.82 times(1.44 times in average) as large as the calculated yield load, while that in the specimen having the thickness-to-bolt -diameter-ratio of four was equal to or smaller than the calculated yield load.
(5) The ductility factor of the specimen having the steel pin joints and the thickness-to-bolt-diameter-ratio of eight and twelve was 1.64 to 2.25 (1.93 in average), while that of the specimen of the thickness-to-bolt-diameter-ratio of four was 1.0. The ductility factor of the specimen without steel pin joints was 1.32 and showed smaller value than the specimen having the steel pin joint.
(6) When the braced frames are designed the thickness-to-bolt diameter-ratio of braced members should be equal or superior to eight and the stress perpendicular to the grain due to the rotation of a brace at the end joint should be avoided by means of a steel pin joint or other equivalent methods.

(7) The ductility factor of the braced frames which agree with the above items should be 2.0 in average or 1.5 considering the variation. The ductility factor of other types of braced frames should be 1.0.
(8) It is possible to obtain the "q" value information from if the soil profile is determined.

23-15-5 A Ceccotti, A Vignoli
On a better evaluation of the seismic behavior factor of low-dissipative timber structures

Summary
Some results concerning the seismic verification of w-dissipation timber structures designed according to Eurocode assuming a behaviour factor q = 2 are given in this paper.

The semi rigid connection design was performed considering suggestions given by the Authors in a previous paper.

The numerical simulation was carried out using a time domain non linear dynamic analysis .

Introduction
Three kinds of timber structures are considered in the draft version of Eurocode 8:
– non-dissipative
– low-dissipative
– medium-dissipative.

But due to the lack of knowledge the value of the behaviour fact reducing design inertia forces ("q") was fixed uniformly equal to 1 for each kind of structure.

Conclusions
From this research it seems possible to state that designing low dissipation semi-rigid connection portal frame timber structures following EC 5 and EC 8 rules but assuming q = 2 and using simple dimensioning rules for semi-rigid joints, it is possible to obtain structures whose resistance easily fulfils the Code strength requirements for seismic zones.

23-15-7 M R O'Halloran, E G Elias
Performance of timber frame structures during the Loma Prieta California earthquake

Cib structural timber design code
The CIB Timber Code provides for the development of design procedures for timber components and special structures. Section 7.4 is intended to provide specific provisions for the development of bracing requirements. Currently, the draft code does not contain references to either standard methods of test, sampling, or analysis of test data reporting wall bracing or diaphragm action.

Objective
The purpose of this paper is to present a review of the structural performance of timber frame buildings following the 1989 Loma Prieta earthquake. Results of this inspection reinforce the need for standardization in design of timber frame structures to resist seismic loading.

Summary and conclusions
A team of engineers from the American Plywood Association (APA)visited the San Francisco Bay area shortly following the Loma Prieta earthquake of October 17, 1989. Damage from the earthquake occurred in scattered locations within the region. Within these locations, the heaviest damage appeared to be to buildings with unreinforced masonry walls. All observed wood-framed buildings which were damaged were either built before the 1973 Uniform Building Code introduced updated earthquake regulations, or incorporated critical construction features but with inadequate connections, or other features which were not nailed or stapled in accordance with minimum code requirements.

It is the conclusion of the APA team of observers that framed construction built to meet current provisions of the Uniform Building Code performed very well. No code deficiencies for this type of construction were observed. However, greater attention must be paid to design and installation of connections and fastenings, and to structural continuity, especially in small buildings which may not be engineered.

24-15-3 M Yasumura
Seismic behaviour of wood-framed shear walls

Abstract
Wood-framed shear walls sheathed with plywood and gypsum boards were subjected to the reversed cyclic lateral loading, and it was found that the reversed cyclic loading affects more on the shear strength of gypsum-sheathed panels than that of plywood-sheathed panels. From the results of time-history earthquake response analysis of these panels, the behaviour factor "*q*" of 2.5 to 3.0 is proposed for the wood-framed shear walls.

Conclusion
Summarizing the results of this study, the following conclusions are lead.
(1) The reversed cyclic loading affected very little on the ultimate properties of plywood-sheathed shear walls, and there were no decrease of ultimate load by the cyclic loading of lower load level.
(2) The reversed cyclic loading made a serious influence on the shear strength of gypsum-sheathed shear walls, and the ultimate load was decreased 27% comparing to that in monotonic loading test.
(3) The elastic limit of plywood-sheathed shear walls may be defined by the shear deformation angle of 1/300 which corresponds to 50 to 60% of the ultimate load, and the yield load may be defined by multiplying by 1.5 the elastic limit which corresponds to 80% of the ultimate load.
(4) In the gypsum-sheathed panel, the shear load at the second and third cycles of 1/300 decreased to 87 and 83% respectively comparing to the first cycle, and it was shown that the elastic limit of gypsum-sheathed panel might be smaller than 1/300 of shear deformation angle. The yield load of gypsum-sheathed shear walls may be defined by the shear deformation angle of 1/300 considering the effect of cyclic loading.
(5) The equivalent viscous damping of plywood-sheathed panel was in a 15% at the first cycle and 12% at the second and third cycle. That of sheathed panel was in average 18.5% at the first cycle and 15% at the second and third cycles, and showed higher values than those of the plywood-sheathed panel.
(6) The ductility factor of 5.0 which corresponds to the shear deforms angle of 1/40 gives the A_u/A_y value of 3.0 for the plywood-sheathed shear walls.
(7) The ductility factor of 5.0 which corresponds to the shear deformation angle of 1/60 gives the A_u/A_y value of 2.5, and the ductility factor of which corresponds to the shear deformation angle of 1/40 gives the A_u/A_y value of 3.0 for the gypsum-sheathed shear walls.
(8) The ductility factor response of plywood-sheathed shear walls varied from 1.1 to 5.9 which corresponded to the shear deformation angle of 1/270 to 1/53 when the accelerograms were linearly scaled for 300cm/sec^2 and mg/A_y=3.0.

26-15-2 K Becker, A Ceccotti, H Charlier, E Katsaragakis, H J Larsen, H Zeitter
Eurocode 8 - Part 1.3 - Chapter 5 - specific rules for timber buildings in seismic regions

Introduction
This paper presents the pertinent state of the drafting and redrafting work on the timber chapter in Eurocode 8 'Buildings in seismic regions'.

As there are still possibilities to complement necessary provisions in the code text or in the 'National Application Document' (NAD) every comment and criticism is very welcome. The paper is aimed at opening of a discussion to provide substantial information.

On one hand it is very important to give guidance and rules for an adequate analysis, design and detailing for timber structures in seismic regions. The experience showed both during last earthquakes, an excellent behaviour as well as failures and more or less heavy damages on timber structures. The minority, the failures, originated from almost completely avoidable mistakes in modelling, design and mostly detailing. The development of a new code gave the possibility to elaborate the corresponding provisions with all the knowledge and experience of several seismic regions in the world.

One the other hand the use of timber for building purposes is to be strengthened face to face with other materials wherever possible. As the reasons for that engagement are well known, it is obvious to avoid huge masses of provisions, rules and prohibitions as for example the concrete part of Eurocode 8 takes almost the same size as the Eurocode 2. Designers, architects and engineers will stand away from timber, when they are exceptionally dealing with a seismic hazard and therefore confronted with too much confinement. For these reasons the code provisions should be short and precise. It is necessary and shall be possible to estimate realistically the stiffness and deformation properties with all timber-related scatter. To give the background information is the duty of conferences like this CIB-meeting and the Universities.

The formulation of several parts depends on the regulations in other parts of Eurocode 8 (e.g. the regularity-classes or the damping ratios), Eurocode 5 (e.g. design rules for nails) or other corresponding codes. As long as these codes are still in revision, redrafting or anyhow not in their final version some provisions may be changed.

The code requirements are mainly pointed on the attempt to provide sufficient ductile and hysteretic mechanisms in the structure. In opposite to other materials like concrete, steel and composite structures the ductility is mainly located in the connections, as the material itself shows in most cases a brittle behaviour.

The following pages will contain the code text. At text parts or pictures, which took more effort to formulate in the pertinent way, information and justification notes are given in boxes.

26-15-3 M R O'Halloran, E L Keith, J D Rose, T P Cunningham
Hurricane Andrew - Structural performance of buildings in south Florida

Abstract

This report describes the structural performance of residential and low-rise commercial structures in South Florida, based on observations from on-site inspections conducted within one week after Hurricane Andrew struck the Miami region on August 24, 1992. All types of construction – concrete, masonry, steel and wood – were inspected by a 45-member task group including a team of three technical and field promotion staff members from the American Plywood Association. The report concentrates primarily on the observed performance of wood systems, but other types of construction also are covered. Observations and conclusions are presented dealing with roofing attachment; wood structural panel roof sheathing -both plywood and oriented strand board (OSB) – and its attachment to framing and gable-end roof systems in particular; walls and connections of components; attachment of metal roof deck panels to steel framing; connections between concrete and metal roof decks and concrete or masonry walls; and attachment of cladding for pre-engineered metal buildings.

This report also provides a review of work completed which recommended fastening schedules for oriented strand board and plywood roof sheathing. The recommended schedules were established to provide resistance to wind uplift pressure, with particular emphasis on high-wind exposures. The approach included laboratory testing, engineering and computer analysis.

Conclusions

As can be seen from the observations discussed above, much of the structural damage done to the structures in South Florida was preventable through the proper design, specification and execution of connections between structural elements. It is hoped that, through publications such as this, knowledge of the importance of making proper connections can be conveyed to the building industry to eliminate the possibility of such preventable tragedies in the future. While it is difficult to imagine a natural disaster greater than Hurricane Andrew, it would be an even greater tragedy if we did not learn from the experience.

29-15-2 M Yasumura, N Kawai, N Yamaguchi, S Nakajima
Damage of wooden buildings caused by the 1995 Hyogo-Ken Nanbu earthquake

Introduction

A strong earthquake occurred in Hanshin area in the early morning of 17 January, 1996. It caused the largest scale of damages since after the Second World War. Over 6,300 persons were killed or lost, over 43,000 persons were injured and about 400,000 buildings were damaged. A large number of fatalities were caused by the collapse of wooden houses. Building Research Institute conducted a survey on the damaged wooden houses due to the 1995 Hyogo-ken Nanbu earthquake. This report presents the outline of the damages of wooden houses and discusses the aseismic design of timber structures in Japan.

30-15-3 E Karacabeyli, A Ceccotti
Seismic force modification factors for the design of multi-storey wood-frame platform construction

Abstract
Time-history non-linear dynamic analyses using twenty-eight earthquake accelerograms were performed on a four-storey wood-frame platform structure. The results confirm the current Canadian seismic force modification factor (*R*=3) for lateral load resisting systems comprising of plywood nailed shear walls. An alternate seismic modification factor (*R*=2) which accounts for the contribution of the gypsum wall board walls in design is also found to be appropriate.

31-7-1 M Yasumura
Mechanical properties of dowel type joints under reversed cyclic lateral loading

Abstract
Timber joints connected with a bolt or a dowel were subjected to the monotonic and the reversed cyclic loading based on the protocols provided in CEN standard and ISO/ TC165/WG7 draft, and the influence of the loading protocols and the reversed cyclic loading on the ultimate performance of joints was investigated. The specimen had the spruce glued-laminated timber and the steel plates of 12 mm thick connected with a bolt or a dowel of 16 mm diameter. The thickness of the timber was 2, 4, 8 and 12 times as large as the bolt diameter. The yield load obtained from the monotonic loading by CEN method showed comparatively good agreement with that by 5 % off-set method. Although there were few differences in the ultimate strength between the monotonic and the reversed cyclic loading, considerable decrease of the ultimate displacement was observed in the reversed cyclic loading.

Introduction
Stiffness, load carrying capacity and the energy dissipation are the most important parameters to evaluate the seismic performance of the timber structures. Among these parameters, load carrying capacity and the energy dissipation shall be estimated by conducting the reversed cyclic loading test of the joints. The test method for determining the seismic performance of joints made with mechanical fasteners has been provided in CEN Standard and also discussed in ISO/TC I 65/WG7 This standard and the draft provide different protocols for the reversed cyclic loading. The loading protocol of CEN Standard is based on the yield displacement corresponding to the yield load. That proposed in ISO/TC I 65/WG7 is based on the ultimate displacement corresponding to the failure load or 80 % of the maximum load after the peak. Therefore, the reversed cyclic loading tests based on these loading protocols were conducted on the dowel type joints to investigate the influence of the loading protocols on the load carrying capacities and the ultimate displacement.

Conclusions
Summarizing the results mentioned above, the following conclusions are led;

1) Yield loads obtained by "1/6 method" showed good agreement with those by "5 % offset method", and they agreed also comparatively well with the calculated values by the yield theory in the bolted joints with steel side plates and a steel center plate.
2) Yield loads obtained from the experiments were approximately 30% smaller than the calculated values in the dowel joints with a steel center plate because of the split of the slit at the center of main member .
3) No significant difference of the maximum loads was observed between CEN and ISO draft loading protocols both in the specimens with steel side plate and a steel center plate.
4) Maximum load of the bolted joints with steel side plates in the reversed cyclic loading reduced 10 to 40% to that of the monotonic loading; however no significant reduction of the maximum load was observed in the dowel joints with a steel center plate.
5) Few differences of the ultimate displacement were observed between CEN and ISO draft protocols.
6) A significant decrease of the ultimate displacement due to the reversed cyclic loading was observed in both the specimens with steel side plates and a steel center plate.
7) In the dowel joint with a steel center plate, the ultimate displacement of the specimen whose main member was twelve times as large as the bolt diameter was much smaller than those having the member thickness of eight times of the bolt diameter because of the split of the slit in the main member due to the bending deformation of a dowel.

8) Joints having the member thickness of eight times of the bolt diameter shows higher energy dissipation than those having the smaller member thickness.

31-15-1 N Kawai
Seismic performance testing on wood-framed shear wall

Abstract
To investigate the seismic performance of a shear wall used in wood frame construction and to confirm the adequacy of the response prediction based on the static loading test, a series of tests, which consists of static loading test, pseudo-dynamic test and shaking table test, was executed. The plywood sheathed shear wall of the specimen in each test was assembled with the same specification. The results of the pseudo-dynamic test and the shaking table test were compared with the result of response analysis using the load-displacement hysteresis model based on the static cyclic test result. The analytical result agreed well with the test results when the hysteresis models are used with considering the influence of the damage in one side to the opposite side.

Conclusions
The earthquake response of a wood framed shear wall was obtained by shaking table test and pseudo-dynamic test. The result of the time history analysis using the hysteresis model of bi-linear and slip based on the static cyclic loading test, agreed well with the results of shaking table test and pseudo-dynamic test when the influence of large displacement in one side to the behaviour in the opposite side is considered. However, the comparison of substitute damping h_s suggests that it is necessary to improve the hysteresis model based on the static test result. And the value of h_s obtained by the pseudo-dynamic test was approximately equal to the summation of equivalent viscous damping h_{eq} and modified viscous damping h'.

31-15-3 L Daudeville, L Davenne, N Richard, N Kawai, M Yasumura
Numerical simulation of pseudo-dynamic tests performed to shear walls

Introduction
Timber structures have satisfactory performances due to the high strength-to-weight ratio of wood and due to the unit actions of diaphragms and shear walls whose seismic behaviour is governed by fasteners. However, considerable damage has been reported to light-frame wood buildings in earthquakes like the 1995 Hyogo-Ken Nanbu one. The presence of large openings, the simplified design methods may explain it.

A cooperative research is developed in Japan for the analysis of structural performances of timber shear walls under seismic loading. The project includes a series of full scale Pseudo-Dynamic (PSD) and shaking table tests of shear walls made of posts, beams and sheathing panels. Rigid walls prevent the structure from vibrations in the directions perpendicular to the motion. The program will be finalised with a series of dynamic, pseudo-dynamic and static tests of two storey buildings with asymmetrical shear wall composition which are supposed to produce more complicated vibration modes such as torsion dimensional movements.

The modelling work is aimed at providing a numerical method for the analysis of performances of timber structures. This tool can help to reduce costly experimental programs, as a development tool to assist researchers in the formulation of seismic procedures, as a predictive tool to study various factors affecting the structural and to interpret experimental data, and as an analysis tool in the design of s earthquake-prone regions.

Many experimental programs have been conducted over full scale shear walls either under static, cyclic or dynamic loading. A common observation from these tests is that the hysteresis trace of a subsystem or subassembly is governed by the hysteretic characteristics of its connection. Thus, it is only necessary to characterise the hysteretic behaviour of joints to characterise the behaviour of wood structures and structural systems.

