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The behaviour of a new type of connection system for light-weight steel structures applied to roof trusses

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Abstract. The Rosette-joining system is a completely new press-joining method for cold-formed steel structures. One Rosette-joint has a shear capacity equal to that of approximately four screws or rivets. The Rosette thin-walled steel truss system presents a new fully integrated prefabricated alternative to light-weight roof truss structures. The trusses are built up on special industrial production lines from modified top hat sections used as top and bottom chords and channel sections used as webs which are joined together with the Rosette press-joining technique to form a completed structure easy to transport and install. A single web section is used when sufficient but can be strengthened by double-nesting two separate sections or by using two lateral profiles where greater compressive axial forces are met. An individual joint in the truss can be strengthened by introducing a hollow bolt into the joint hole. The bolt gives the connection capacity a boost of approximately 20%. A series of laboratory tests have been carried out in order to verify the Rosette truss system in practice. In addition to compression tests on individual sections of different lengths, tests have also been done on small structural assemblies and on actual full-scale trusses of a span of 10 metres. Design calculations have been performed on selected roof truss geometries based on the test results, FE-analysis and on the Eurocode 3 and U.S.(AISI) design codes.

Key words: Rosette-joint; truss testing; light-weight steel; roof truss; cold-formed steel; steel sheet joining.

1. Introduction

Rosette joining has several advantages over other common joining methods used in light-weight steel construction, such as riveting, bolting and welding. The joint is formed using the parent metal of the sections to be connected without the need for additional materials. Nor is there need for heating, which may cause damage to protective coatings. The Rosette technology was developed for fully automated, integrated processing of strip coil material directly into any kind of light-gauge steel frame components for structural applications, such as stud wall panels or roof trusses (cf. Fig. 1). The integrated production system makes prefabricated and dimensioned frame components and allows for just-in-time (JIT) assembly of frame panels or trusses without further measurements or jigs.

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Fig. 1 Rosette-joint in a roof truss

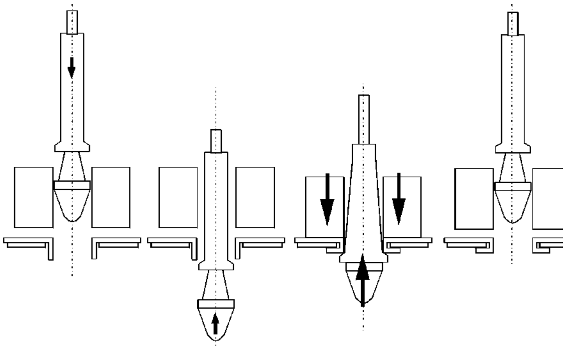

Fig. 2 Rosette joining process

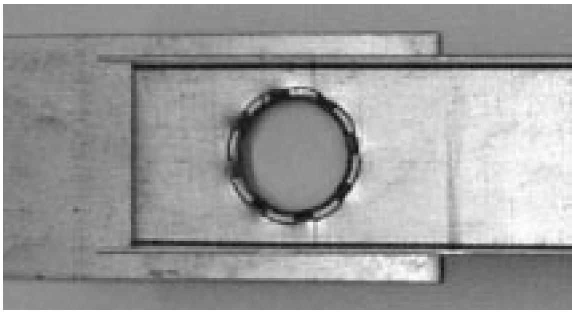

Fig. 3 Rosette-joint

Fig. 4 Rosette tube-bolt

2. The Rosette-joint and the Rosette roof truss system

2.1. The joining process

The Rosette-joint is formed in pairs between prefabricated holes in one jointed part and collared holes in the other part. First, the collars are snapped into the holes. Then the Rosette tool heads penetrate the holes at the connection point, where the heads expand, and are then pulled back with hydraulic force. The expanded tool head crimps the collar against the hole. Torque is enhanced by multiple teeth in the joint perimeter. The joining process is illustrated in Fig. 2 and the finished Rosette-joint is shown in Fig. 3.

2.2. The bolted joint

The Rosette tube-bolt is shown in Fig. 4. The bolt is used in fixing frame components together. However, the bolt has also a strengthening effect on the Rosette-joint. Rosette connection holes serve as nuts when the bolt is screwed through the chord member. The shear capacity of the Rosette-joint with Rosette tube-bolt has been investigated using a T-joint specimen simulating the steel truss application (Kesti and Mäkeläinen 1999). The bolted connection has a capacity approximately 20% higher than the basic joint.

