CIB-W18/19-16-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SIMULATION OF FIRE IN TESTS OF AXIALLY LOADED WOOD WALL STUDS

by

J König
Swedish Institute for Wood Technology Research
Sweden

MEETING NINETEEN
FLORENCE
ITALY
SEPTEMBER 1986

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 excentricity. A test procedure is proposed for the determination of fire resistance of load-carrying wood-stud walls.

1. Introduction

In Sweden and several other countries the building regulations stipulate that the external walls in small one-family houses should resist fire for 30 minutes. In the case of loadbearing external walls the wall must be able to carry the design load during this time-period, and the same requirement has also to be satisfied by the components which form supports and stiffen the wall.

The majority of external walls used in Sweden are wood-stud walls which are built up of timber studs with cladding on both sides. During recent years a number of light-weight studs have been introduced. The most common one is a stud where the flanges are built up of wood and the web is built up of fibre board.

In Sweden today it is most usual to determine the fire resistance of load-bearing wood-stud walls by performing full scale fire-tests. A real wall exposed to fire is loaded by both axial and transversal loads. In order to facilitate testing the Swedish code gives a simple rule which says that the effect of transverse loading can be simulated by an excentricity of the axial load /1/. For small houses this excentricity has to be at least 20 mm. The code implies, though it does not expressly state, that the location of the axial load remains the same during the whole fire test. As can be expected, it is often not possible to satisfy the requirements for fire resistance under these conditions.

In order to elucidate the mechanical behaviour of axially loaded wood studs exposed to fire, a series of tests were conducted. The charring of the timber was replaced by planing on one side of the stud. The three other faces of the stud were assumed to be protected from fire by mineral wool and the sheating.

The aim of the fire simulation was not to consider the thermal effects of fire on the mechanical properties of the wood material but rather to gain an understanding of the mechanical behaviour of a wood stud with a view to developing a rational method of calculation. The thermal effects can be determined separately.

A complete description of this study will be found in /2/.

2. Performed tests

Totally 8 tests were conducted. Six of the specimens were formed by 2,4 m long 45 x 120 mm timber studs of the Swedish grade "Ö", which has a characteristic bending and compression strength of 15 N/mm2. Two of the specimens were 200 mm deep light-weight studs of the same length with 45 x 45 mm timber flanges and 6 mm thick fibreboard webs. For the solid studs short pieces of 45 x 120 mm timber were used as sole and head plates (specimens 1 - 6). In the case of light-weight studs (specimens 7 and 8) special light-weight members were used as sole and head plates.

In order to study the effect of different support conditions cellular rubber air-tightening profiles were attached to the sole and head plates. In half of the tests the lower support plate was given an inclination of 3,5 % in order to simulate the effect of the deflection of the roof truss or of an uneven foundation as site concrete. This inclination is larger than can normally be anticipated. The support conditions of the specimens are shown in Fig. 1.

The specimens were placed in a test rig with stiff, non-rotating, support plates, thus allowing the specimen to deform dependent on the support conditions. By using three load cells under the lower support plate it was possible to determine the location of the axial load.

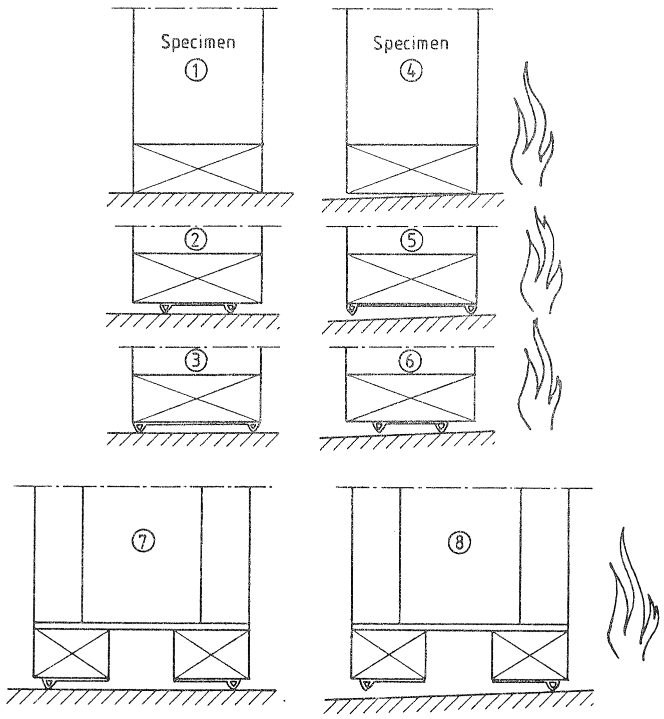


Figure 1. Support conditions. The support plate at the upper support is horizontal for all specimens.