Some authors worked on the modelling of joints for the simulation of the response of shear walls under static loading or under cyclic or dynamic loading.

Some important phenomena, especially in the cyclic models, where not taken into account in the literature. For example, for several cycles at the same imposed displacement strength degradation between the first cycle (loading cycle) and the other cycles (cycles) were not modelled. Another experimentally observed phenomenon is difference between the monotonic curve and the peaks envelope of the loading cycles, a strength degradation occurs gradually as the number of cycles increases. In both cases, dissipated energy is affected and the behaviour of the wall under seismic loading may be very different.

The current models available in the literature are sometimes inappropriate for joints different configurations and material components. A model is proposed in this paper that takes into account the general hysteretic features of wood joints. This model is simple and easy to identify.

Comparisons between numerical and experimental responses of shear walls with plywood sheathing nailed on timber frames tested in pseudo-dynamic conditions are given.

Conclusion

A simplified FE analysis of timber structures under severe loading is presented. All the degration phenomena are assumed to occur in joints. According to literature, dowel type joints exhibit the same load-slip behaviour under monotonic or reversed cyclic loading. The main features of this typical behaviour were characterised from tests on nailed plywood-to-lumber connections and a model that relates the force in the connection to the relative displacement between wooden members is proposed.

This model was implemented in a FE code for the simulation of the PSD load-displacement response of three shear walls with plywood sheathing.

Despite of coarse assumptions concerning the elastic properties of materials or the non nailed joints, the experimental and numerical results fit quite nicely.

Some shaking table tests were carried out to walls in Japan. At the time this paper is written, the results are not available yet.

The simulation of the dynamic tests will allow validating completely the proposed model.

31-15-4 H G L Prion, M Popovski, E Karacabeyli
Force modification factors for braced timber frames

Introduction

Braced frames are very often the simplest and most economical structural systems used in timber construction to resist lateral loads. This type of system is often used in glued-laminated timber construction and in heavy post and beam construction. Bracing essentially provides triangulation by means of diagonal members inserted in the rectangular bays of the frame. In concentrically braced frames, there is essentially no eccentricity in the joints and the lateral forces are resisted by almost pure axial loads in the braces. Diagonal braces, however, have the disadvantage of obstructing movement of people and goods and are usually located in the plane of the wall.

Timber structures are generally recognized to perform well when subjected to earthquake ground motion. This is attributed largely to wood's high specific strength (strength-to-weight ratio) and its enhanced strength under short term loading. Traditional timber systems also have a high degree of redundancy as well as sufficient ductility and energy absorption capacity. Favourable system geometry, e.g. symmetrical plan and elevation, also contributed to its performance. Regardless of the demonstrated satisfactory performance, however, it was shown that timber construction in itself is not a guarantee for adequate structural performance during an earthquake. This was particularly evident during some of the recent strong earthquakes such as Northridge, California (1994) and especially Kobe, Japan (1995). During these earthquakes, many timber buildings collapsed causing extensive damage and human loss. The lessons learned from Kobe are of particular interest for the performance of braced timber systems. A majority of the buildings that collapsed were of the post and beam construction type, which is essentially a braced framing system. While many factors contributed to the poor performance, the lack of an adequate bracing system was the main reason for collapse.

The seismic response of a braced timber structure in general is a complex issue, involving many different interacting factors, which need to be understood and quantified. One of the most important considerations is to provide a system that can absorb large amounts of energy and thus lower the earthquake-induced forces, while maintaining adequate stiffness to avoid excessive deformations. To satisfy these requirements, the seismic design process must include a careful balance of strength, stiffness and ductility. In braced timber frames significant deflection of the frame is dependent on the joint deformations. The stiffer nature of the braced frames represents an advantage in the case of low to medium intensity earthquakes, because less movement causes less damage to non-structural elements and serviceability requirements are easily met. On the other hand, it often also means less potential for energy absorption, leading to higher forces and lower system redundancy.

Concluding remarks and recommendations

Braced timber frames can be used as efficient lateral load resistance systems in buildings. Because energy absorption capacity and overall ductility of these frames is typically influenced by the connections used, adequate connection design is of particular interest when these frames are used in high-risk earthquake zones. Results from current research on seismic performance and force modification factors for braced timber frames are presented in this paper. Static and dynamic tests were conducted on a several connection details with different diameter bolts and high strength glulam rivets, with steel side plates. The resulting hysteresis loops were used to develop non-linear mathematical models on a connection and frame level. The models were then used in numerous non-linear time history dynamic analyses, from which it was possible to determine the influence of different connection details on seismic response of braced timber frames.

From the presented results, it is evident that the seismic response of braced timber frames is heavily influenced by the behaviour of the connections. The results suggest that braced frames with different connections should not be assigned the same R-factor. Braced timber frames with mild steel (ASTM A-307) bolted connections, with slenderness ratios (*l/d*) of 10 or higher showed far more adequate seismic performance than those frames that utilized bolts with lower slenderness ratios. Further research is needed, however, to study the effects of other parameters such as end distance, spacing, number of rows number of bolts in a row etc., for the development of general recommendations that will ensure ductile behaviour of braced frames with bolted connections. Until such research is undertaken, an R-factor of 1.5 appears to be reasonable for braced timber frames with slender bolts.

On the other hand, glulam riveted connections showed promising results when used for braced timber frames. They were the only connections tested that consistently showed non-brittle deformations in the wood along with yielding of the connectors even at large displacement levels. In addition, riveted connections do not have as many parameters that influence their cyclic behaviour, other than those implemented in the CSA086.1-94 design guidelines. Braced frames with glulam riveted connections designed in rivet yielding mode may be assigned an R factor of 2.0, in recognition of their higher and more consistent ductility capacity.

32-15-2 N Kawai
Application of capacity spectrum method to timber houses

Summary

Capacity spectrum method (CSM) is known as a convenient prediction method for earthquake response of buildings, and is now discussed in Japan as one of the procedures to confirm the seismic performance of buildings in a technical standard under the revised Building Standard Law of Japan. In this paper, the results of CSM are compared with the results of time history analyses (THA), using one mass system models and two mass system models with the load-displacement hysteresis model based on the results of static cyclic loading tests on shear walls. The comparison of the results for one mass systems model shows that equivalent linear response tends to give a smaller prediction than the time history analysis when the equivalent viscous damping ratio, h_{eq}, of first loop of cyclic loading protocol proposed in ISO/TC165 is used. When h_{eq} is reduced to 80% of the original in CSM, the prediction gives better agreement for one mass system models in maximum response displacement. Also for two mass system models, the results of CSM with 80% of h_{eq} agreed well with those of THA in maximum response displacement. However, there was a case that CSM gives smaller prediction of relative story displacement, which is 64% of a result of THA.

Conclusions

The earthquake response predictions by capacity spectrum method were compared with the results of time history analyses using one mass system models and two mass system models, load-displacement relationships of which are determined by those of shear wall(s). The results are summarized as follows:

According to the results of the examination using one mass system models, capacity spectrum method gives good prediction compared with the results of time history analyses in maximum displacement, when the equivalent viscous damping ratio reduced to 80% of the original is used.

Also for two mass system models, capacity spectrum method gave good agreement in maximum story displacement with reduced values of equivalent viscous damping to 80%.

However, it seems to be risky to use the result of capacity spectrum method directly for the evaluation of maximum relative story displacement, as there was a case that gives smaller prediction, which is 64% of a result of time history analysis.

33-15-2 N Kawai
Prediction of earthquake response of timber houses considering shear deformation of horizontal frames

Summary
Related to the revision of Building Standard Law of Japan, the application of capacity spectrum method (CSM) is discussed as a convenient prediction method for the earthquake response of structures. For the application of CSM, there are some problems in the procedure to replace the structure to a single degree of freedom model, such as distribution of external forces, assumption of damping, and so on.

In this paper the predictions by CSM were compared with the results of time history analysis (THA) against ten artificial waves, using horizontally lumped mass system models, in which the shear deformation of horizontal frames are considered. The models of one-storied houses were used, which has 3 rows of shear walls in x-direction and 4 rows in y-direction, and the earthquake load is applied in y-direction. Vertical frames (rows of shear walls) and the parts of horizontal frames are modelled as shear panels, which have load-displacement hysteresis model based on the cyclic loading tests on a plywood sheathed shear wall.

As a whole, the predictions by CSM agreed well with the results of THA when the modified value of damping is used, which is calculated using 80 % of equivalent viscous damping corresponding to the first loop under the cyclic loading protocol of ISO/WD16670. And the CSM using damping calculated with 100 % of equivalent viscous damping corresponding to the stabilized loop seemed to give the upper limit of maximum displacement obtained by THA. However, CSM sometimes gives different distribution of response displacement from THA, in case the deformation by torsional behaviour and horizontal shear deformation is relatively large. For a solution of this problem, modification of CSM is proposed to use the distribution of external force with adding torsional moment corresponding to the existing eccentric moment. When the distribution of external forces with 20 % of eccentric moment in CSM, correlation of the predicted maximum response displacement between THA and CSM was improved.

Conclusions
To confirm the applicability of Capacity Spectrum Method (CSM) to timber houses 1 shear deformation in horizontal frames, predictions of maximum response displacement CSM were compared with the results of time history analysis (THA) using 21 type horizontally lumped mass system models. The predictions by CSM agreed well with results of THA when the modified value of damping is used, which is calculated using 80 % of equivalent viscous damping corresponding to the first loop under the cyclic loaf protocol of ISO/WD16670. And the predictions by CSM using damping ratio corresponding to stabilized loop were near to the upper limit of predictions by THA. However, CSM sometimes gives different distribution of response displacement from THA in case the deformation by torsional behaviour and horizontal shear deformation is relatively large. For this problem, a modification to use external force to which torsional moment is added is proposed. When the 20% of existing eccentric moment is added for the model with eccentricity with asymmetrically distributed masses, the prediction by the modified CSM gives better agreement with THA in the distribution of maximum rest displacement including torsional behaviour.

34-15-3 M Yasumura
Evaluation of damping capacity of timber structures for seismic design

Introduction
Linear equivalent response method is one of the seismic design methods to ensure the structural safety of buildings against severe earthquake motion. This design method was introduced to Japanese new building codes in June 2000. Different from the equivalent energy method, it does not need the behaviour factor (q) which depends on a lot of experiences and highly engineering judgements. The major parameter required for the linear equivalent response method is a force-displacement relationship and the equivalent damping of the structure. The equivalent viscous damping ratio obtained from the static reversed cyclic test is generally applied to determine the equivalent damping of the structure.

The structural behaviour of wood-framed construction against horizontal loads is highly dependent on those of shear walls. Therefore, monotonic and reversed cyclic loading tests were conducted on wood-framed shear walls to obtain the parameters for determining the hysteretic model of shear walls. Pseudo-dynamic test was also conducted on the same type of shear walls and the experimental results were compared with the calculation by non-linear earthquake response analysis and the equivalent linear response analysis.

Viscous damping is one of the issues difficult to determine. In general damping factor of 2 to 5% is assumed for dynamic analysis. In this study, high-speed loading test was conducted to determine the damping factor of wood-framed shear walls. It was found that the loading rate has an effect on the lateral resistance at the horizontal displacement of 10 to 70 mm, but it makes no influence on the load carrying capacity of shear walls.

Conclusions

Simulation by Non-linear time-history earthquake analysis agrees comparatively well with pseudo-dynamic test results. This kind of model is appropriate for predicting seismic behaviour of wood-framed shear walls. Equivalent linear response method using equivalent viscous damping ratio also gives generally good approximation to determine the maxima displacement response, however special consideration should be taken for some kind of seism. High speed test of shear walls showed that we can expect 2 to 5% damping factor up to horizontal displacement of approximately 70mm (story drift of 1/35), but we can not exp viscous damping at a large deformation in wood-framed shear walls.

35-15-5 E Fournely, P Racher
Cyclic and seismic performances of a timber-concrete system - local and full scale experimental results

Introduction

Timber-concrete composite floors are light solutions to comply with design criteria: stiffness, load capacity, fire barrier and resistance, phonic isolation, comfort toward vibrations... Most of these characteristics are particularly important in seismic zone. A lot of mechanical functions have to be assumed by a floor in a structure undergoing severe vertical and horizontal loading. Obviously, bending strength is the first function of a floor. Nevertheless, diaphragm function and connection with vertical structure is also extremely important. The total or semi-rigid connection of to layers provides a large strength for floor elements in bending. This connection also undergoes shear forces in order to transfer horizontal loading (wind, buckling effects, seismic actions...) from beams to diaphragm and from diaphragm or beams to vertical structures. Figure 1 gives an illustration of different configurations for a composite floor.

35-15-7 A Ceccotti, T Toratti, B Dujic
Design of timber structures in seismic zones according to EC8- 2002 version

Introduction

Eurocode 8 is the European code for "Design of structures for earthquake resistance". Part 1 includes "General rules, seismic actions and rules for buildings". Chapter 5 of Part 1 is devoted to specific rules for concrete buildings, chapter 6 is for steel buildings, chapter 7 for steel-concrete composite buildings, chapter 8 for timber buildings, and finally chapter 9 is for masonry buildings. The final draft (prEN 1998-1) is dated May 2002 and it is now ready to undergo the procedure to be accepted as an EN standard. This version of the code supersedes the previous 1994 version (ENV 1998-1). The intention of this paper is to highlight the major differences between the two versions and their consequences on timber buildings design. Finally a four storey building design example is provided to non-European Colleagues for possible inter-codes comparisons.

Conclusions

Eurocode 8 is a seismic design code that considers the same design philosophy for all building materials. In particular the designer of timber structures is allowed to perform a global elastic analysis. Eurocode 8 presents few relatively simple and conservative design rules, which are easy to apply, for the most important structural forms, classified according to their ductility and energy dissipation level. Last version of the code (prEN, 2002) leaves basically untouched the design dissipative structures while increases the demand for non-dissipative designed structures.

36-15-4 M Yasumura
Pseudo-dynamic tests on conventional timber structures with shear walls

Introduction

As the seismic performance of timber structures composed of shear walls is mainly governed by the mechanical properties of shear walls, the evaluation of seismic performance of shear walls is essential for the seismic design of timber structures. The lateral resistance of shear walls can be determined by the following four criteria obtained from the reversed cyclic lateral loading tests'); the initial stiffness, the yield strength, the ultimate strength and the ductility). Although these criteria are indispensable for the evaluation of shear walls, they do not always predict the dynamic response of structures during the earthquakes. Pseudo-dynamic test is one of the most efficient methods to estimate the seismic performance of timber structures. To conduct a pseudo-dynamic test of shear walls, we need to assume a mass which is the most sensitive to the earthquake response. In general, we consider the shear wall of the first story and apply the mass supported by that particular wall. However, the effects of shear walls of the upper story may not be negligible on the response of that of the first story.

To examine these problems, we conducted pseudo-dynamic tests on two-story timber structures with plywood-sheathed shear walls having an opening of different configurations, and the effects of the balance of the racking strength between the first and the second stories were studied. We also compared the responses of the first story with those of the test results of the individual shear walls.

The test results showed that the maximum displacement response of the first story became smaller when the resistance of the shear walls of the second story was smaller and that the maximum displacement responses of the first story were very close to those of pseudo-dynamic test results of the individual shear walls when the resistance of the shear walls of the second story were sufficiently large. These results indicate that the pseudo-dynamic tests of a single shear wall element with the mass supported by that particular wall show generally equal to or at least safe-side estimation of the response of the first story, and this method is appropriate to evaluate the seismic performance of the shear walls of the first story which are the most critical during the earthquake.

Time-history earthquake response analysis is another effective method to evaluate the seismic performance of timber structures. A good model of the cyclic behaviour of shear walls is required to predict precisely the response. In this study the parameters of the hysteretic model of shear walls were determined from the reversed cyclic tests of the individual shear walls. The simulation predicted quite well the response of the first story, but tended to under-estimate the response of the second story. Further studies may be necessary to predict more precisely the seismic response of whole structures.

Conclusions

The following conclusions were drawn from the experimental and analytical studies:

1) The pseudo-dynamic tests of a shear wall with the mass supported by that particular walls show generally equal to or safe-side estimation of the response of the first story, and this method is appropriate to evaluate the seismic performance of the shear walls of the first story which are the most critical during the earthquake.
2) The simulation by the lumped mass time-history earthquake response analysis predicted quite well the response of the first story, but tended to underestimate the response of the second story. Further studies may be necessary to predict more precisely the seismic response of the whole structure.
3) The tensile forces were concentrated on the corner post near the loading points and showed almost close values of 22 to 28 kN regardless of the opening configuration except for the specimen SHS. The tensile force of the third posts (C or B) varied from 4 to 19 kN according to the different opening configuration. As the effects of reducing the tensile force of the joints at the foot of inner posts are negligible, they should be considered in the design of joints connecting the posts to the sill.