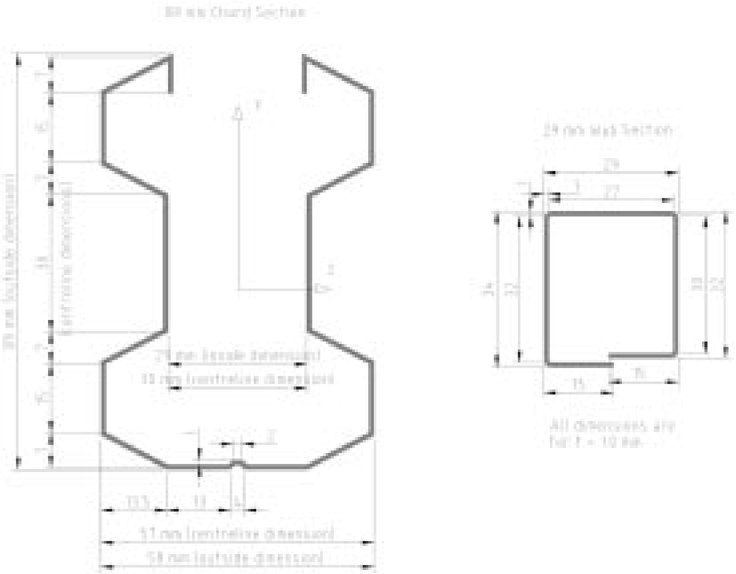

Fig. 5 The profiles used in the Rosette truss system

2.3. Structural truss members

Rosette - trusses are assembled on special industrial production lines from modified hat sections used as top and bottom chords and channel sections used as webs, as portrayed in Fig. 5, which are joined together with the Rosette press-joining technique to form a completed structure. The profiles are manufactured in two size groups using strip coil material of thicknesses from 1.0 to 1.5 mm. A single web section is used when sufficient, but it can be strengthened by double-nesting two separate sections and/or by using two or several lateral profiles where greater axial loads are met. The collar parts of the Rosette joints are in chord members and the holes in the web members. This has an effect on the capacity of the joints and therefore also on the overall design of the truss.

At the present time, the application of the Rosette truss system has been examined in the 6 to 15 metre span range.

3. Experimental research

3.1. General

An extensive test programme has been carried out to investigate the properties of the Rosette joint (Kesti, Lu and Mäkeläinen 1998), the chord and web members (Kaitila 1998) and complete roof trusses (Kaitila 1998, Kaitila and Mäkeläinen 1998, Kaitila and Kesti 2000). This chapter will describe the tests and their results.

3.2. Material properties

The material used for the all the tests was galvanized (hot-dip zinc-coated) structural steel

Table 1 Mean material properties determined in the direction of cold-forming

Test(s) the material is used for	Nominal sheet thickness [mm]	Measured core sheet thickness [mm]	Yield stress f_y [N/mm^2]	Ultimate tensile stress f_u [N/mm^2]	Modulus of elasticity E [N/mm^2]
Simple joint tests, cross-tension tests and web compression tests	1.0	0.93	377	473	202371
	1.5	1.44	394	486	203582
T-joint tests and roof truss test	1.0	0.94	363	468	209602
	1.5	1.42	381	479	198288
Chord compression tests	1.0	0.94	367	457	192222