In order to determine the bending stiffness the specimens were loaded with two transverse point loads and the displacements measured as shown in Fig. 2.

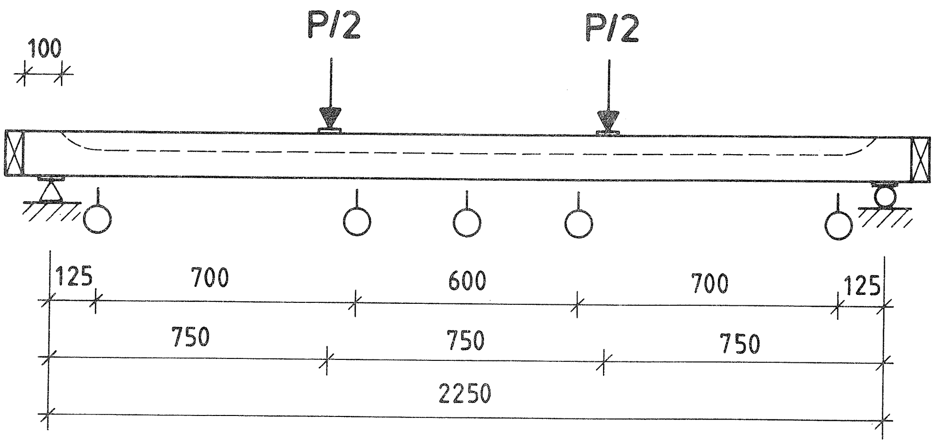


Figure 2. Application of loads and gauges for determination of bending stiffness.

The following procedure was adopted for all the tests. First the specimen was loaded with the transversal forces until a mid-span-deflection of 5 mm was reached after about two minutes. Immediately after this the specimen was unloaded and placed in the test rig for axial load. Buckling in the weak direction was prevented by bracing. The axial load was applied until the maximum load N_{max} was reached after not less than four minutes. The maximum load was held constant for five minutes and the stud was unloaded again. The load-values of the three load-cells and the mid-deflection were registred in steps of 1 kN and, when the maximum N_{max} was held constant, every minute.

After unloading a 5 or 10 mm thick layer of the stud was removed by planing, except for the regions close to the ends. This is shown on the broken curve in Fig. 2. The procedure of loading and planing was continued until the stage of collapse was reached. Normally the ultimate load N_n was lower than the maximum load N_{max}. The test results for the collapse stage and the stage before are given in Table 1.

The maximum load N_{max} was chosen slightly lower than the allowable load. This was calculated with respect to buckling in the case of specimens 1 to 6 and with respect to compression perpendicular to the grain in the sole plate in case of specimens 7 and 8.

Typical curves for axial load versus mid-deflection are shown for all stages of specimen 4 in Fig. 3 and for the collapse stage of specimens 1 - 6 in Fig. 4. It can be seen that the deflections, before ultimate load is reached, are largest in specimens with 70 mm wide rubber profiles and smallest in specimens without rubber profiles. In each of these pairs the deflection was larger when the lower support had an inclination.

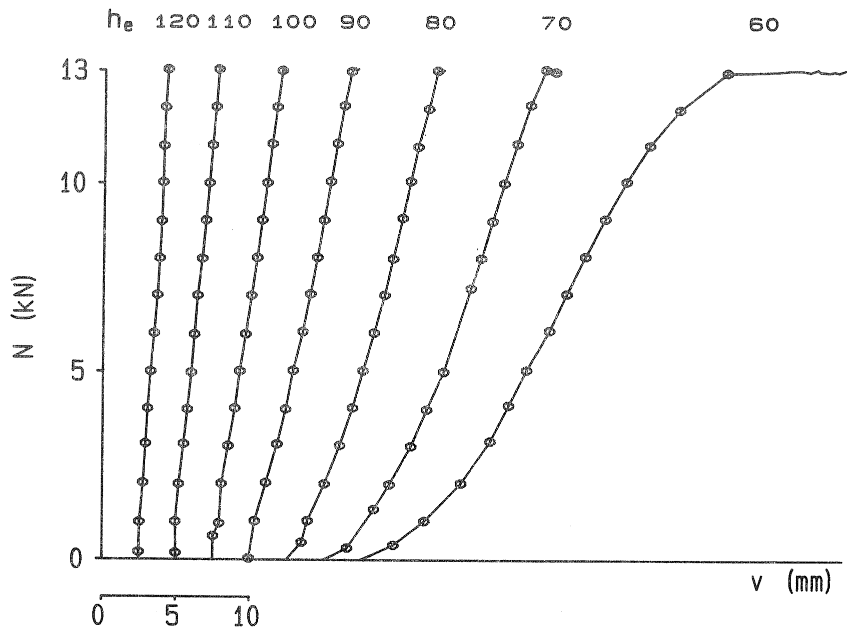


Figure 3. Axial load versus mid-deflection for all stages of specimen 4.