36-15-5 B Kasal, P Haller, S Pospisil, I Jirovsky, A Heiduschke, M Drdacky
Experimental investigation of laminated timber frames with fiber-reinforced connections under earthquake loads

Abstract

1:4 scaled models and full-size two-story laminated timber frames were

subjected to a series of shake-table experiments. Two different designs were tested: (i) typical design and (ii) frames with densified and fibre reinforced connections. The full-size frames had the footprint of 2400 x 3300 mm and a height of 5200 mm. The tests included sinusoidal sweeps of low amplitudes, arbitrary loading simulating earthquakes of different magnitudes (0.25-1.0g), and a sinusoidal dwell at the first natural frequency. Individual beam-to-column connections were tested to establish a nonlinear moment-rotation relationship that was used to model the connection behaviour. 1:4 scaled models of frames and joints were tested prior to full-scale experiments. The fibre-reinforced connections enhanced the performance of the frames due to the higher load-carrying capacity and ductility.

Conclusions

Reinforcing beam-to-column connections of laminated heavy timber frames increases the moment and energy dissipation capacity of the joints and structural systems with these joints. Using densified material further enhances the connection performance provided that such connections are reinforced to prevent brittle failures in tension perpendicular to fibres.

Standard cyclic connection tests used in this work reach the practical deformation range too rapidly and do not give enough information about joint behaviour under small rotations. Shake table tests of small and full-size frames revealed qualitatively same behaviour of both structures but similitude theory cannot be used to make inferences from small-scale tests due to the nonlinear behaviour of the system.

36-15-6 F Lam, D Jossen, J Gu, N Yamaguchi, H G L Prion
Effect of test configurations and protocols on the performance of shear walls

Introduction

A series of full scale static and reversed-cyclic tests on 1.82 m x 2.67 in Japanese post and beam shear walls sheathed with OSB panels were conducted at the University of British Columbia to evaluate their performance under different configurations and protocols. Lateral performance of shear walls tested with hold-down devices or tie rods (hydraulically controlled) are compared. Different cyclic test protocols are also considered including protocol recognized by Japanese Ministry of Land, Infrastructure and Transport (MLIT), Consortium of University for Research in Earthquake Engineering (CUREE) developed near-fault protocol, and a University of BC (UBC) protocol. Different failure modes were observed under the various test configurations and protocols. There was significantly increase in the ultimate strength when hold-down devices and dead weight were used in comparison with the hydraulically controlled tie rod system. The envelope of the cyclic load deflection curves from the CUREE near-fault protocol and the UBC protocol matched the monotonic test results well.

Conclusions

To investigate the influence of different configurations and protocols on the performance of Japanese shear wall, Japanese protocols, UBC protocol and CUREE developed near-fault protocol were followed. The test results showed that the backbone curves of the UBC and CUREE near-fault protocols, agreed with the monotonic test results. The protocols with long load sequence gave a similar backbone curve prior to achieving peak loads with monotonic test results. After peak loads, significant drop in capacity can be observed compared to the monotonic test results resulting from premature failure of the nail connectors.

Comparing the results from two test phases indicates that hold-down devices significantly increased the strength of walls. The test results also show that the variation of wall strength was not significant when nail withdrawal type failures of the sheathing to frame connectors were expected.

36-15-7 J D Dolan, A J Toothman
Comparison of monotonic and cyclic performance of light-frame shear walls

Abstract

A total of 31 shear walls were tested under monotonic and cyclic loading to (1) determine the resistance to lateral loading of shear walls with various sheathing materials; (2) examine the effect of fully reversed cyclic loading on the specimens; and (3) compare the monotonic and cyclic performance of the walls. Tests were conducted on 1.2 x 2.4m (4 x 8ft) walls, constructed of oriented strand board (OSB), hardboard, fiberboard, or gypsum wallboard. Four-foot long walls were chosen because it represents the maximum ratio that can be used for gypsum wallboard. The 2:1 aspect ratio is also the maximum aspect ratio permitted by the United States build-

ings codes for wood structural panels used to resist seismic forces without further reduction in design values. Two walls of each sheathing material were tested under both monotonic and cyclic loading. All walls in this study incorporated the use of overturning anchors in the form of mechanical hold-downs. The monotonic tests were conducted according to ASTM E564, while the cyclic tests were conducted to the recently adopted ASTM standard E2126. A comparison of the monotonic and cyclic tests leads to several key dissimilarities. In general, the performance of the shear walls when tested cyclically decreased, although the degree of reduction depends on the sheathing material. Hardboard panels tested under cyclic loading performed similar to the monotonic tests. The peak load during cyclic tests was actually larger than its corresponding monotonic test. Fiberboard panels also performed in a similar manner when tested under cyclic and monotonic loading. OSB panels did not perform as well when subjected to cyclic loading. The peak load was reduced by 12%, and the energy dissipation was 27% lower when tested under cyclic loading. Gypsum panels performed poorly under cyclic loading. Peak load reduction was 16%, failure displacement decreased by 38mm (1.5 in.), and the amount of energy dissipation was nearly reduced by one-half when subjected to cyclic loading.

Conclusions

A total of 31 walls were tested during this study to determine and compare the effects of monotonic and cyclic loading on shear walls with various sheathing materials. The sheathing materials investigated are oriented strand board (OSB), hardboard, fiberboard, and gypsum wallboard. All of the walls were 1.2 x 2.4 m (4 x 8 ft), and to be conservative there were no gravity loads applied to the walls. All of the tests were performed with hold-downs. Monotonic tests were performed according to ASTM E564, while the cyclic tests were performed according to ASTM E2126. The conclusions drawn from the cyclic tests are:

(1) In general, the performance parameters decreased when the cyclic tests were compared to the monotonic tests. The most drastic reduction was observed during the gypsum-sheathed tests, followed by the OSB-sheathed tests.

(2) Hardboard panels tested under cyclic loading performed similar to the monotonic tests. The peak load during cyclic tests was actually larger than its corresponding monotonic test (8%). The displacement at failure decreased by 9%, and the energy dissipation was reduced by only 4% when subjected to cyclic loading.

(3) Fiberboard panels also performed in a similar manner when tested under cyclic and monotonic loading. The peak load only decreased by 0.27 kN (0.06 kips), which was a reduction of 4%. The displacement at failure decreased by 7%, and the energy dissipation was reduced by 5% when subjected to cyclic loading.

(4) OSB panels tested cyclically did not perform as well as when tested monotonically. The peak load reduction was 1.33 kN (0.3 kips), or 12%, and the energy dissipation was 27% lower when tested under cyclic loading.

(5) Gypsum panels performed poorly under cyclic loading. The peak load reduction was 16%, the failure displacement decreased by 38mm (1.5 in.), and the amount of energy dissipation was nearly reduced by one-half when subjected to cyclic loading.

37-15-1 M Yasumura, M Uesugi, L Davenne
Estimating 3D behavior of conventional timber structures with shear walls by pseudodynamic tests

Abstract

One of the most important causes of the seismic damage of timber buildings is the unbalance of the lateral resistance of shear walls and the lack of the shear stiffness of the horizontal diaphragm. These damages were often observed during the 1995 Hyogoken-Nanbu Earthquake on conventional commercial buildings that had large openings in front of the building. The objective of this research is to analyse the influence of the unbalance of the lateral resistance of shear walls and the shear stiffness of the horizontal diaphragm on 3D dynamic behaviour of the structures by pseudo-dynamic tests. The specimens were conventional post and beam timber structures with shear walls of 3m width, 3 m depth and 3m height. Specimens had 7.5 mm plywood sheathed shear walls and a horizontal diaphragm sheathed with 24mm thick plywood. There were two types of horizontal diaphragms, i.e., the rigid diaphragm in which all the perimeters of plywood sheathings were nailed and the semi-rigid one that were only nailed in transversal direction. Four specimens with different combination of shear walls and diaphragms were prepared. Specimens WW-R and WW-S had shear walls with window opening in both loading direction and

rigid and semi-rigid diaphragms, respectively. Specimens WS-R and WS-S had shear walls with opening and slit openings in loading direction and rigid and semi-rigid diaphragms, respectively. Both frames perpendicular to the loading direction were sheathed with plywood without openings. The lateral loads were applied to one side of the top of the specimen to give the eccentric loading. The lateral load based on the accelerograms of El Centro NS scaled up to 0.4 G. The mass was assumed to be 10t. The test results were compared with the simulation with the lumped mass 3D model. It was shown that the model predicted quite well the earthquake response of 3D structures and useful for the parameter studies to estimate 3D behaviour of timber structures.

Conclusions
Summarizing the results mentioned above, the following conclusions are led.

1) The initial shear stiffness and the strength of the rigid diaphragm with blocking was approximately three times higher than those of the flexible one without blocking, and the nail joints between the sheets material played an important roll to transfer the shear force while the tongue and groove joints were not efficient if some adhesives or mechanical joints were not applied.
2) There are few differences of the displacement response at the top of the wall in loading side between the specimen having rigid horizontal diaphragm and the flexible one. The displacement response at the top of the wall in unloading side with rigid diaphragm is approximately twice as large as those with flexible diaphragm.
3) The deformation of the specimen with rigid diaphragm is mostly rotation and the shear deformation of the diaphragm is very small. The deformation of the specimen with flexible diaphragm is mostly the shear deformation of the diaphragm and the rotation of the diaphragm is smaller than that with rigid diaphragm.
4) The simulation predicted quite well the earthquake response of the structures. The lumped mass 3D model proposed in this study is suitable for conducting a parameter study on 3D dynamic behaviour of timber structures.

37-15-5 B Yeh, T D Skaggs, T G Williamson, Z A Martin
Acceptance criteria for the use of structural insulated panels in high risk seismic areas

Abstract
One of the fastest growing segments of the US housing construction industry is the use of structural insulated panels or SIPs. These components are also used extensively in commercial construction in the US and other countries. Emphasizing the growing importance of SIPs worldwide is a recent work item initiated by ISO TC 165 in the development of an international standard for the design of SIPs. In the US, SIPs are typically constructed using a foam core with outside layers of oriented strand board (OSB). While it would seem logical that a structural component having double "skins" of OSB would perform well when subjected to high lateral forces, such as those experienced during an earthquake, the US building codes have limited the use of SIPs to low to moderate seismic zones. This limitation is due to the concern that the sealants used in the manufacturing and installation of SIPs may affect their seismic performance.

Since the US building codes do not explicitly cover SIPs, building officials have the authority to accept their use under code-compliance evaluation reports, which are typically published by the ICC Evaluation Service (ICC-ES). These code reports are based on analysis of test data in accordance with ICC-ES AC04, Acceptance Criteria for Sandwich Panels. This paper describes efforts by APA - The Engineered Wood Association and the Structural Insulated Panel Association (SIPA) to revise AC04 in gaining recognition for the use of SIPs in high seismic risk zones.

APA conducted a series of cyclic load tests using conventional wood framed shear walls and SIP walls. Results of this study confirmed that the SIP walls had equal or better performance than the conventionally framed walls. An analytical procedure for cyclic SIP shearwall tests was developed by APA and was approved by ICC-ES in the revised AC04, which now permits the use of SIPs in high seismic risk zones. To achieve this acceptance, a SIP manufacturer will be required to conduct the aforementioned tests in accordance with the new provisions of AC04.

Conclusions
The aforementioned comparison is based on matched wall tests. The basis for the evaluation is light framed wood walls sheathed with wood structural panels, which are a listed system in terms of seismic design coefficients.

Based on cyclic testing of the known system, a normalization technique is used such that data from conventional walls is compared to data collected on SIP assembles. There are four criteria that are examined:
1. Ultimate Load
2. Stiffness
3. Deflection at allowable story drift, and
4. Normalized cumulative energy dissipation

If the SIP systems demonstrate equivalence to the light framed systems based on the above criteria, it can be considered that the SIP systems have equivalent seismic design coefficients, as given in the 2003 IBC (ICC), for light framed walls sheathed with wood structural panels. Based on limited testing of SIP assemblies with sealants, the assemblies can meet the aforementioned criteria. It should be noted that since the performance of SIP systems is sensitive to fastener type and sealant formulation, additional matched tests may be required with significant changes in sealant formulation and fastener type.

38-15-1 E Karacabeyli, M Yasumura, G C Foliente, A Ceccotti
Background information on ISO standard 16670 for cyclic testing of connections

Abstract
ISO (International Organization for Standardization) Technical Committee on Timber Structures (ISO TC 165) convened in 1995 a working group (WG7) for the development of international standards for connections. As a first priority, the WG7 worked with a group of international experts and developed the ISO Standard 16670 "Timber Structures – Joints made with mechanical fasteners – Quasi-static reversed-cyclic test method" to provide a cyclic test method as a basis for derivation of parameters which are required in seismic design of timber structures. The cyclic test protocol in this standard was used in various research studies in testing of joints as well as shear walls. In this paper, the basic features and application of the ISO 16670 are presented along with its differences with respect to other standards.

Conclusion and Recommendations
The background information and some key features of ISO Standard 16670 for cyclic testing of joints in timber structures under earthquake loads are presented. Because the standard was developed through international collaboration, it is used and referenced in many countries. The inclusion of ISO 16670 cyclic displacement schedule in the ASTM Standard 2126 (cyclic testing of shearwalls) is a positive step towards international harmonization. It is recommended that a similar step be considered by other national standards committees. It is also recommended that research studies include reference tests conducted with ISO 16670 schedule to enhance the comparability of results from other studies.

As performance-based engineering becomes more common in the design of structures, designers will increasingly rely on the performance data provided by testing, which reflects the demands from in-service conditions. International standards provide a consistent basis for performance comparison of systems and exchange of technical information, and facilitate cooperative efforts to develop analytical models and improved design procedures for timber construction.

39-15-2 M Follesa, M P Lauriola, C Minowa, N Kawai, C Sandhaas, M Yasumura, A Ceccotti
Which seismic behaviour factor for multi-storey buildings made of cross-laminated wooden panels?

Abstract
Day by day, multi-storey buildings made of cross-laminated wooden panels (XLam) are becoming a stronger and economically valid alternative to their counteracts built with concrete and masonry. Throughout Europe and even in seismic prone zones, this construction type is gaining a broader acceptance.

However, until now, in Eurocode 8 this constructive system is not yet included and no recommendations are given regarding constructive details. Especially regarding the value of the seismic behaviour factor to be used in seismic design of this new typology of wooden buildings, no comprehensive investigations have yet been undertaken.

In this paper, results from shaking table tests on a three-storey cross-laminated wooden building are presented and the value of the seismic behaviour factor is found on the base of the actual response of the building to one quake.

Discussion

Being the design ground acceleration PGA_{code}, equal to $0.35g$, by applying the procedure given in 2.2, the q value is:

$$q = 1,20 / 0,35 = 3,4$$

Of course the above value is valid only referring to the used Nocera Umbra ground motion record. A series of different quakes should be used with the same procedure. This is obviously impossible; therefore the importance of a good mathematical model that can simulate different quakes and cases is obvious.

In any case, the above value has its own significance as an indicator. Moreover, it must be considered that the building has passed without any important reparation at least 14 "destructive" quakes in a row. It has kept its shape even with the last quake that has produced the near-collapse state. That means that this typology seems very promising when the design philosophy in seismic areas would convert to the NDD – no damage design – approach.

39-15-3 A Heiduschke, B Kasal, P Haller
Laminated timber frames under dynamic loadings

Summary

This paper discusses the results of time-history analyses of four-story moment-resisting timber frames subjected to various seismic loadings. The purpose of the case study was to document the difficulties, which arise due to large story drifts of the frames caused by the soft character of moment transmitting connections using dowel-type fasteners. Two frames were designed: one according strength and the second according stiffness requirements stipulated by the seismic code – EC8. From the design procedure it follows that, in general, the serviceability limit state design is the limiting criteria. Unacceptable lateral drifts made it necessary to increase the joint stiffness. Since connection stiffness increases significantly with the depth of the member, the member size inclusive the number of the fasteners was enlarged. This, in combination with densified and textile reinforced wood resulted in a frame design with an adequate lateral stiffness keeping the inter-story drift within the limit stipulated by the seismic code. The numerical model used to simulate the structural response of the frames utilized the hysteretic formulation of nonlinear connection behavior. The results of the dynamic pushover analysis were used to evaluate and verify the behavior factor q. In the seismic code, the elastic spectrum is modified with this factor to obtain an inelastic design response spectrum that is used to determine equivalent static forces.