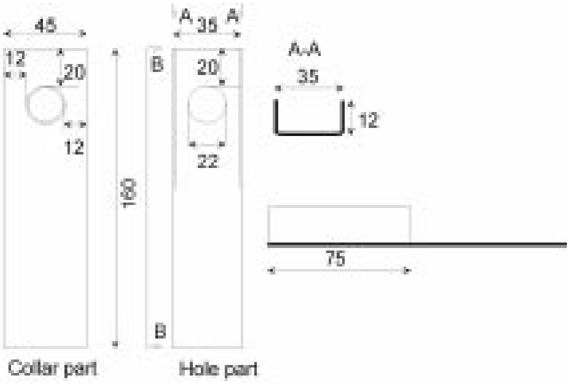

Fig. 6 Connected parts of test specimens in simple shear tests

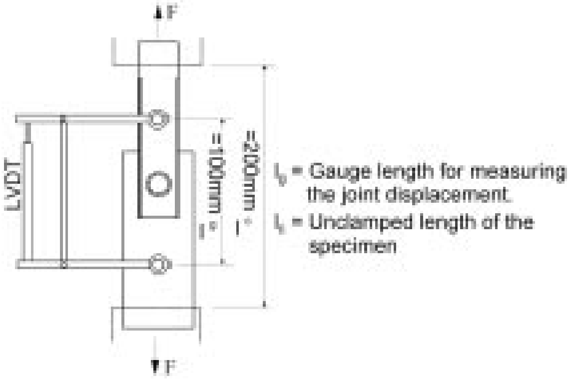

Fig. 7 Test set-up in simple shear test

S350GD+Z with a nominal yield stress of 350 N/mm^2 and nominal sheet thickness of 1.0 mm or 1.5 mm. The tensile coupon tests for the steel sheets used in different tests were carried out and the recorded mean material properties are shown in Table 1.

3.3. Tests on the basic Rosette-joint

3.3.1. Shear tests

Connected parts of the test specimens for the shear tests are illustrated in Fig. 6. The dimensions of the test specimens were chosen to represent those in the applications, such as in steel trusses and wall stud assemblies. The diameter d of the studied joint is 20 mm. The test specimens were fabricated with a side-lipped hole part to avoid curling of the specimen. The shear tests were carried out for the specimens with collar and hole part of both 1.0 mm or 1.5 mm thickness, and for the specimens with hole part and collar part of different thicknesses. The test specimens were cut longitudinally to the rolling direction.

The test specimens in the first test series were aligned between gripping devices of the Roell & Korthaus universal testing machine. A maximum elongation rate of 1.0 mm/min was used in all tests. The test was terminated when about 3 mm elongation was reached. The tests were conducted with an LVDT displacement measurement system. The gauge length for measuring the elongation was 100 mm. The total deformation is a combination of the elongation of the joint and elastic

Table 2 Failure loads [kN] of shear test specimens (HP = Hole part, CP = Collar part)

Test specimen	HP 1.0 mm CP 1.0 mm	HP 1.5 mm CP 1.5 mm	HP 1.0 mm CP 1.5 mm	HP 1.5 mm CP 1.0 mm
1	8.38	12.49	9.93	9.51
2	8.37	12.39	9.90	9.69
3	8.45	12.11	9.99	9.44
4	8.37	13.69	9.95	9.57
5	8.37	12.66	9.99	9.67
6	8.35	12.38		
7	8.38	12.91		
8	8.32	12.41		
9	8.36	12.38		
10	8.58	12.36		
Mean value	**8.39**	**12.58**	**9.95**	**9.58**
St. dev.	0.07	0.44	0.04	0.11

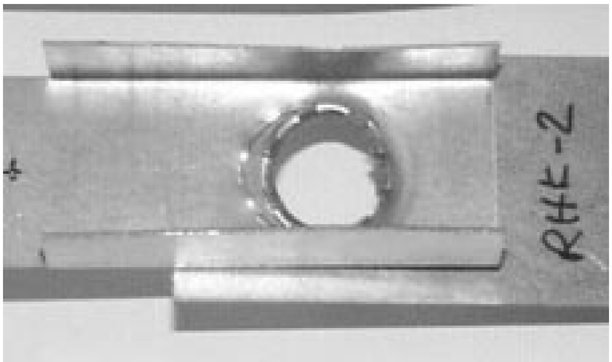

Fig. 8 Failure mode of simple shear test specimen

elongation of the steel itself. The test set-up in the shear test series is illustrated in Fig. 7.

The shear strength for the simple shear test specimens is presented in Table 2. The specimen of 1.0 mm thickness failed in sheet bearing by local buckling of the compressed edge of the hole part as shown in Fig. 8. In addition to the local buckling, the specimen of 1.5 mm thickness also had considerable deformations in the collar part. When the collar thickness was 1.5 mm and the hole thickness 1.0 mm, it was observed that the collar supported the hole better and thus improved the local buckling behaviour. The shear strength was 19% higher than that of the 1.0 mm–1.0 mm pairs. In the case of the 1.0 mm collar part and 1.5 mm hole part, the shear strength was 14% higher than that of the 1.0 mm–1.0 mm pairs. In this case, the failure occurred due to yielding of the collar part.