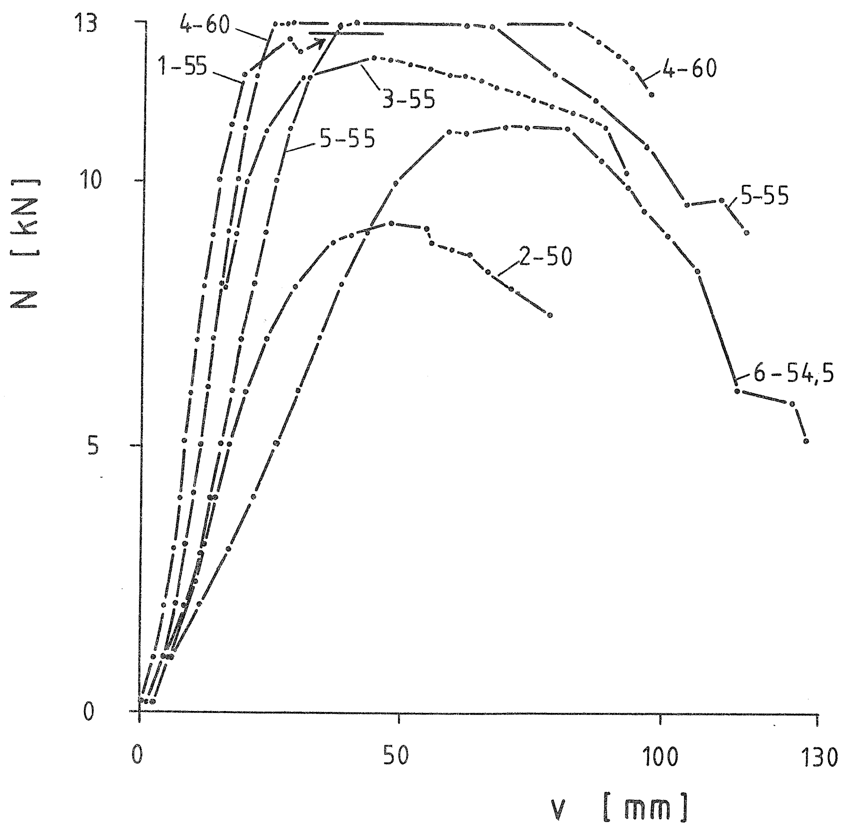


Figure 4. Axial load versus mid-deflection for collapse stage of specimens 1 - 6.

Typical curves for the location of the axial load and the geometrical centre of gravity of the mid-section of the stud, see the explanations in Fig. 5, are shown in Fig. 6. The curves for the other specimens are not shown here. The location of the axial load in the ultimate section is about the same for specimens 1 - 6, see Table 1, whereas its location is variable for the initial full section, depending on the stiffness and geometrical conditions at the supports. The specimens 7 and 8 also show a similar behaviour.

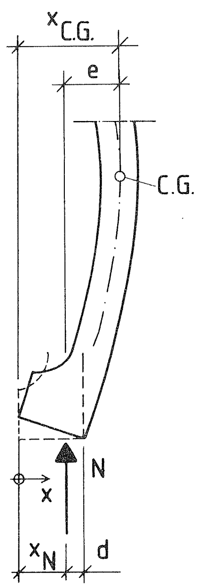
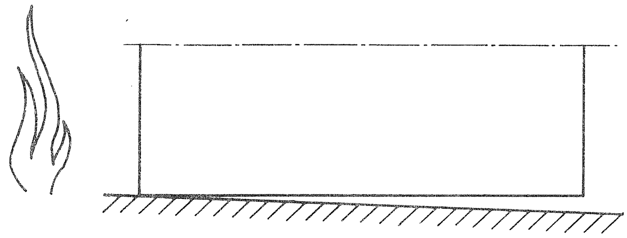


Figure 5. Location of the axial load and the geometrical centre of gravity of the mid-section.