Conclusions

From the design procedure it follows, that in general the SLS design will be the limiting criteria in the design process of moment-resisting frames. The inter-story drift of the frame designed in accordance to the strength and ductility criteria, exceeded the seismic code limitation at the SLS. The unacceptable deformations made it necessary to increase the joint stiffness. Since the stiffness increases significantly with the depth of the member size inclusive the number of the fasteners was enlarged. This, in combination with densified and textile reinforced wood, resulted in a frame design with an adequate lateral stiffness keeping the inter-story drift within the limit stipulated by the code. Over-sized members and a lateral strength larger then the design story shear characterize this frame-S, designed according to the stiffness requirements.

From the modal analysis one can conclude that for the evaluation of the first fundamental period of the system, the connection behavior has to be taken into account. For the frame-C, long periods ($T > 2$s), were evaluated due to the low rotational stiffness of the connections. The structures were not within the period range of typical earthquakes and this result in small inertia forces. In order to get reasonable structural deformations, specific earthquakes, characterized by soft soil conditions were selected for the nonlinear time-history analysis.

The frame-S had an adequate lateral stiffness to keep the inter-story drift at a reasonable level for strong ground motions up to 0.6g. In either simulation the frame performance improved with story drift reductions of 25% to 160% when using densified wood.

The cyclic test on connections have shown that textile reinforced joints have at least medium ductility, so that these frame types can be classified as structures having a medium capacity to dissipate energy. As a consequence, a q-factor of 2.5 can be assumed for the studied statically indetermined frames. The numerical time-history analysis showed that a q-factor of 2.5 is acceptable for the investigated frames.

39-15-4 S Pampanin, A Palermo, A Buchanan, M Fragiacomo, B Deam
Code provisions for seismic design of multi-storey post-tensioned timber buildings

Introduction

Recent developments and successful preliminary experimental validations of innovative types of ductile connections for multi-storey seismic-resisting laminated veneer lumber (LVL) timber buildings have opened major opportunities for extensive use of structural timber in seismic regions. These particular solutions, named jointed ductile connections or hybrid systems are based on post-tensioning techniques to assemble structural LVL members for both frame and shear wall systems which are designed to exhibit controlled rocking deformations during seismic loading. These systems have been proposed and successfully tested using concepts developed for high-performance seismic-resisting precast concrete buildings, currently being approved in major seismic codes and design guidelines worldwide. The extremely satisfactory results of quasi-static cyclic and pseudo-dynamic experimental tests on exterior beam-column joint subassemblies, column-to-foundation connections and shear wall systems have provided valuable confirmation of the high seismic performance of these LVL systems, as well as the reliability of the adopted design criteria and methodology. In this paper, after a brief introduction to the concept of post-tensioned seismic-resisting LVL structures and an overview of experimental results, particular focus will be given to seismic design aspects, within a performance-based design approach, as a sound basis for the preparation of seismic design code provisions.

Conclusions

Innovative damage-resistant solutions have been developed for the seismic design of multi-storey LVL timber buildings, following current international trends towards performance based seismic design and technological solutions for high seismic performance, based on limited levels of damage. The results of an ongoing extensive experimental campaign have confirmed the enhanced performance of jointed ductile connections (also referred to as hybrid systems) with a combination of post-tensioned tendons and energy dissipaters. When compared to traditional solutions widely used in timber construction (e.g. nailed, bolted or steel dowel connections) limited levels of damage can be achieved thanks to controlled rocking mechanisms at the critical connection interfaces. Re-centering properties, leading to negligible residual deformations and limited cost of structural repairing, are provided by unbonded post-tensioned tendons. Simple and reliable design and modelling procedures, developed for precast concrete structures and implemented in major seismic codes, can be adopted with minor modifications for the design of innovative high performance LVL structures and can be proposed for adoption in the next generation of timber design codes and guidelines. It is clearly anticipated that the flexibility of design and the speed of construction of prefabricated LVL buildings, combined with the intrinsic enhanced seismic performance of hybrid systems, creates unique potential for future development and increased use of this type of construction in low-rise multi-storey buildings on a world scale.

40-15-4 M Popovski, A Peterson, E Karacabeyli
Seismic behaviour of tall wood-frame walls

Abstract

A series of 13 quasi-static tests were performed to determine the behaviour of tall wood-frame shearwalls subjected to seismic loads. The walls tested were 4.9 m x 4.9 m in size and included different sheathing-to-stud nailed connections, two types of studs and blocking (spruce-pine-fir dimensional lumber and laminated strand lumber), various stud spacing, various stud-to-plate connections, sheathing material and thickness. The research results showed that with efficient stud spacing, nailing pattern, stud-to-plate connection details, and appropriate sheathing thickness, both spruce-pine-fir (SPF) and laminated strand lumber (LSL) studs are viable material options for tall walls. Walls that used LSL studs spaced up to 2440 mm on centre were able to withstand large lateral forces and dissipate high amounts of hysteretic energy. Tall walls with SPF studs spaced 610 mm on centre, aside from being able to withstand large lateral forces, showed increased ability to sustain large deformations provided that close nail spacing is used. An arrangement consisting of commercially available double hurricane ties and a joist hanger was found to be an effective stud-to-plate connection resisting the shear and uplift forces.

Conclusion and Design Recommendations

In this paper, results are presented from a series of quasi-static tests aimed to determine the seismic behaviour of tall wood-frame walls. Tall walls

4.9 m x 4.9 m in size were tested with a variety of sheathing-to-stud connections, stud material, stud spacing, stud-to-plate connections, as well as sheathing material and thickness. Findings and observations from this project are anticipated to be useful for development of design guidelines for tall walls subjected to seismic loads in wood design codes and standards. Some of the design recommendations are given bellow:

(a) Tall-walls with both, SPF and LSL studs can be effective lateral load resisting systems. Stud tables for dimensional lumber and some engineered wood products such as laminated veneer lumber spaced at 305 mm, 406 mm and 610 mm on center subjected to wind loads are given in the "Tall Wall Workbook" (CWC, 2007);
(b) From a cost competitive point of view, tall walls with engineered wood product studs, such as LSL, should be used with larger stud spacing (1.2 in and larger) and thicker sheathing (up to 28.6 mm). The larger stud spacing per se, was not found to be detrimental to the tall wall performance against lateral loads, without the presence of significant simultaneously applied vertical load;
(c) Use of thin (9.5 mm) sheathing should be avoided in combination with studs made of denser wood products such as LSL, due to the increased presence of nail pull-through the sheathing failure mechanism. Such mechanism doesn't allow for full potential of the nailed connections to be developed. On the other hand, nails in tall walls made with 25.4 mm thick plywood were able to develop three or more plastic hinges during testing, thus providing the wall with high lateral load capacity and energy dissipation;
(d) Tall walls may be designed as blocked or unblocked. Blocked tall walls, especially in case with engineered wood studs, are more cost effective per unit of shear resistance;
(e) For lumber stud walls, use of higher strength lumber or engineered wood products for the top and bottom plates is recommended to resist the induced uplift and anchorage loads;
(f) For walls with a stud spacing larger than 610 mm, top plate deformation between the studs shall be checked, as it may become an issue, especially when transferring large gravity loads in case when roof joists are placed perpendicular to the wall line;
(g) Stud-to-plate connections are of great importance for the overall performance of tall walls. An arrangement consisting of off-the-shelf dual hurricane ties and a hanger proved to be an efficient and cost-effective solution for the stud-to-plate connections, for both, site built and prefabricated tall walls. Use of asymmetrical stud to plate connections, such as single tie or custom made connectors that connect the non-sheathed side of the stud only, should be avoided in high seismic zones due to susceptibility of the studs to torsional failure;
(h) Attention should be paid to the design of the anchoring devices for tall walls, so that they can efficiently transfer the load to the foundation, without damaging the bottom plate. For example large plate washers may be used to prevent splitting of the bottom plate;
(i) The effect of vertical load (in the amount of 20kN/m) on a tall wall with dimensional lumber studs resulted in a slight increase in the lateral load capacity, ductility and energy dissipation compared to a non-vertically loaded wall with similar properties;
(j) Application of gypsum wallboard on the other side of the wall increased the stiffness and the lateral load capacity of the tall wall. The maximum load, however, occurred at displacement drifts lower than in equivalent shearwalls without gypsum boards. For walls with LSL studs for example, the maximum load occurred at a drift of 1.8% for wall without gypsum vs. 1.2 % for wall with gypsum wallboards. The ultimate displacement drifts, however, occurred at relatively similar drift levels (1.9 % vs. 2%);
(k) Test results suggest that tall walls with wood-based panel on one side and a gypsum wallboard on the other should be assigned a lower ductility-related force modification factor (Rd factor) than tall walls with wood-based panels only. A similar provision is already introduced in the Canadian Standard for Engineering Design in Wood for standard shearwalls.

41-15-5 P Scläddle, H J Blass
Behaviour of prefabricated timber wall elements under static and cyclic loading

Introduction

Prefabricated Timber Wall Elements (PFTE) represent a simple, easy to handle and sustainable construction system. In a current research project at Universität Karlsruhe the PFTE building system is tested under both vertical and horizontal loading to determine its shear wall capacities. For this purpose a new testing assembly for vertical and horizontal loads able to produce various boundary conditions was installed at Universitdt Karlsruhe. The shear walls were tested following ISO/CD 21581 [3] while assuming boundary conditions reflecting the intended construction details. In this paper the test results are presented and are compared with test results of conventional timber frame walls.

Discussion and future prospects

All tests were carried out using ISO/CD 21581 [3]. Boundary conditions were assumed to reflect the actual building conditions. At high vertical loads the shear capacities were achieved. The practicability of ISO/CD 21581 [3] is determined; the applicability also for exceptional timber construction systems is proven.

The system with PFTE showed good performance in monotonic and cyclic testing as well. In monotonic tests the results for maximum horizontal load and for stiffness values are quite similar to conventional timber frame systems.

PFTE showed excellent results for the energy dissipation in cyclic loading, enlarging its potential range of application to seismic and windstorm prone areas. Further work is being done to improve the hold-down of the vertical tensile studs. The PFTE system can cover the same application range as conventional timber frame buildings, yet it is easy to handle and therefore cost effective.

Future research work will be developing a finite-element model to simulate the system properties and to give basics to be implemented in codes.

42-15-2 M Popovski, E Karacabeyli, Chun Ni, G Doudak, P Lepper
New Seismic design provisions for shearwalls and diaphragms in the Canadian standard for engineering design in wood

Abstract

This paper summarises the newly developed seismic design provisions for shearwalls and diaphragms that were introduced in the 2009 edition of the Canadian Standard for Engineering Design in Wood (CSA O86). The new provisions address seismic design loads for wood diaphragms, shearwalls, anchor bolts, hold-down connections, shearwall-to-diaphragm connections and similar load transfer elements. In addition, the provisions include clauses for wood-based diaphragms used in hybrid buildings with masonry, concrete, or steel vertical seismic force resisting systems (SFRSs). Two different design approaches are provided: a) cases when diaphragms may yield; and b) cases where diaphragms are stiff and are not expected to yield. The new design provisions have significantly improved the alignment of CSAO86 with the current 2005 edition of the National Building Code of Canada (NBCC), as well as with the upcoming 2010 edition of NBCC.

Conclusion

The paper describes in detail the newly developed seismic design provisions for shearwalls and diaphragms that are included in the 2009 edition of the Canadian Standard for Engineering Design in Wood (CSAO86, 2009). The new design provisions have significantly improved the alignment of CSAO86 with respect to the current 2005 edition of the National Building Code of Canada (NBCC), as well as with the upcoming 2010 Edition of NBCC.

43-15-2 P Schädle, H J Blass
Influence of different standards on the determination of earthquake properties of timber shear wall systems

Introduction

Shear wall tests on two modern timber construction systems were carried out by Karlsruhe Institute of Technology (KIT). Regarding test results such as stiffness, horizontal load-carrying capacity, hysteresis shape and hysteresis equivalent viscous damping ratio, all results are similar to or even better than the well-known timber frame system. This means that both systems should also be suitable for the use in seismic active areas.

Innovative systems usually do not fit into the design concepts according to Eurocode 8 (EC8), thus their behaviour factor cannot be found there. The only approach to classify systems into a ductility class specified in EC8 is the declaration of a static ductility. This is insufficient because important characteristics like the energy dissipation and the boundary conditions of the tests are not taken into account. Since no uniform standard for the determination of seismic properties of timber construction systems exists, several problems are identified.

Following from the insufficient static-ductility-approach, the ductility classes for the systems would be too conservative. Thus the evaluation of the behaviour factor q for the tested systems was carried out using a numerical simulation, taking into account the essential properties of the system considered. Difficulties when determining q are described in this paper since several standards influence the value of the behaviour factor. A possible solution of this problem is proposed.

5.2 FIRE

19-16-1 J König
Simulation of fire in tests of axially loaded wood wall studs

Abstract
The results from tests of axially loaded wood studs are presented. During the tests, material was removed by planing, thus simulating the charring in the case that the stud is exposed to fire on one side only. The studs were placed between stiff, non-rotating endplates. The stud ends were allowed to rotate and thus the axial load was able to change its location and decrease load eccentricity. A test procedure is proposed for the determination of fire resistance of load-carrying wood-stud walls.

Conclusions
The support conditions of a column with end hinges can be more favourable in a structure than they normally are assumed to be in calculations or tests. Axially loaded wood studs behave in such a manner when their slenderness ratios are large. By means of the large rotation of the stud ends, the axial load is allowed to move in the direction of the deflection. The influence of support conditions as intermediate layers of cellular rubber profiles and the inclinations of support or roof trusses is negligible when the slenderness ratio is large. These conditions are particularly pronounced in case of fire causing charring only on one side of the stud. The assumption that the axial load has a fixed location will give very conservative results.

The following outline of procedure for determining fire resistance of load-bearing wood stud walls with both axial and transverse loads is proposed. Since the thermal effects on stiffness and strength of small size timber components are not yet known to the author, fire tests under load still seem to be inevitable. The following steps should be taken:

1. Fire-testing of a wall unit under load. The wall unit is placed in a test rig with fixed end plates, allowing the upper and lower wall ends to rotate. The axial load should be chosen to be close to the design load. If the wall unit has not collapsed after the specified period of fire resistance, the fire is put out and the axial load increased until collapse load is reached. The collapse behaviour, ductile or brittle, is registered. The profile of effective, residual cross-section is measured.
2. Specimens made of one stud plus effective parts of cladding are made. The stud has a cross-section which is approximately equivalent to the residual cross-section of the fire-tested studs (step 1).
3. "Cold tests" of one part of the specimens with axial load are undertaken until collapse. The ultimate load should normally be lower than the ultimate load obtained from the fire test.
4. "Cold tests" are conducted on the other part of the specimens with transverse load until collapse.
5. Assuming that the thermal effects on bending strength are approximately the same as in the case of axial compression of the stud, the bending capacity during fire can be calculated by decreasing the result from the "cold tests" in the same proportion as was obtained from comparison of the test results from step 3 and 1.
6. Now the load bearing capacity for combined axial and transverse loads can be determined by using the interaction formula
$$\frac{N}{N_d}+\frac{M}{M_d}\leq k$$
where Nd is the characteristic axial load bearing capacity obtained from step 1 and Md is the characteristic moment bearing capacity determined as described above. The coefficient k should be chosen between 0.9 and 1.0.

The use of this method offers the advantage that different load combinations can be chosen and expensive fire testing minimized.

24-16-1 J König
Modelling the effective cross section of timber frame members exposed to fire

Abstract
The results from fire tests on light, partly protected timber frame members under pure bending are used in order to model the effective cross section of this type of structural members. Since the test results made it possible to regard the influence of some parameters as the influence of the load level in relation to load capacity at normal temperature, the state of stresses, the loading rate and density, the modelling of structural timber in fire is discussed in a wider perspective and some of the consequences for the requirements of an analytical model are described.

Conclusions

It is obvious that, in general, different notional charring rates should be used for modelling the effective section not only with respect to load capacity and stiffness but also with respect to the state of stresses. In many applications this distinction is not necessary as in the case of fire-exposure on four sides or of very large sections. While in calculations of bending load capacity a loss of strength should be taken into account, it is not necessary to regard the loss of the modulus of elasticity when the flexural stiffness of the member is determined. Therefore in design rules notional charring rates should be linked together with rules about, whether, or how much the strength and the modulus of elasticity has to be reduced. In the case of the specimens of the investigation referred to here, it was possible to use only two different charring rates in connection with different reductions of strength and modulus of elasticity, see Figure 16. In other applications this may be different.