3.3.2. T-joint tests

In the second series, the shear strength of the Rosette-joint was examined using a T-joint of the steel truss. The behaviour of the Rosette-joint in the steel truss connection was verified in this test. The specimen consists of a chord and web member connected perpendicularly with a joint on both chord web faces (see Fig. 9). The loading was either tension or compression. The T-joint tests were

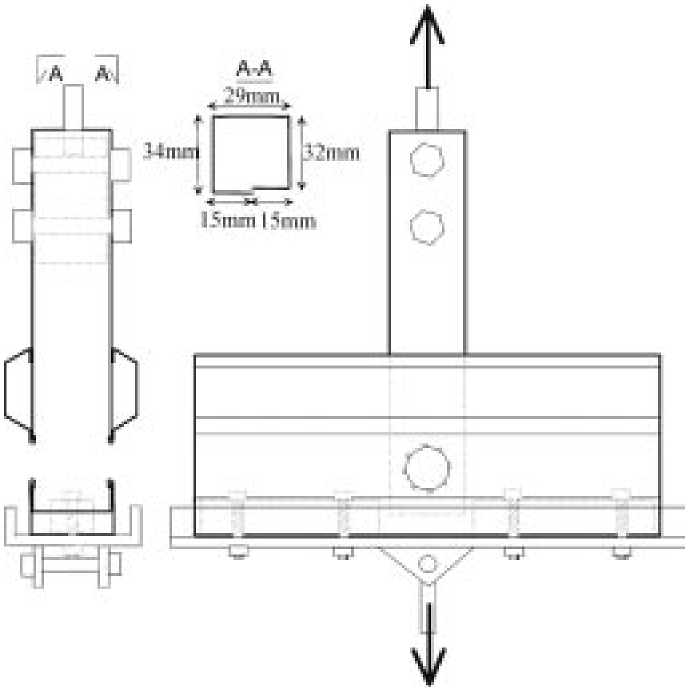

Fig. 9 T-connection test set-up in tension test

Table 3 Failure loads of T-joint specimens with two Rosette-joints, web 1.0 mm

Test specimen	Web 1.0 mm Chord 1.0 mm		Web 1.0 mm Chord 1.5 mm	
	Failure load [kN] in tension	Failure load [kN] in compression	Failure load [kN] in tension	Failure load [kN] in compression
1	16.24	16.81	18.82	20.20
2	16.74	17.01	17.06	21.05
3	16.75	16.40	17.35	20.84
4	15.67	16.93	17.23	19.72
5	16.32	16.81	16.96	20.30
Mean value	**16.34**	**16.79**	**17.48**	**20.42**
St. dev.	**0.44**	**0.23**	**0.76**	**0.53**

carried out for the specimens with chord and web part of both 1.0 mm or 1.5 mm wall thickness, and for the specimens with chord part and web part of different thicknesses.

The tests were conducted applying compression or tension force to the web. In the T-joint tension test, the specimens were bolted to U-section using a 20mm x 20mm steel bar through the chord member bottom flange. The U-section was connected to the bedding through a hinge. The web member was attached to the testing machine using steel bar inside the web. The T-joint tension test set-up is shown in Fig. 9. In the compression test, the test specimen was aligned to the bedding and force was applied to the web part through the ball bearing. An elongation rate of 1.0 mm/min was used in tests. The relative displacement between the web and the top of the chord member was measured.

In the T-joint tests, the load-bearing capacity of one joint was about the same as the capacity of

Table 4 Failure loads of T-joint specimens with two Rosette-joints, web 1.5 mm

Test specimen	Web 1.5 mm Chord 1.0 mm		Web 1.5 mm Chord 1.5 mm	
	Failure load [kN] in tension	Failure load [kN] in compression	Failure load [kN] in tension	Failure load [kN] in compression
1	19.98	20.01	25.99	28.26
2	19.99	20.06	26.23	28.75
3	19.67	19.73	26.65	27.74
4	20.23	19.85	24.53	28.41
5	19.81	19.22	27.09	28.48
Mean value	**19.94**	**19.77**	**26.10**	**28.33**
St. dev.	**0.21**	**0.34**	**0.97**	**0.37**

the folded test specimen in the simple shear test, as shown in Table 3. In the tension test, in the case of the chord part of 1.5 mm wall thickness and the web part of 1.0 mm wall thickness, the joint failed by curling of the lip of the web member leading to about 14% lower shear strength than in compression test. Quite similar behaviour was obtained in the case where both members were thickness of 1.5 mm. The shear strength in tension was about 8% lower than in compression test.