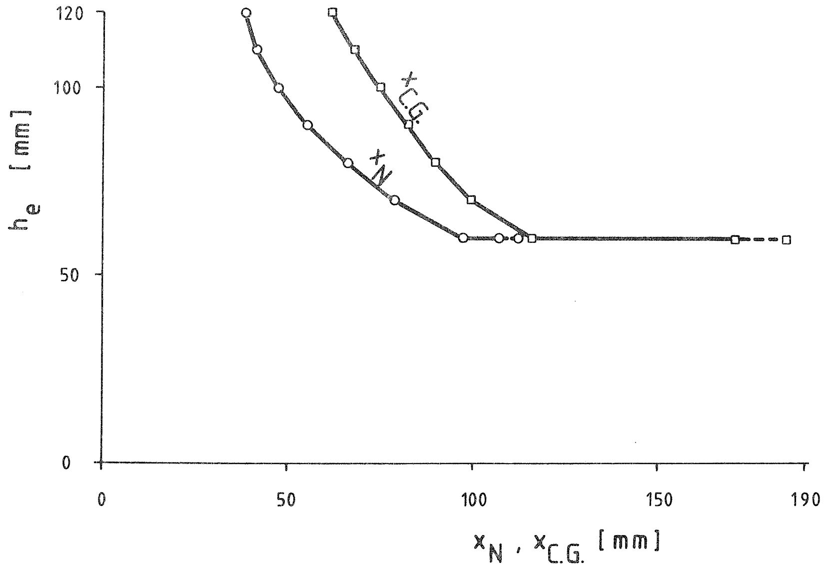


Figure 6. Location of the axial load and the geometrical centre of gravity of the mid-section at different stages of specimen 4.

The displacement of the axial load during the test is caused by the rotation of the stud ends. Thus the load excentricity, that is the distance between the two curves in Fig. 6, becomes smaller than is normally assumed in the case of column buckling with fixed end-hinges.

3. Comparison of test results and theoretical calculations

In the CIB Structural Timber Design Code /3/, section 5.1.7, a buckling curve is given for calculation of axial load-bearing capacity of columns. Using the bending stiffnesses which were determined by means of the tests with transverse loads, the theoretical load-bearing capacity N_{cd} was calculated for the specimens 1 to 6, see Table 2. In this table the slenderness ratios λ are also given.

In these calculations the modulus of elasticity used was the mean value of all stages of the specimen. The gauge length was 2.0 m, see Fig. 2. As the specimens had no obvious imperfections, these were chosen to equal 1/1000 of the length. The CIB buckling curve gives results which are considerably on the safe side compared with the test results. These results can be expected since the buckling curve was derived assuming fixed end hinges.

4. 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 choosen 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 registred. 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

$$\frac{N}{N_d} + \frac{M}{M_d} \leqslant k$$

where N_d is the characteristic axial load bearing capacity obtained from step 1 and M_d 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.

TABLE 1. Test results at and before collapse.

Specimen	Depth at collapse stage h_e mm	Depth before collapse stage[a)] h_e mm	Maximum load N_{max} kN	Ultimate load N_u kN	Time N_u was held	Location of load N_u d mm
1	55	60	13,0	12,80	0	18,8
2	50	60	13,0	9,22	0	15,2
3	55	60	13,0	12,36	0	17,9
4	60	60	13,0	13,00	2 h,4 min	10,3
5	55	55	13,0	13,00	8 min	19,0
6	54,5	60	13,0	11,07	0	17,7
7	85	110	18,0	12,92	0	15,4
8	135	160[b)]	18,0	17,81	0	23,7

a) In this stage the maximum load was held for at least 5 minutes.

b) Without flange.

TABLE 2. Comparison of experimental and theoretical axial load capacity

Specimen	N_u kN	λ	N_{cd} kN	$\frac{N_u}{N_{cd}}$
1	12,80	157	10,51	1,22
2	9,22	173	6,82	1,35
3	12,36	157	10.21	1,22
4	13,00	144	11,58	1,12
5	13,00	157	11,88	1,09
6	11,07	158	10,55	1,05

$m = 1,175$

$s = 0,110$

14

Literature cited

/1/ Byggnadsdelars bärförmåga vid brand. PFS 1984:1. Rules for approval of loadbearing capacity for building elements subjected to fire. The National Swedish Board of Physical Planning and Building.

/2/ Report to be published by the Swedish Institute for Wood Technology Research. Tentative title "Simulering av brand vid provning av axialbelastade träreglar".

/3/ CIB Structural Timber Design Code, CIB Report, Publication 66, Working Group W18, 1983.