Assuming a simple effective cross section of rectangular shape it is possible to achieve good agreement between calculated and test results. It is therefore unnecessary to introduce rounded edges in the model. In most cases the increase in accuracy of the model should be illusory whilst its use would be more complicated.

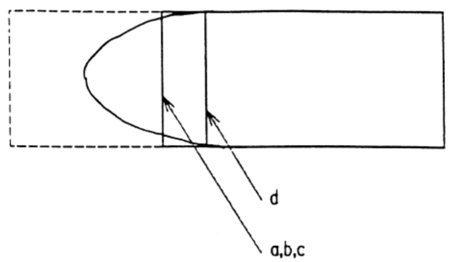


Figure 16 Effective cross sections for uncharred core shown in Figure 1 linked to the following assumptions:
a) Ultimate load - fire-exposed side in tension - $f=0,75\ f_m$
b) Ultimate load - fire-exposed side in compression - $f=0,55\ f_m$
c) Flexural stiffness - fire-exposed side in tension - E not reduced
d) Flexural stiffness - fire-exposed side in compression - E not reduced

By using the notional charring concept it is possible to specify expressions which are specific to different applications. Thus the concept is sufficiently general and should be useful in future applications.

25-16-1 J König
The effect of density on charring and loss of bending strength in fire

Summary

The influence of density on charring of timber exposed to standard fire is studied, evaluating test results by Norén. It was found that both the effective and measured charring rates vary about 10 % in the density interval between 290 and 420 kg/m^3, representing characteristic densities of strength classes C14 to C40 in EN 338. It was found that there was no influence of density on the loss of bending strength.

Conclusions

The influence of density on the charring rate is considerably smaller than given in the CIB Timber design code. Since it is small in the interval of most used structural classes, it should he disregarded in practical applications.

25-16-2 F Bolonius Olesen, J König
Tests on glued-laminated beams in bending exposed to natural fires

Summary

A series of fire tests with so-called natural fire exposure of loaded glued laminated beams was performed. The fire exposure of three sides during the tests was governed by a temperature-time relationship determined according to an energy balance method (opening factor method) with different fire load densities and opening factors. The results confirmed the rate of charring on the wide side of the member obtained by Hadvig. The average charring depth on the lower side of the member was greater than the average charring depth on the wide side when the width of the lower side was smaller than about 180 mm.

Considering the mechanical behaviour, the tests showed that a loss of strength and stiffness of the residual cross section occurred, and that it

continued through the cooling period caused by continuous heat flow to the inner parts of the cross section.

Conclusions

In structural fire design the charring rates obtained by Hadvig should he used for the wide vertical sides of a member. The charring rates on the narrow side according to Hadvig could not be confirmed. The reason for this might be that the number of tests was too small.

Compared to the conditions at standard fire exposure, the mechanical behaviour at natural fire exposure is different due to the changes of temperature in the residual cross section during the cooling period. In natural fires the bending strength and stiffness is lower than in standard fire. The influence of elevated temperature is no longer concentrated to the outer layer of the residual cross section. Thus the concept of a reduced bending strength of the residual cross section should he applied.

26-16-1 J König
Structural fire design according to Eurocode 5, Part 1.2

Summary

In June 1993 the final draft Eurocode 5, Part 1.2 - Structural Fire Design - was approved to be published and introduced as a European prestandard for provisional application in the EC and EFTA countries. In this paper an overview is given on the contents of Eurocode 5, Part 1.2, its background, requirements, methods and design philosophy.

31-16-1 J König
Revision of ENV 1995-1-2: Charring and degradation of strength and stiffness

Summary

Calculating the load-carrying capacity of fire exposed timber members, the designer needs information on, firstly, charring depths in order to determine the cross-sectional properties of the residual cross section, and secondly, the degradation of strength and stiffness. In the present Fire Part of Eurocode 5, charring is dealt with in a crude way. Especially, it is not possible to consider charring of protected timber surfaces in a rational way. The present model may lead to unsafe or in other cases unduly conservative results. According to the proposed model, charring is properly defined, and a distinction is made with regard to different charring and protection phases.

For parametric fire exposure, the present method is not operational. Reviewing the background papers and extending the method for determination of strength and stiffness degradation according to annex A of ENV 1995-1-2, a revised model is proposed for the determination of relationships between load-carrying capacity and time.

Introduction

ENV 1995-1-2, the Fire Part of Eurocode 5 was published by CEN in November 1994. This European pre-standard and the Fire Parts of the other Eurocodes have now (April 1998), somewhat delayed, passed the stage of Two-year enquiry. CEN Member Bodies, i.e. the National Standards Organisations were asked to submit comments on this European pre-standard (ENV) to be considered during the redrafting for conversion to a European Standard (EN). The result of this enquiry was that a majority voted for redrafting and conversion to EN of all Fire Parts.

The intention of the Two-year enquiry was to collect National comments based on the experiences from experimental application during the first two years after the publication of the pre-standard. Due to considerable delay in introducing the Fire Part of Eurocode 5 (and the other Fire Parts) as National Pre-standards for experimental use, however, only very few comments were submitted expressing the views of designers working in the field.

This paper discusses the two properties of timber that are most essential with respect to the load-carrying performance of timber structures in the event of fire: The charring of the timber member giving rise to a reduction of the cross section of the member, and the degradation of strength and stiffness (modulus of elasticity) of the residual cross section due to the influence of temperature and moisture. It is stressed that the format should be simple, although it should open up for extended application using data not given in the Fire Part or not yet available.

33-16-1 J König
A design model for load-carrying timber frame members in walls and floors exposed to fire

Summary

A design model is presented for the calculation of the load-bearing capacity of timber framed wall and floor assemblies exposed to standard fire exposure. It is implied that the cavities of the assemblies are filled with rock or glass fibre insulation, providing partial protection against charring of the wide sides of the timber members. The design model consists of a charring, model giving a simplified rectangular residual cross section, and a mechanical model describing the strength and stiffness properties of the residual cross section. The charring model takes into account different charring rates that are dependent on the protection provided by the lining, and the conditions during the post-protection stage after complete failure of the lining. Criteria are given for the failure of linings including the length of fasteners. The reduction of strength and stiffness parameters is taken into account by multiplying cold values by modification factors. The model parameters were derived from test data, heat transfer calculations and mechanical modelling of the cross section using temperature dependent strain-stress relationships.

33-16-2 J König, T Oksanen , K Towler
A review of component additive methods used for the determination of fire resistance of separating light timber frame construction

Summary

Verification of fire resistance of separating constructions is performed with respect to criteria I (insulation) and E (integrity). While fire testing is still the most common way of verification, design by calculation will be more common in the future. Several calculation methods called component additive methods have been in use since several years. All of them stipulate that in a construction built up of several layers or membranes, the total fire resistance is the sum of the contributions to fire resistance of each of the layers. This paper reviews and compares the design method given by ENV 1995-1-2 with methods being used in North America, the United Kingdom and Sweden. A proposal for the revised Fire Part of Eurocode 5 is presented.

Conclusions

The determination of the fire resistance of floor and wall assemblies by calculation is a complex task. In this paper four methods for the determination of the fire resistance with respect to the separating function of light timber framed assemblies are presented. Two of them include also the verification of the load-bearing function. All of them have advantages and drawbacks, due to conflicting goals: accuracy versus simplicity. The opinion of the Project Team redrafting the Fire Part of Eurocode 5 is that a design code should contain rational calculation methods, while tabulated data for constructions should be found in design manuals.

This criterion would exclude the UK-method.

The North American method is easy to use, however the logic behind is not obvious to the designer.

Since the Swedish method describes the real performance more adequately without being too complicated, it is proposed to adopt this method in the EN 1995-1-2 and to combine it with the existing ENV-method. Since the method is still incomplete, some corrections must be made to fit test results. It is also proposed to replace the tabulated data of coefficients of positions by simple equations. Since the method was developed only for wall constructions, it must be extended to include floor application, e.g. by introducing a general reduction of coefficients of position. The effect of joints should be included as in ENV 1995-1-2. It is proposed to apply the method for the heat transfer paths given in the ENV with the exception of heat path b that should include all layers in order to take into account joints on the unexposed side. For that heat path b the value t_{ins} according to equation (3) should be increased, in order to adapt the time criterion to an temperature increase of 180 K instead of 140 K.

The main advantage of the proposed method is that it is more general and open for future extensions both by tests and general calculation methods.

Since integrity failure due to premature failure of linings may occur resulting from excessive deflections of load-bearing elements, a corresponding rule should be included. From fire tests can be seen that deflections of load-bearing constructions become excessive during the last 5 to 10 minutes prior to collapse.

33-16-3 B Källsner, J König
Thermal and mechanical properties of timber and some other materials used in light timber frame construction

Summary
The Fire Part of Eurocode 5 gives the option of applying, simplified, more complex and general design rules. For assessment of general design rules, or the derivation of simplified design rules for thermal and mechanical analysis, material properties are needed such as thermal conductivity, specific heat capacity and reduced mechanical properties at elevated temperatures in combination with effects of moisture. Unfortunately there is a large variation of values given by different sources. This paper discusses different values and the dependence of calibrated properties on the applied model and gives the background of the values proposed to be adopted in the Fire Part of Eurocode 5.

Conclusions
In the calibration procedure used, the main principle was to use thermal properties that could be found in the literature and to modify them, where necessary, in order to get results in agreement with results obtained from testing of fire exposed timber or wall and floor assemblies. The thermal properties should be calibrated such that they could be used independent of the physical arrangement of the materials in the structures. Thermal properties to be adopted in a design code cannot be true values as such; they are rather apparent or fictitious values. They should reflect the physical properties and phenomena in an understandable way such that designers – who for this type of analysis should be specialists in the field of fire design – can use them in an appropriate way. The thermal and design data presented here give safe results and should therefore be suitable for adoption in EN 1995-1-2.

34-16-1 I Totev, D Dakov
Influence of the strength determining factors on the fire resistance capability of timber structural members

Introduction
The main objective of this research is to obtain data for the influences of the strength determining factors on the fire resistance of softwood beams, subjected to bending. The relationship between growth rate and charring rate was researched. It is easy to establish the number of growth rings. It allows rapid approximate assessment of the fire resistance through relationship between growth rate and charring rate.

Knots and fissures are important factors for assessment of the behaviour of the timber elements in fire conditions as well.

Conclusions
Relationships are obtained between the number of growth rings, charring rates and the modulus of elasticity. Estimations for fire-resistance of exiting timber structures are possible because it is easy to determine the number of growth rings. The distance between the knots and bottom edge of the beam is a useful bit of information about expected fire-resistance. Regrading of the beams helps in carrying out the correct assessment. Methods that predict fire resistance based on data about growth rate, disposition and size of knots and fissures could be developed. They could be employed by insurance companies when assessing the fire risk of existing timber structures.

34-16-2 J König, B Källsner
Cross-section properties of fire exposed rectangular timber members

Summary
The Fire Part of Eurocode 5 gives the option of using methods of different complexity for the determination of the uncharred residual cross section and the reduction of strength and stiffness parameters. By using advanced methods of heat transfer calculations, using the thermal properties of wood and the char layer given in Draft prEN 1995-1-2, it is shown that the notional charring rates given in the code are reasonable, allowing the designer to disregard corner roundings. This paper gives the background of the different methods for the reduction of strength and stiffness. Advanced calculations using the thermo-mechanical properties of timber given in Draft prEN 1995-1-2, show that the reduced cross section method gives fairly accurate results for members in bending, while it is unsafe in relation to the advanced method for members in compression or tension. The reduced properties method, however, is unsafe for members in bending, compression and tension.

34-16-3 A Mischler, A Frangi
Pull-out tests on glued-in rods at high temperatures

Summary
Connections and reinforcements with glued-in steel rods are becoming more and more important in timber engineering. The load-carrying behaviour of this connection type at room temperature is investigated in many research projects.

Not only the knowledge of the resistance at room temperature is needed but, in some cases, also the fire resistance of a connection. At present, there is only little information available about the fire resistance of this connection type. One problem is that the different adhesives are more or less sensitive to high temperature. Therefore, the fire resistance of these connections has to be investigated.

This paper presents results of tests at different temperatures with axially loaded rods glued in glued laminated timber parallel to the grain.

Conclusions
Tests on screws and glued-in rods with oven heated specimen showed the influence of the temperature on the properties of the timber and the adhesive. Comparative tests with specimen subjected to ISO-fire and oven heated specimen showed, that the strength and stiffness values obtained in fire tests are smaller than the values measured on oven heated specimen.

35-16-1 J König
Basic and notional charring rates

Summary
In the Fire Part of Eurocode 5 basic and notional charring rates are given. In order to simplify the determination of the load bearing capacity of cross sections, a notional charring rate is given to be used on all sides of the cross section. It is shown that the same notional charring rate can be used also for very small or narrow cross sections with a great influence of two-dimensional heat transfer, for example very narrow timber joists exposed on three sides.

The basic charring rate is strictly valid only in the case of one-dimensional heat flux and its use requires separate consideration of corner roundings of the char line. A criterion is derived to define when the notional charring rate should be applied.

From the results of recent fire tests notional charring rates are derived to be applied to heavy laminated timber plates, such as nail laminated timber decks and walls where gaps may open between laminations due to drying in service.

Conclusions
It has been shown that the concept of notional charring rates is a tool that can be widely used to simplify the calculation of cross-sectional parameters by determining an equivalent rectangular residual cross-section. This is an option in order to simplify the calculation or when the real shape of the residual cross-section cannot be determined unless advanced heat transfer calculations are performed.

37-16-1 J König
Effective values of thermal properties of timber and thermal actions during the decay phase of natural fires

Abstract
For the thermal analysis of structural or non-structural timber members, using conventional simplified heat transfer models, thermal conductivity values of timber are normally calibrated to test results such that they implicitly take into account influences such as mass transport that are not included in the model. Various researchers and designers have used such effective thermal conductivity values, originally determined for standard fire exposure, to evaluate other fire scenarios such as natural fires. This paper discusses in qualitative terms some parameters that govern combustion of wood and their influence on effective conductivity values. Reviewing fire tests of timber slabs under natural fire conditions, it is explained why effective conductivity values, giving correct results for the ISO 834 standard fire scenario, should not be used in other fire scenarios. Therefore, the thermal properties of timber given in EN 1995-1-2 are limited to standard fire exposure. As shown by heat transfer calculations, the effective thermal conductivity of the char layer is strongly dependent on the charring rate and varies therefore during a natural fire scenario. It has also been shown that char oxidation during the decay phase in a natural fire has a significant influence on the temperature development in the timber member,

since char surface temperatures exceed the gas temperature in the compartment or furnace. Using increased effective gas temperature as thermal action during the decay phase, and varying conductivity values for the char layer, fairly good agreement could be obtained regarding the temperature development in the timber member and the char depth.

Conclusions

Due to the complex nature of heat transfer in wood and charcoal, including phenomena like the formation of fissures in the char layer and the internal convection due to pyrolysis gases and vaporized water, it is necessary to use effective rather than physically correct thermal properties when simplified models are used that do not explicitly take into account mass transport. For timber structures, this need appears to a considerably greater extent than for concrete structures where moisture transport plays a role. The formation of fissures in the char layer due to char contraction has a great influence on the effective conductivity of the char layer.

It has been shown that effective thermal properties that have been verified for standard fire exposure would give incorrect results when applied to other fire scenarios such as parametric fire curves for natural fires. Due to convective cooling due to the reverse flow of pyrolysis gases, the heat flux through the char layer into the wood is delayed. Since this delay is dependent on the rate of charring, the effective conductivity is a function of the charring rate; thus it is also a function of the rate of increase or decrease of temperature.

Char oxidation may occur during the decay phase in natural fires giving rise to char surface temperatures that are considerably above the gas temperature in the furnace or fire compartment. It has been shown that, by assuming effective conductivity values and modified thermal actions using an effective gas temperature in the compartment, good agreement can be obtained between the results from calculations and fire tests.

The present limitation of the use of effective thermal properties should be overcome by deriving them from analytical models that take into account the phenomena described above. Since, for the time being, no such model exists, as an alternative, extensive series of fire tests with various fire scenarios need to be performed. Such fire tests should also be a prerequisite for the validation of models that take into account the relevant parameters governing combustion of wood.