3.4. Tests on individual truss members

3.4.1. General

A series of tests have been done on individual truss members in order to assess their behaviour in compression and tension. The profiles used in the tests (Fig. 10) on these members were slightly different from the ones currently used in the Rosette roof truss system (Kaitila 1998). Since the accomplishment of these tests, the profiles have been further optimized to better meet the

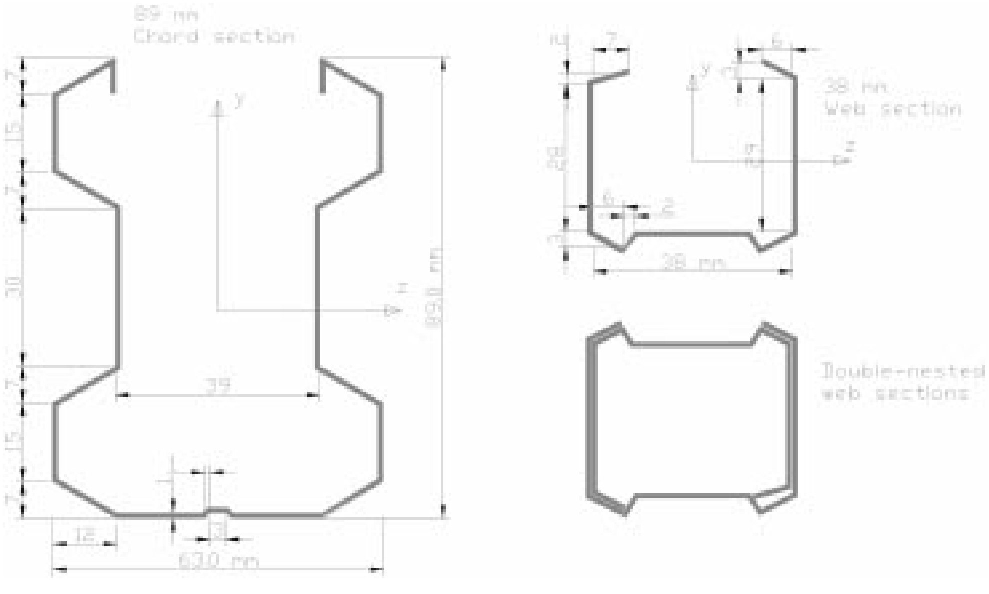

Fig. 10 Cross-sections of the 89 mm Rosette chord and 38 mm web members

requirements of an overall efficient and economical roof structure design.

3.4.2. Tests on web members

Axial compression tests were carried out on four differently arranged sets of 38 mm web sections of measured cross-sectional thickness 0.94 mm in order to verify their actual failure mode and load. The specimens in groups 1 to 3 were prepared for testing by casting each end in concrete, thus providing rigid end conditions. All specimens, including group 4, were placed firmly on solid smooth surfaces and the compressive force was applied axially on the gravitational centroid of the members.

The test results are summarised in Table 5. In test groups 1 and 2, they are quite consistent with analytical values determined according to Eurocode 3, Part 1.3. Group 3 consists of two specimens of web members with two profiles freely nested one inside the other. The analytical compression capacity was obtained by simply multiplying the capacity of a single profile by two. The average maximum load from the tests was approximately three-fold the test value for a single profile. This high value is due to the greater torsion resistance of the nested profiles when compared to single profiles.