37-16-2 A Frangi, A Mischler
Fire tests on timber connections with dowel-type fasteners

Introduction

The present Swiss fire regulations allow the use of combustible materials only for a fire resistance of up to 30 minutes. With the revision of the fire regulations in the year 2005 the fire authorities will allow the use of timber also for the fire resistance class of 60 minutes. This will lead to new markets for timber, particularly for multi-storey buildings.

As the existing design rules for timber connections in fire are only valid for fire resistances up to 30 minutes, a large research project on the fire behaviour of timber connections is currently carried out in Switzerland sponsored by the Swiss Agency for the Environment, Forests and Landscape. The research project aims to enlarge the theoretical and experimental background of the fire behaviour of connections used in modern structural timber engineering. The application of the existing design rules for a fire resistance of 60 minutes will be checked based on the results of experimental tests. Further, new design models for timber connections with fire resistances of 60 minutes will be developed and validated. The design models should be published in a revised SIA Documentation 83, commonly used in Switzerland for fire resistance calculations of timber structures.

From the variety of timber connections, mainly multiple shear steel-to-timber connections with dowels and slotted-in steel plates are studied in the ongoing research project. In recent years, the load-carrying capacity of this type of connection has been thoroughly investigated at the Swiss Federal Institute of Technology of Zurich (ETH). Besides of a high load-carrying capacity and a ductile failure mode, the advantage of this connection is the protection of the slotted-in steel plates against the fire. Therefore a high fire resistance may be achieved.

As experimental tests on timber connections subjected to fire have been recently performed mainly using timber-to-timber connections, the fire behaviour of steel-to-timber connections was experimentally analysed with an extensive testing programme. Further, the fire tests were completed by a series of tests at room temperature. The paper describes the main results of the tests conducted on timber connections.

Conclusions

The structural behaviour of timber connections was experimentally analysed by a series of tests at room temperature as well as under ISO-fire exposure. From the variety of timber connections, multiple shear steel-to-timber connections with dowels and slotted-in steel plates and connections with steel side plates and annular ringed shank nails are studied in the ongoing research project.

The tests at room temperature on multiple shear steel-to-timber connections with dowels and slotted-in steel plates showed high load carrying capacities as well as a ductile failure due plastic deformations of the steel dowels.

All unprotected multiple shear steel-to-timber connections with dowels tested under constant load of $0.3F_u$, and subjected to ISO-fire exposure showed failure times between 30.5 and 35 minutes. A reduction of the load level to $0.15F_u$ and $0.075F_u$ led only to an increased failure time of about 3 and 8 minutes. A variation of the number or diameter of the steel dowels improved the fire resistance of max. 2 minutes. It can be generally concluded that the fire resistance of multiple shear steel-to-timber connections with dowels and slotted-in steel plates loaded with $0.3F_u$ is around 30 minutes. The connections with dowels protected by timber boards or gypsum plasterboards showed failure times of around 60 minutes. By increasing the timber covers of the steel dowels and the slotted-in steel plates as well the end distance to the dowels by 40 mm, the fire resistance of the connections reached more than 70 minutes. Thus, from a fire design point of view these modifications were very favourable in order to increase significantly the fire resistance of the connections.

Unprotected connections with steel side plates and annular ringed shank nails failed already after about 12 minutes due to large deformations of the nails and the steel side plates directly exposed to fire. By protecting the steel side plates by a fire proof-coat the fire resistance of the connections was increased up to around 30 minutes.

The test results permit to check the design rules given in EN 1995-1-2 and to verify advanced calculation models recently developed by Povel. Further, new simplified design models for timber connections with fire resistances of 60 minutes will be developed and compared to the test results. The temperature development will also be analysed using a FE-model. The design models should be published in a revised SIA Documentation 83.

38-16-1 C Erchinger, A Frangi, A Mischler
Fire behaviour of multiple shear steel-to-timber connections with dowels

Introduction

In recent years, the load-carrying capacity of multiple shear steel-to-timber connections with slotted-in steel plates and steel dowels has been thoroughly investigated at the Swiss Federal Institute of Technology in Zurich (ETH). Besides of a high load-carrying capacity and a ductile failure mode, the advantage of this connection is the protection of the slotted-in steel plates against fire. Therefore, a high fire resistance may be achieved.

In the last couple of years, many countries have started to introduce performance based fire regulations or liberalized the use of timber for buildings. These regulations open the way for new applications, in particularly for an extended use of timber structures in multistory buildings.

The fire behaviour of steel-to-timber connections was experimental analysed with an extensive programme sponsored by the Swiss Agency for the Environment, Forests and Landscape. The connections were tested with different timber thicknesses and different end distances of the fasteners. In addition to the unprotected connections some connections were tested protected by timber boards or gypsum plasterboards. The fire tests were completed by a series of tests at normal temperature.

Eurocode 5, part 2 gives design rules for symmetrical three-member connections made with nails, bolts, dowels, split-ring connectors, shear-plate connectors or toothed plate connectors. Besides simplified rules, the fire resistance of unprotected and protected connections can be calculated according to the "Reduced Load Method". The parameter k describing the exponential functions was determined for different connections using available test results. For steel-to-timber connections with dowels a value of k is only given for a diameter greater or equal than 12 mm. On the other hand, no value is given for multiple shear steel-to-timber connections with smaller diameters as tested in the ongoing research project.

Based on the main results of the fire tests, the paper analyses the efficiency of different strategies in order to increase the fire resistance of the connection. Further, particular attention is given to the comparison between test results and the design rules given in EN 1995-1-2. Based on the test results, the parameter k is calculated and compared to other connections to improve the design rules in EN 1995-1-2 for this connection type.

Conclusions

The paper analyses the efficiency of different strategies in order to increase the fire resistance of multiple shear steel-to-timber connections with dowels and slotted-in steel plates. Especially connections with a smaller diameter of the dowels were analysed. The laser-scanning method was applied to evaluate the residual cross-section. This method was used at the ETH Zurich for the first time and shaped up as a valid alternative to the mainly used manual methods. Compared to EN 1995-1-2 the charring rates showed a very good agreement. The influence of the heat flux through the fasteners was analysed by comparing the measured temperatures with calculation models developed for timber members without steel elements and fasteners under ISO-fire exposure.

The parameter k used in the "Reduced load method" according to EN 1995-1-2 was determined for a diameter of the dowels of 6.3 mm to 13,7. The results of the fire tests with a diameter of the dowels of 12 mm were compared to EN 1995- 1-2 and to steel-to-wood connections with bolts and slotted-in steel plates with a diameter of the bolts of 12 mm. While the value of $k = 0.085$ given in EN 1995-1-2 for steel-to-wood connections with bolts and slotted-in steel plates is confirmed by the test results, for multiple shear steel-to-wood connections with dowels and slotted-in steel plates a less conservative value may be assumed based on the results of the fire tests performed.

38-16-2 V Schleifer, A Frangi
Fire tests on light timber frame wall assemblies

Introduction

In the last couple of years, many countries have started to introduce performance-based fire regulations or liberalized the use of timber for buildings. These regulations open the way for new applications, particularly for an extended use of timber structures in multi-storey buildings. In taking advantage of the new possibilities it is essential to verify that the fire safety in timber buildings is not lower than in buildings made of other materials.

Currently, an extensive research project on timber construction in fire is carried out at ETH Zurich. One subproject studies the fire behaviour of light timber frame wall assemblies. The assemblies studied consist of solid timber members (studs) and non-combustible or combustible linings (or combinations) with or without cavity insulation made of rock, glass or wood fibre. Unlike heavy timber structures where the char-layer of fire exposed members performs as an effective protection of the remaining unburned residual cross-section, the fire performance of light timber frame wall assemblies depends on the protection provided by the linings and the cavity insulation.

The objective of the research project is the verification and extension of existing design methods of the separating function of wall and floor assemblies (e.g. as given in the Eurocode 5, part 1-2). Further new simplified calculation models will be developed based on new experimental and theoretical results.

This paper presents experimental results of an ongoing large testing program planned to enlarge the experimental background of the fire behaviour of light timber frame wall assemblies. Further the test results are compared to design rules given in Eurocode 5, part 1-2. Due to the important influence of the linings on the fire resistance, the type of material, thickness, position within the wall assemblies and number of layers was analysed.

Conclusions

The fire behaviour of light timber frame wall assemblies is currently analysed by a series of small-scale tests under ISO-fire exposure. The extensive testing program permits to study the influence of different parameters on the fire behaviour of light timber frame wall assemblies. In particular, the influence of the thickness and the position of the linings as well as the number of layers are studied.

All fire tests were conducted on the small furnace at the Swiss Federal Laboratories for Materials Testing and Research (EMPA) in Dübendorf. The linings studied on the fire exposed side were influenced by the layer behind them. For linings with the same material, the fire behaviour of the lining exposed to fire was influenced favourably. An unfavourable influence could be observed for the gypsum plasterboards on the fire exposed side with insulation materials. Further, this influence was independent on the type of the insulation material.

Mainly attention was given to the comparison of the tests results with the calculation model according, to EN 1995-1-2. For the lining studied the time and the charring depth of the layer behind the fire exposed lining were compared. The calculated values were in a good agreement with the test results of the gypsum fibreboards as well as the gypsum plasterboards type F. Only the small gypsum fibreboard with a thickness of 10 mm showed in the tests slightly unsafe fire behaviour as the calculation. The calculated time of start of charring of gypsum plasterboards type A was also confirmed by the test results. However, the failure time of the fire tests was underestimated and therefore the charring depth of wood based panels protected by gypsum plasterboards type A was smaller than the calculation according EN 1995-1-2. Based on the fire tests the calculated failure time of gypsum plasterboards could be increased. However, this assumption has to be confirmed by large scale tests. Furthermore the calculated time of start of charring of multilayer wood panels should be decreased, because the char-layer of these linings felt off earlier than the char-layer of solid panels.

39-16-1 T G Williamson, B Yeh
Fire performance of FRP reinforced glulam

Abstract
One of the emerging advanced engineered wood technologies in the United States and other countries is the use of high strength fiber-reinforced polymers (FRP) to reinforce the tension zone of structural glued laminated timber (glulam). Glulam is used in numerous long-span commercial building applications where the glulam is exposed for architectural reasons. For some occupancy use classifications, the U.S. building code mandates that these exposed glulam members be rated for one-hour fire construction. The most structurally efficient use of FRP reinforcement is to place the FRP on the outermost face of the glulam member. However, there is no published information on how a glulam beam reinforced in this manner will perform when directly exposed to fire.

In order to address this concern, representatives of APA - The Engineered Wood Association (APA), the Market Development Alliance (MDA) Engineered Wood Team, and the University of Maine Advanced Engineered Wood Composites Center (AEWCC) sponsored a fire test program of FRP reinforced glulam beams at Omega Point Laboratories in San Antonio, Texas. The purpose of the testing was to gain an understanding of the performance of FRP composites exposed to fire when used as a structural reinforcement for glulam beams.

This paper describes the results of two test programs conducted in accordance with ISO Standard 834. The first test program was a pilot study involving relatively small FRP reinforced glulam beams. Based on the results of this pilot study, a second test program was then undertaken to evaluate the fire performance of FRP reinforced glulam beams designed to achieve a one-hour fire rating. The results of these tests led to the development of a design methodology to permit establishing fire ratings for FRP reinforced glulam.

Conclusions
Based on these tests, it was clearly demonstrated that a glulam beam with an FRP applied to the bottom face of the beam and directly exposed to a fire (no bumper lamination) can be designed to achieve a one-hour fire rating when evaluated in accordance with the ASTM E 119 or ISO 834 fire test protocol. It was also shown that for the two different FRP reinforcement layers used in this study there were no discernible differences in overall fire and structural performance. These results should open up new market opportunities for FRP reinforced glulam when a one-hour fire rating is required with the FRP applied to the outermost tension face.

Results of these tests suggested that the FRP reinforced glulam could be designed for fire rating in a similar manner as the conventional non-reinforced glulam. Test data from these studies supported the proposed mechanics-based methodology by increasing the depth of the FRP reinforced beam to achieve a one-hour fire rating. As manufacturers in the U.S. seek building code acceptance of proprietary FRP reinforcement systems, this testing will provide the basis for justifying one-hour rated assemblies using FRP reinforced glulam beams.

39-16-2 J König, B Källsner
An easy-to-use model for the design of wooden i-joists in fire

Abstract
A numerical study was conducted with the objective to determine cross-sectional and strength properties of I-shaped joists subjected to charring under ISO 834 standard fire exposure. The outcome of this work should be design parameters in terms of Eurocode 5 (EN 1995-1-2) for I-joists in bending where the tension flange is on the fire exposed side of the joist; these are the time of start of charring, charring depths and modification factors for strength. The I-joists are assumed to be integrated in floor assemblies consisting of joists, linings made of gypsum plasterboard or

wood-based panels, cavities completely filled with batt-type rock or glass fibre insulation, and a decking. Heat transfer analyses were performed using SAFIR. This software permits to study the effect of the lining falling off at specified times that are known from full-scale testing or using the criterion of insufficient penetration length of fasteners into unburnt wood. For the determination of the notional charring depth in the flange and the modification factors of the whole cross-section, a computer program CSTFire, written as a Visual Basic macro embedded in Excel, was developed, using the temperature output from the heat transfer calculations and relative strength and stiffness values given by EN 1995-1-2, i.e. compressive strength, tensile strength and moduli of elasticity in compression and tension. The notional charring depth is calculated such that the notional residual cross-section of the flange remains rectangular and the section modulus of the I-section is unchanged. The effect of various parameters on the notional charring rate is shown, such as charring phases (i.e. a distinction is made whether the I-section is initially unprotected, protected by a lining, or unprotected after failure of the lining), flange dimensions and depth of cross-section. Modification factors for bending and shear strength are shown as functions of the notional charring depth for different charring phases. In order to simplify these relationships, simple expressions are given for increased user-friendliness and code specification.

39-16-3 A Frangi, M Fontana
A design model for timber slabs made of hollow core elements in fire

Introduction

Prefabricated timber assemblies made of hollow core elements are often used for slabs in residential and commercial buildings. Besides the advantage of element prefabrication and a high structural performance, the thermal and acoustic insulation of the timber assemblies can be significantly improved by insulating batts in the cavities and sound absorbers placed behind the perforated acoustic layer.

Timber is a combustible material and thus differs from most other common structural building materials. When sufficient heat is applied to wood, a process of thermal degradation (pyrolysis) takes place producing combustible gases, accompanied by a loss in mass. A charred layer is then formed on the fire-exposed surfaces and the char layer grows in thickness as the fire progresses, reducing the cross-sectional dimensions of the timber member. Because of its low thermal conductivity, the char layer protects the remaining unburned residual cross-section against heat. Because of the small size of the timber members of the hollow core elements, the fire action can lead to very irregular residual cross-sections with charring depths much greater than for heavy timber structures. For fire resistance calculations it is therefore of primary importance to know the development of the charring depth during the fire exposure.

A comprehensive research project on the fire behaviour of timber slabs made of hollow core elements has been recently performed at the ETH Zurich. The objectives of the research project were to enlarge the experimental background of timber slabs in fire and to permit the development of a simplified design model for the fire resistance of timber slabs made of hollow core elements. In addition to a large number of small-scale fire tests, the fire behaviour of the timber slabs was experimental analysed with 2 large-scale fire tests. All fire tests were based on ISO-fire exposure and performed at the Swiss Federal Laboratories for Materials Testing and Research in Dübendorf. The test specimens were manufactured by the Swiss firm Lignatur, Waldstatt. Lignatur elements consist of hollow core elements made of spruce (picea abies) with a mean density of 450 kg/m3. The strength properties of the timber elements correspond to the strength class C24 according to EN 338. Figure 1 shows a typical cross-section of Lignatur timber assemblies made of hollow core elements. The vertical members have a thickness of 33 mm.

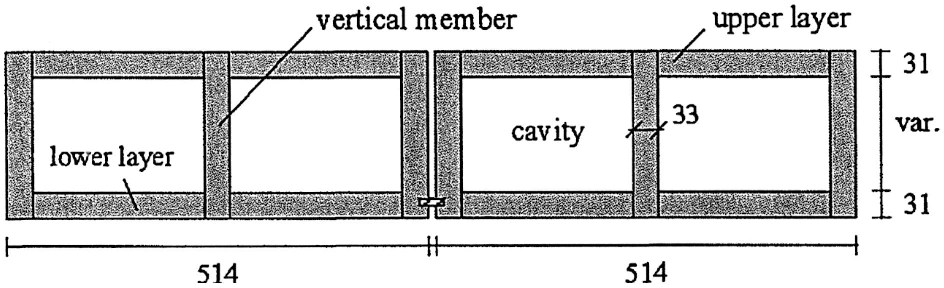


Figure 1. Typical timber slab made of hollow core elements

The paper describes the simplified design model for the calculation of the fire resistance of timber slabs made of hollow core elements. Particular attention is given to the analysis of different strategies used in order to improve the fire behaviour of the timber slabs in fire. The first part of the paper describes the simplified calculation model, in the second part the test results are compared to the simplified calculation model.

Summary and conclusions

The paper presents a simplified design method for the calculation of the fire resistance of timber slabs made of hollow core elements. The simplified design method is based on the reduced cross-section method according to EN 1995-1-2 and takes into account two different charring phases, before and after the fire-exposed layer is completely charred. For simplicity linear relationships between charring depth and time are assumed for each phase. Further it is assumed that the vertical timber members are not exposed to fire on 3 sides. This can be achieved in two different ways:

– the fire-exposed timber layer is so designed that a fire penetration into the cavities is prevented
– the cavities are filled with insulation material, so that after failure of the fire-exposed timber layer charring occurs mainly on the narrow side of the vertical members, while the wide sides are more or less protected by the insulation.

As cavity insulation, rock fibre batts which remain intact up to 1000°C and in place after failure of the fire-exposed timber layer can be used. On the other hand, cavity insulation made of glass fibre batts is not recommended because it melts when exposed directly to fire temperatures, being incapable of protecting the wide sides of the vertical member.

Before the fire-exposed timber layer is completely charred, the timber assembly is exposed to fire only on one side and a more or less homogenous regular one-dimensional charring similar to that of a heavy timber slab can be assumed, as confirmed by fire tests on timber assemblies performed within the framework of the research project. The charring rate measured during the fire tests at the fire-exposed lower layer as well at the vertical members varied between 0.60 and 0.82 mm/min. For the calculation of the charring depth during the first phase a notional charring depth $\beta_{1,n}$= 0.8 mm/min can be assumed giving safe results. This value corresponds to the notional charring rate given in EN 1995-1-2 for solid timber.

Because of the small thickness of the vertical members of the hollow core elements, a superposition of the heat flux from the sides and below occurs during the second phase and increased charring has to be considered in comparison to one-dimensional charring. Thus the notional charring rate $\beta_{2,n}$ during this phase is mainly influenced by the thickness of the vertical members. For the hollow core elements tested with a thickness of the vertical members of 33 mm a notional charring rate $\beta_{2,n}$ = 1.6 mm/min. can be assumed. This is confirmed by fire tests conducted within the framework of this research project and other fire tests. Although the insulation material is able to protect the wide sides of the vertical members, the fire tests showed that because of the small size of the vertical members the temperatures measured in the vertical members are higher than in comparison to heavy timber cross-sections. For the calculation of the factor d_0, which takes into account the temperature-dependent reduction in strength and stiffness in the heat affected zones of the vertical members, an advanced calculation model has been used. The cross-section of the timber assembly has been divided into *n* finite elements with different stiffness and strength properties as a function of the measured temperature Θ_i *(t).*The reduction of the *E*-modulus and bending strength has been assumed according to EN 1995-12. Under assumption of a factor d_0 = 20 mm a good agreement between the advanced and the simplified calculation model was observed.

The global behaviour of the timber slabs made of hollow core elements was analysed with two fire tests on slabs performed in ENIPA's horizontal furnace (3.0 x 4.85 m). The fire tests showed a fire resistance of more than 60 minutes and 90 minutes respectively. When verifying the simplified design method, a good agreement between fire test results and the simplified design method was observed.

40-16-1 J König, J Schmid
Bonded timber deck plates in fire

Abstract

Laminated deck plates, made of edgewise or flat wise laminations, are increasingly used as structural elements in housing and commercial buildings, both in floors and walls. In structural fire design, EN 1995-1-2 gives a simplified method for the calculation of the mechanical resistance of structural timber members. Apart from charring, the effect of elevated temperature is taken into account by assuming a zero-strength layer below the charring depth of thickness 7 mm, reached after the first 20 minutes of the fire exposure. For rectangular cross-sections this model gives reasonably good agreement with advanced calculations. EN 1995-1-2 also permits the application of this model to timber slabs exposed on one side. This pa-

per gives some results from advanced calculations – using the thermal and thermo-mechanical properties of wood given by EN 1995-1-2 – of the bending resistance of both homogenous and cross-laminated timber slabs exposed to fire. It is shown that the depth of the zero-strength layer should be increased considerably above the value of 7 mm given by EN 1995-1-2 for beams and columns. Also, there is a significant effect of the state of stress of the fire exposed side of the slab, that is when the fire exposed side is in compression, the reduction of bending strength is greater than in the opposite case.

Conclusions and further research needs
For the design of timber beams and columns, EN 1995-1-2 gives two alternative methods. The effective cross-section method introduces a zero strength layer of uniform depth of seven millimeters to take into account the reduction of strength and stiffness properties of the member. The results of simulations of fire exposed homogeneous and cross-laminated timber plates show that the application of the effective cross-section method would require zero-strength layer depths that are dependent on plate depth, the composition of layers and the state of stress on the fire exposed side of the plate. A considerable advantage of the method would disappear. After simplification, the zero-strength layer depths can be given as linear functions of the plate depth.

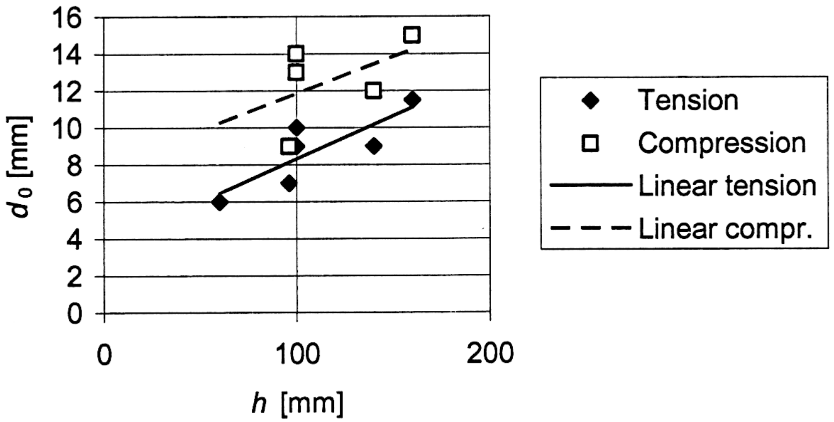


Figure – Zero-strength layer depth vs. plate depth

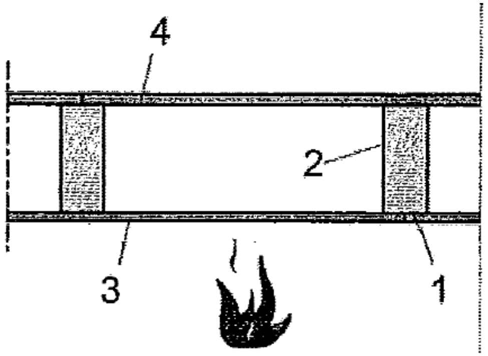
These results are preliminary. More timber plate configurations need to be investigated and it must be checked if these zero-strength layer depths also apply to bending stiffness when these elements are used as wall. Also, the effect of fire protective claddings needs to be investigated. It can be expected that, in some cases, considerably smaller zero-strength depths would apply.

Since the accumulation of moisture has a significant effect on strength and stiffness at elevated temperature, it should also be investigated if there is any effect of the adhesive on the transport of vapour in the plate.

40-16-2 A Frangi, C Erchinger
Design of timber frame floor assemblies in fire

Introduction
Timber frame floor assemblies are typical structural elements used in timber engineering. The floor assemblies consist of solid timber beams with claddings of gypsum plasterboards, wood based panels or combinations of these layers. The cavities may be filled with insulation made of rock, glass or wood fibre. The fire behaviour of timber frame floor assemblies is characterised by different charring phases. After the fire exposed claddings fall off, the timber beams are exposed directly to high temperatures leading to an increased charring rate in comparison to timber elements that are initially unprotected from fire exposure. Thus the fire performance of timber frame floor assemblies is influenced by the protection provided by the claddings. Further the size of the timber beams and the protection provided by the cavity insulation play an important role on the fire performance of the floor assemblies.

Design models of timber structures in fire usually take into account the loss in cross-section due to charring of wood and the temperature dependent reduction of strength and stiffness of the unburned residual cross-section. For timber frame wall and floor assemblies whose cavities are completely filled with insulation, Annex C of EN 1995-1-2 provides a simplified calculation model based on the reduced properties method. The design model based on fire tests performed on light timber frame assemblies with studs and joists with small cross-sections. For timber frame wall and floor assemblies with void cavities only very little information is available in Annex D of EN 1995-1-2.



Definition:
1. Narrow side of timber beam exposed to fire
2. Wide side of timber beam facing the cavity
3. Fire protective cladding (lining) on exposed side of timber frame floor assembly
4. Fire protective cladding (lining) on side of timber frame floor assembly not exposed to fire

Figure 1 Timber frame floor assemblies with void cavities

A comprehensive research project on the fire behaviour of timber frame floor assemblies is currently ongoing at the ETH Zurich in cooperation with the Swiss Federal Laboratories for Materials Testing and Research (EMPA). The aim of the research project is the development of a design model for the fire resistance of timber frame floor assemblies with and without cavity insulation, primarily based on the reduced cross-section method. In addition to small-scale fire tests on protective claddings, the fire behaviour of timber frame floor assemblies was experimental analysed with a large-scale fire test. All fire tests were based on ISO-fire exposure and performed at the EMPA in Dübendorf.

The paper analyses the fire behaviour of timber frame floor assemblies with void cavities (see figure 1). The results of FE-thermal simulations in combination with results of fire tests permit to develop a simplified charring min model to be included in EN 1995-1-2. In the first part of the paper the results of FE-thermal simulations are described. The second part of the paper presents the simplified charring model for timber frame floor assemblies with void cavities.

Summary and conclusions

Design models of timber structures in fire usually take into account the loss in cross-section due to charrinc, of wood and the temperature dependent reduction of strength and stiffness of the unburned residual cross-section. The paper presented a simplified charring model for timber frame floor assemblies with void cavities. The charring model was developed following the method and terminology given in EN 1995-1-2 so that it is easy to use and can be included in Annex D of EN 1995-1-2. The simplified model based on extensive 1~E-simulations and takes into account for the post-protection phase (i.e. after failure of the fire protective claddings) the influence of high temperatures as well as the heat flux superposition on the charring rate of the timber beams. The FE-model for the thermal analysis was verified by fire tests on protected specimens exposed to one-dimensional charring.

The simplified charring model was developed for the case that the failure of the claddings occurs at the same time of start of charring t_{ch} (typically for protective claddings made of wood-based panels or wood panelling as well as for gypsum plasterboards type A or H). For the case $t_f > t_{ch}$ (typically for protective claddings made of gypsum plasterboards type F) the simplified charring model should be modified in order to include charring, of the timber beams until failure of the protective claddings. For the calculation of the mechanical resistance of the timber beams of the floor assemblies with void cavities, also the temperature dependent reduction of strength and stiffness of the unburned residual cross-section shall be taken into account, e.g. by modification factors for the material properties or the use of an effective reduced residual cross-section. The modification factors or the zero-strength layer can be calculated combining the results of the thermal analysis with the structural analysis of the timber beams in fire.

41-16-1 J König, J Norén, M Sterley
Effect of adhesives on finger joint performance in fire

Abstract

In an experimental investigation, fire tests were performed on small-sized glued laminated timber beams in bending. The beams consisted of three lamellae with the lamella on the tension side being finger jointed in the middle, thus forming a weak link to initiate failure both at ambient temperature and in fire. Four structural adhesives were tested: one phenolic resorcinol formaldehyde adhesive (PRF) as the reference adhesive representing a traditional adhesive with excellent fire performance, one melamine urea formaldehyde adhesive (MUF) and two polyurethane adhesives (PUR), the latter representing novel adhesives, which are today commonly used in Europe in the production of load-bearing engineered wood products.. The resistance of the beams at ambient temperature was determined

prior to the fire tests. The applied loads during the fire tests corresponded to load ratios from about 20 to 40 % of ambient resistance. The fire tests showed that the moment resistance of the beams finger-jointed with MUF and PUR adhesives was 70 to 80 % of the moment resistance of the beams finger-jointed with PRF. Since these adhesives offer advantages in terms of increased production capacity, cost-efficiency and environmental aspects, it is important that these adhesives are not banned from the marked place due to too rigorous design rules. It is proposed to modify the design of bonded timber connections, taking into account the thermo-mechanical properties of the bond by introducing modification factors for e.g. finger joint strength, and to create a new classification of structural adhesives with respect to their fire performance.

Conclusions and further research needs
The fire tests showed that there is a considerable loss of bending resistance of the test beams. Within the interval of time considered in this investigation, for load ratios between 0,2 and 0,4 of the PRF beams, the moment resistance of beams with finger joints bonded with structural PUR and MUF adhesives was between 70 and 80 % of the moment of PRF adhesive. There is no apparent influence of the unprotected and protected beams.

Since the number of tests and adhesives was small, test series should be performed with a larger number of specimens in order to achieve greater statistical reliability. Further, a possible influence of test beam configurations should be investigated, e.g. larger glued laminated beams and composite products such as I-joists should be considered.

The tests reported here were designed with the purpose of obtaining finger joint failure both at ambient temperature and in the fire tests. In commercial grades, however, failure would often be caused by knots or other defects. Since the random occurrence of weak zones – i.e. finger joints, knots or other defects – has an effect on the resistance of the beams, it should be investigated to what extend fire safety is influenced by the performance of various adhesives. In glued laminated beams with lamellae with a large knot area ratio these defects may be the dominating cause of failure, whereas high quality lamellae will cause more finger joint failures.

42-16-1 S Winter, W Meyn
Advanced calculation method for the fire resistance of timber framed walls

Introduction
In the Meyn dissertation, a model was developed for determining two- and three-dimensional temperature distributions in the stud and cavity region of timber framed walls. The entire model is designed for calculation using an FEM program. Physical methods can also be used in a model based on manual calculation.

The model values, or the physical properties of the material defined in the model, were implemented in the ANSYS FEM program. The thermal verification of failure times is possible for any kind of wall construction by the use of solid wood, wood-based panels, gypsum plasterboard, fire protection gypsum plasterboard and mineral fibre insulation made of glass or rock wool. Materials, not listed here, for which thermal material data is still available, can also be included in the model. The ordering, thickness, density and moisture content of the materials are all freely definable. It is also possible to have a partially-insulated wall construction or one without any insulation.

To make the simulation of the thermal behaviour of timber framed walls as realistic as possible, crack formation in panel materials, opening of joints and changes to material properties caused by moisture transport was included in the complex model. These physical effects are triggered by specific temperatures in the FEM model. They are described in detail in the 3 chapters below.

42-16-2 C Erchinger, A Frangi, M Fontana
Fire design model for multiple shear steel-to-timber dowelled connections

Introduction
Dowelled connections are made of circular cylindrical steel dowels, fitting tightly in predrilled holes and used for transferring loads perpendicular to the dowel axis. Multiple shear steel-to-timber connections with slotted-in steel plates and steel dowels (see Fig. 1) show a high load-carrying capacity and a ductile failure mode (plastic deformation of the dowels) at ambient temperature if minimum spacing and distance requirements of the

dowels are respected, for example according to EN 1995-1-1. The load-carrying capacity of the connections depends primarily on the embedment strength of the timber members and the yield moment of the dowels. Due to the protection of the slotted-in steel plates against fire provided by the timber side members (see Fig. 1), a high fire resistance may be achieved. In order to accurately predict the structural performance in fire, knowledge on the temperature distribution in the cross-section as well as the influence of steel elements (slotted-in steel plates and steel dowels) on the charring of the timber members is required.

This is challenging and complex due to the influence of several parameters like dowel diameter, geometry of the connection, different failure modes, different thermal properties of timber and steel as well as the thermal interaction between timber members and steel elements. Thus, only limited work has been carried out on the fire behaviour of steel-to-timber connections and current standards do not contain consistent calculation models for the fire design of steel-to-timber connections taking into account the influences of the different parameters. The reduced load method given in EN 1995-1-2 is only valid for laterally loaded symmetrical three-member connections (two shear planes, one steel plate in the middle) under ISO-fire exposure. The relative load-carrying capacity versus time given as a one-parameter exponential model is based on a still limited number of fire tests carried out on timber connections with bolts and nails. For multiple shear steel-to-timber connections with two or three slotted-in steel plates no design models in the fire situation exist so far.