Test-group 4 differs from the first three groups in its overall arrangement and motives. The idea was to examine the way the joints connecting the web profile to the chord profiles perform under axial loading, and how much rotational support they give to the web profile that has been initially considered hinged at both ends. Each of the three test specimens consisted of a 1,060 mm long web profile element connected by Rosette-joints at each of its ends to a 400 mm long piece of chord profile. The length of the specimens was chosen great enough to prevent the failure of the joints before buckling occurred. The distance between the midpoints of the joints was then 1,003 mm for all three specimens. The average maximum test load value was approximately 39% larger than the analytical value calculated with an effective buckling length reduction factor of $K_b = 1.0$. The test load value corresponds to an analytical buckle half-wavelength of 780 mm ($K_b = 0.78$). This indicates that it would be safe to use an effective buckling length reduction factor of $K_b = 0.9$, as is quite

Table 5 38 mm web compression test results (*T* = torsional buckling, *F* = flexural buckling, *D* = distortional buckling)

Test Group #	Test piece number #	Total length-after setup mm	Theoretical Buckle Half-wavelength mm	Analytical Compression Capacity kN	Test Result kN	Ratio between test result and analytical result	Failure Mode
1	1	660	330	33.44	34.24	1.02	*T+D*
	2	660	330	33.44	36.02	1.08	*T+D*
	3	660	330	33.44	36.80	1.10	*T+D*
				Average:	**35.69**	**1.07**	
2	4	1061	530.5	25.56	23.04	0.90	*T*
	5	1060	530	25.56	25.06	0.98	*T*
	6	1060	530	25.56	26.94	1.05	*T*
				Average:	**25.01**	**0.98**	
3	7	1063	531.5	45.14	74.72	1.66	*F+T*
	8	1061	530.5	45.14	75.61	1.68	*F+T*
				Average:	**75.17**	**1.67**	
4	11	1000	1000	9.27	13.21	1.42	*T*
	12	1000	1000	9.27	13.24	1.43	*T*
	13	1000	1000	9.27	12.20	1.32	*T*
				Average:	**12.88**	**1.39**	

Table 6 89 mm chord compression test results (*TF* = torsional-flexural buckling mode)

Test Group #	Test piece number #	Total length after setup mm	Theoretical Buckle Half-wavelength mm	Analytical Compression Capacity kN	Test Result kN	Ratio between test result and analytical result	Failure Mode
1	1	1258	629	52.68	47.28	0.90	*TF*
	2	1255	627.5	52.68	46.92	0.89	*TF*
	3	1255	627.5	52.68	49.85	0.95	*TF*
				Average:	**48.02**	**0.91**	
				St. deviation:	1.60		
2	4	1754	877	32.95	34.65	1.05	*TF*
	5	1751	875.5	32.95	34.54	1.05	*TF*
	6	1755	877.5	32.95	34.37	1.04	*TF*
				Average:	**34.52**	**1.05**	
				St. deviation:	0.14		

usual practice in roof truss structures.

3.4.3. Tests on chord members

Similar compression tests to those carried out on individual web profiles (test-groups 1 and 2) have been performed on chord profiles. The test results and their comparison with analytical values are shown in Table 6. The actual structure will include continuous chord members that are connected to web members at different intervals and laterally supported by braces at 600 mm intervals.

It can be concluded that the design procedure used for the evaluation of the compression capacities is quite compatible with the test results. The analytical calculations and FE-analyses performed predicted a torsional-flexural buckling mode with a stronger deflection in the *y*-direction and the test results supported this prediction. Also the maximum loads observed in the tests comply with the analytical values to an acceptable degree.

3.5. Full scale truss test

3.5.1. General

The aim of the full scale truss test was to verify the behaviour of the individual structural