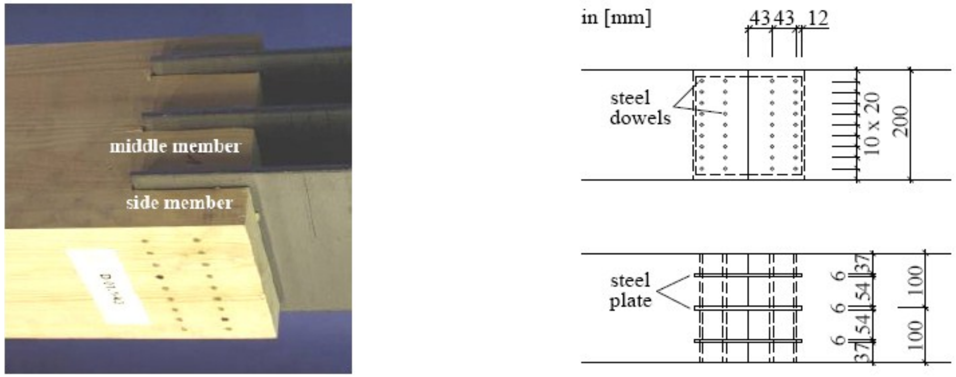


Figure 1 Typical multiple shear steel-to-timber connection with three steel plates (left) and example of the geometry of tested connection D1.1 (right)

A comprehensive research project on the fire behaviour of multiple shear steel-to-timber connections with two or three slotted-in steel plates and steel dowels has recently been performed at ETH Zurich. The objective of the research project was the development of a fire design model for multiple shear steel-to-timber connections with slotted-in steel plates and steel dowels. In addition to 25 tensile tests at ambient temperature to determine the load-carrying capacity, the fire behaviour of the connections was experimentally analysed with 18 fire tests under constant tensile load. All fire tests were performed under ISO-fire exposure on the horizontal furnace at the Swiss Federal Laboratories for Materials Testing and Research (EMPA) in Dubendorf. The paper first describes the main results of extensive experimental and numerical analyses on the fire behaviour of multiple shear steel-to-timber connections with slotted-in steel plates and steel dowels. Particular attention is given to the analysis of the influence of the steel elements on the charring of the timber members. Then, the design model for multiple shear steel-to-timber dowelled connections with slotted-in steel plates in fire is presented and compared to fire tests.

Conclusions

The load-carrying capacity of timber structures is often limited by the resistance of the connections. Thus, highly efficient connections as multiple shear steel-to-timber connections with slotted-in steel plates and steel dowels are needed for an efficient design. Connections with slotted-in steel plates achieve a high fire resistance because the steel plates are protected by the timber side members. The results of an extensive experimental analysis showed that shear steel-to-timber dowelled connections with two or three slotted in steel plates designed for ambient temperature with a width of the timber members of 200 mm reached a fire resistance of at least 30 minutes. A reduction of the load level from 30% to 15 or 7.5% of the average load-carrying capacity measured at ambient temperature did not lead to a significant increase of the fire resistance. By increasing the thickness of the side members as well as the end distance of the dowels by 40 mm the connections reached a fire resistance of more than 60 minutes. Connections protected by timber boards or gypsum plasterboards can reach a fire resistance of 60 minutes or more depending on the thickness and type of protection. Thus, from a fire design point of view the increase of the side member thickness or the protection by boards are efficient in order to increase the fire resistance significantly.

The load-carrying capacity of multiple shear steel-to-timber dowelled connections with slotted-in steel plates primarily depends on the charring of the timber members and the resulting temperature distribution in the residual cross-section. An extensive numerical analysis showed that slotted-in steel plates and steel dowels strongly influence the charring of the timber members and the temperature distribution in the residual cross-section. Based on the results of the experimental and numerical analyses, an analytical design model for the calculation of the load-carrying capacity of multiple shear steel-t-timber dowelled connections with slotted-in steel plates subjected to tension was developed. The design model is in analogy with the Reduced cross-section method according to EN 1995-1-2. The model takes into account the influence of the steel elements on charring and temperature distribution in the cross-section and allows accurate fire design of multiple shear steel-to-timber dowelled connections with slotted-in steel plates with different geometries for a fire resistance up to 60 minutes.

42-16-3 M Fragiacomo, A Menis, P Moss, A Buchanan, I Clemente Comparison between the conductive model of Eurocode 5 and the temperature distribution within a timber cross-section exposed to fire

Introduction

Timber is a natural and sustainable resource which is being used more and more as structural material in buildings. Since timber is combustible, the fire design needs special attention. When a timber structural member is exposed to fire, physical, thermal and mechanical degradation phenomena occur in the material, leading to a very complex behaviour. Simplified design methods suggested by current codes of practice such as the Eurocode 5 Part 1-2 assume strength capacity under fire to be dependent upon the residual cross-section, which is evaluated by computing the thickness of the charring layer lost from the original unburned section during the time of exposure to fire. In such a simplified design approaches, the charring rate is defined as the rate of movement of the 300ºC isotherm in the wood. Information on the temperature distribution within a timber cross-section during a fire is therefore crucial to compute the charring rate for different wood species and wood-based materials such as sawn timber, glue-laminated timber and laminated veneer lumber (LVL). The temperature distribution is also required for advanced numerical models where the structural fire resistance is calculated by taking into account the degradation of the mechanical properties (strength and modulus of elasticity) of timber with the temperature.

Analytical models based on experimental results for the temperature profiles of beams and slabs exposed to the standard ISO fire curve or different fire conditions were proposed by Frangi and Fontana and Janssens. The temperature profiles can also be calculated using numerical models calibrated on experimental test results . A rigorous modelling of the heat conduction process within timber members exposed to fire should consider all complex phenomena taking place in the material, including the mass transport of water vapour after evaporation, the charring of wood, the crack formation and the physical properties of the char layer. Due to the complexity of such rigorous models, reference to simplified conventional heat transfer models is usually made, where the phenomena listed above are implicitly accounted for in the thermal and physical properties of the material. Until now, most of the numerical analyses were performed using thermal parameters derived by calibration of the models on experimental results. Different proposals can be found in literature for the variation of thermal and mechanical parameters with temperature, showing little agreement among them. Such relationships are affected by several variables such as the shape of the fire curve, the wood species, the type of wood–based material, the conditions of the surrounding environment, etc. It is therefore important to perform further research into the thermal and physical parameters of timber when exposed to fire in order to improve the accuracy of the temperature prediction and, consequently, the accuracy of the fire resistance of timber members.

The paper investigates the temperature distribution in cross-sections made from spruce timber and radiata pine laminated veneer lumber (LVL) exposed to fire. Small (146×60 mm) and larger (300×105 mm and 360×133 mm) LVL members were tested in New Zealand under two-dimensional fire exposure at the University of Canterbury and at BRANZ, respectively. Experimental results of 2D and 1D fire exposures were then compared with numerical values obtained by implementing a model of the thermal conduction process in the Abaqus FE code. Different proposals among those found in literature for the variation of thermal and physical properties with temperature were compared. A new proposal is made in this paper which leads to a more accurate prediction of temperature distribution, particularly for larger sections subjected to 2D fire exposures.

Concluding remarks

This paper presents the outcomes of experimental tests conducted on small and larger laminated veneer lumber (LVL) cross-sections subjected to two-dimensional fire exposure.

A 2D FE conductive model was implemented in Abaqus and validated on the experimental tests and numerical analyses carried out on timber sections subjected to one-dimensional fire exposure. Different proposals for the thermo-physical parameters were investigated. The Eurocode 5 proposal was found to predict the temperature distribution of small cross sections subjected to 1D fire exposure with excellent accuracy. For small LVL cross sections subjected to 2D fire exposure, however, the approximation was lower but still acceptable, whereas the 2D heating process of the larger cross-sections was predicted with a delayed temperature rise, particularly in the interior fibres. A new proposal for the conductive model was therefore made. This proposal assumes the same variations of density and specific heat as recommended in Eurocode 5 Part 1-2, but considers a variation of the conductivity according to Frangi up to 550°C, and then slightly increased values with respect to Eurocode 5 in the range 550°C to 1200°C. The new proposal leads to better predictions for the larger cross-sections and slightly worse, but still acceptable, predictions for the small LVL cross-section subjected to 2D fire exposure, and for the timber section subjected to 1D fire exposure, where the heating process of the inner fibres is slightly anticipated. It must be pointed out, however, that the small cross-section has less technical relevance than the other cross-sections as it is a narrow member with very low inherent fire resistance, probably needing additional passive protection if a given fire rating has to be achieved.

In order to generalize the new proposals, further experimental-numerical comparisons should be carried out on large members made from glulam or different wood-based materials. The numerical model will then be used for coupled thermal-stress analyses aimed at investigating the fire resistance of timber beams, timber-concrete composite structures, and connections.

43-16-1 J Schmid, J König
Light timber frame construction with solid timber members – Application of the reduced cross-section method

Abstract

In timber members exposed to fire a zone of about 35 to 40 mm depth below to the char layer, although unburned, is heated above ambient temperature. Due to the elevated temperature this zone, strength properties and the modulus of elasticity of the residual cross-section must be reduced. Two methods, known as reduced properties method and reduced cross-section method, respectively, are used in practice. In the first one the strength and stiffness properties of the cross-section, e.g. bending strength or modulus of elasticity, are multiplied by modification factors for fire, while in the second one, the residual cross-section is reduced by a so-called zero-strength layer, whereas the strength and stiffness properties remain unreduced.

For the calculation of the mechanical resistance of wall and floor assemblies in fire consisting of light timber frame members with rectangular cross-sections of solid timber and cavities filled with insulation, EN 1995-1-2 gives a design model using the reduced properties method. In order to simplify the calculation the original data were re-evaluated and expressions for zero-strength layers were derived to allow the use of the reduced cross-section method. For bending, the zero-strength layers were calculated to achieve the best fit of bending resistance in the range of load ratios between 0,2 and 0,4. Only for load ratios smaller than 0,2 the results are slightly non-conservative. For axially loaded members, the zero-strength layers were determined to give the same or lower bending stiffness than according to the reduced properties method. The axial resistance of studs, however, calculated using the method of EN 1995-1-1 with properties relevant for the fire situation, is somewhat greater when the reduced cross-section method is used.

Conclusions

It has been shown that the reduced properties method for the calculation of the mechan of timber frame members (studs or joists) in wall and floor assemblies can be replaced 1• cross-section method using zero-strength layers instead of modification factors for simplify design work. For members in bending, the bending resistance according to the 1 section agrees well with results from the reduced properties method since the zero-streng determined to achieve the best fit for load ratios in the range between 0,2 and 0,4 important in practice. Only for load ratios smaller than 0,2 the results are slightly non For axially loaded members the calculated resistance is somewhat greater according t cross-section method when the method for axially loaded members given in EN 1995-1 however with properties relevant for the fire situation. Since the zero-strength layer is eq modification factors for fire, the difference between the calculated axial resistances is the different weighting of parameters (such as slenderness ratios, relative slenderness rat in the expressions given in EN 1995-1-1.

43-16-2 J Schmid, J König
Fire exposed cross-laminated timber - modelling and tests

Abstract

This paper presents a simple design model using the effective cross-section method for the structural fire design of CLT, i.e. the determination of the mechanical resistance with respect to bending (floors).

Performing advanced calculations for a large number of lay-ups of various lamination thicknesses, using the thermal and thermo-mechanical properties of wood, charring depths and the reduction of bending resistance of CLT were determined as functions of time of fire exposure. From these results zero-strength layers were derived to be used in the design model using an effective residual cross-section for the determination of mechanical resistance.

The model also takes into account different temperature gradients in the CLT in order to include the effect of slower heating rate when the CLT is protected by insulation and/or gypsum plasterboard. The paper also gives results from fire-tests of CLT in bending using beam strips cut from CLT with adequate side protection in order to achieve one-dimensional heat transfer. Reference tests at ambient temperature were performed to predict the moment resistance of the beams being tested in fire.

Conclusions

It has been shown that the complex performance of CLT exposed to fire can be described by advanced computer simulations, using the thermal and thermo-mechanical properties of wood given by EN 1995-1-2 [1] and that the simulation results are verified by test results from fire tests. In order to present a user-friendly easy-to-use design model for members in bending, the concept of the reduced cross-section method given in [1] was adopted and zero-strength layers determined for consideration of the reduced strength and stiffness properties at elevated temperatures. The simplified model gives reliable results, while the adoption of the zerostrength layer equal to 7 mm, as given in EN 1995-1-2 [1] for beams and columns, normally gives non-conservative results.

43-16-3 J O'Neill, D Carradine, R Dhakal, P J Moss, A H Buchanan, M Fragiacomo
Timber-Concrete Composite Floors in fire

Introduction

Timber-concrete composite floors are a combination of timber joists and concrete topping, creating a flooring system best utilising the advantages each material has to offer. Timber is used as the main tensile load bearing material due to its high strength-to-weight ratio, while concrete is used in floor slabs for its advantages in stiffness and acoustic separation. The strength of the system is dependent on the connection between timber and concrete, thus the connection must be strong, stiff, and economical to manufacture, to ensure that the flooring system is economically viable. The benefits in aesthetics, sustainability and economical savings due to fast erection time will undoubtedly be a significant factor to their widespread use in the future. Timber-concrete composite structures are not a new technology, and arose in Europe in the early twentieth century as a means of strengthening existing timber floors by the addition of a concrete slab. Due to the many advantages they possess over traditional timber floors, they are now being used in new construction . This is currently under investigation in many parts of the world such as Sweden, the United States [4], Germany, Switzerland and New Zealand.

There are many different types of composite flooring design, the main two categories being either solid timber slab type designs or beam type designs.

Beam type designs consist of timber beams (either sawn timber, glulam or LVL) being used as floor joists, upon which a solid membrane (usually a plywood sheathing or steel deck) is fixed and a concrete slab is cast above. The forms of connections between the timber and concrete are extremely varied, some of which are glued, non-glued, and notched connections [8]. Glued connections consist of a form of steel reinforcement (rebars, punched metal plates, steel lattices) which is glued into the timber members and continues out into the concrete slab. Non-glued connections can consist of screws partially screwed into pre-drilled holes in the timber, inclined steel bars driven into tight holes, or shear studs screwed into the timber member. Notched connections consist of a notch cut out of the timber member which the concrete is cast into, and a stud can be incorporated in the notch for better performance. A number of these different types of connections have been investigated by Ceccotti, Lukaszewska et al. and Yeoh et al.

Solid timber slab designs are generally composed of a solid timber decking from nailed timber planks with a concrete slab cast directly on top. Slab type floors generally utilise a grooved connection, the concrete is cast into grooves or trenches in the top of the timber 2 decking which allows a large shear area of concrete to timber to be utilised, resulting in a very stiff and complete composite connection. Kuhlmann and Michelfelder have conducted extensive research on the strength and stiffness of grooved timber slabs.

The type of composite floor under study was a semi-prefabricated beam type system comprising of "M" panels that were built with laminated veneer lumber beams sheathed in plywood as permanent formwork for the concrete. The plywood had holes cut to accommodate the shear connection between the beams and the concrete slab. Both notched connections and toothed metal plates were used in this research. The panels can be prefabricated off-site then transported to site and craned into position, allowing the concrete slab to be cast in-situ.

Conclusions

A design method has been proposed for timber-concrete composite floor systems with different types of connections. With the input of floor geometry, material properties, loading conditions and corresponding safety factors, the design method outputs an expected fire resistance time for the floor to reach structural failure. It was used to derive a number of resistance span tables for varying spans of these timber-concrete composite floor systems with regard to section sizes available and different combinations of live and superimposed dead loads. Two full-scale floors were fire tested. The reduction in section size of the timber beams due to charring governed the failure of the floors. Due to the composite action achieved by the connections, the floor units were able to withstand prolonged exposure to the test fire, well exceeding one hour. The effect of reduced section properties due to elevated timber temperatures was found to have a small impact on the overall performance of the test floors. Test data and visual observations aided in the development of an analytical model that is able to predict the expected fire resistance of these floors, taking into account some major time dependent variable properties, loading conditions, material properties and floor geometries.

In conclusion this research has shown that the fire performance of unprotected timber-concrete composite floor is excellent. A large degree of safety is possible without risking structural collapse, and this research should serve as a guideline to the expected performance. Further means of fire protection to these floors such as passive protection (fire rated suspended ceilings, gypsum plasterboard encasement) or active protection (sprinkler systems) will serve to further increase the fire resistance of these floors.