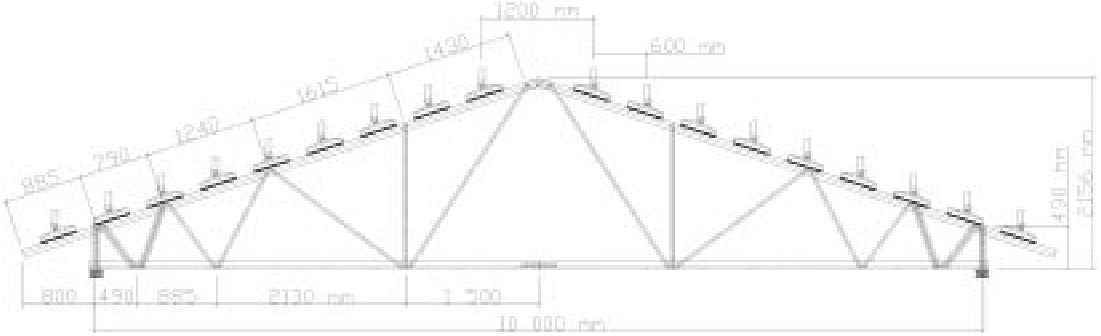

Fig. 11 Nominal geometry of test truss with load cylinders

Fig. 12 General view of truss before testing

components in the actual end structure. The testing procedure follows the instructions of Eurocode 3 Part 1.3 Appendix A4. The test truss is a symmetric truss with a span of 10.0 metres and an eaves length of 800 mm. The truss is symmetrical about its centreline with a top chord inclination of 18.435 degrees (equal to a pitch of 1/3). The top chords are connected to each other at mid-span using a folded plate structure that is connected to each chord through two Rosette connections. The design of the test truss was based on typical Finnish loadings. The truss is simply supported at its ends and its movements out of its own plane are restrained along the top chord. The truss is schematised in Fig. 11 and pictured before testing in Fig. 12.

Two similar trusses have been tested previously (Kaitila 1998, Kaitila and Mäkeläinen 1998). In the previous trusses, the truss members have always been produced manually, but now the first industrial Rosette production line has been opened and the third truss was produced on that line so that this test verifies also the performance of the industrially produced truss members. The profiles were manufactured according to Fig. 5.

3.5.2. Design of the test truss

3.5.2.1. Design loads

The tested truss was designed according to Eurocode 3. A first analysis was performed with the following nominal loading:

- -Self-weight of the roof truss
- -Western Finland snow load 1.6 kN/m^2 (on the ground and including roof form factor μ = 0.8)
- -Suspended structure dead load 0.15 kN/m^2 on the bottom chord

3.5.2.2. Computer model and analysis results

A STAAD Pro FE-analysis was performed for the determination of the force distribution and the deflections of the truss under service and design loading. The critical members were chord number 11 and the corresponding member on the other side of the truss (see Fig. 13). These members' utilisation factor in design was 1.01. The critical web members were members 56 and its equivalent on

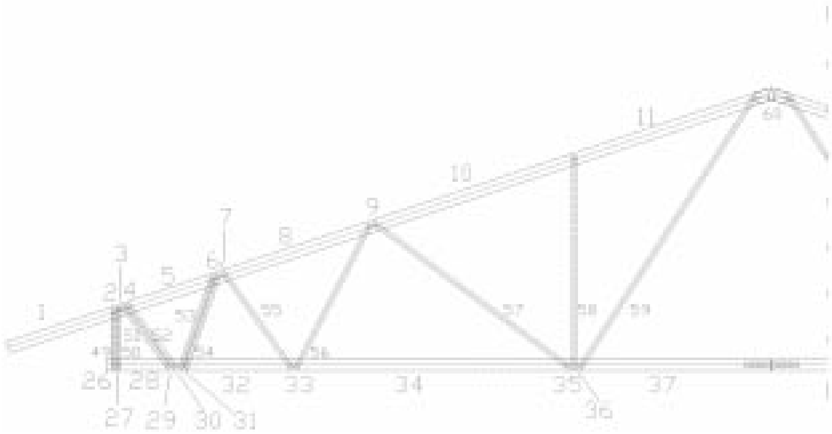

Fig. 13 Member numbering

the other side of the truss with a utilisation ratio equal to 0.68. The critical joints were those connecting the inner vertical webs at the supports to the top chord with a utilisation ratio equal to 0.95.

3.5.3. Truss data

The dimensions of the actual truss differed quite little from the nominal values. The span of the truss was 10,000 mm, the mid-span height was 2,153 mm and the height of the truss right above the support was 488 mm measured along the outer edge of the outermost vertical web. The horizontal length of the eaves was 770 mm measured from the outer edge of the outermost vertical web to the lower flange of the top chord. These values are practically identical to those designed. The total mass of the actual truss was 75 kg.

The actual dimensions of the manufactured profiles were measured. Actual dimensions differed quite little from the nominal values. The maximum errors were of a magnitude of 1 mm. The openings (inside gap between the lips) of the chord sections were measured in the longest top chord members of the truss. The width of a web member was also similarly measured. The magnitudes of the maximum initial imperfections were about 5.3 mm for the chords and 2 mm for the webs.

3.5.4. Outline of test procedure

3.5.4.1. Support conditions

The truss is supported at the ends of the bottom chord with pinned supports. All horizontal displacements are prevented at the left-hand support and are free in the plane of the structure at the right-hand support. The support plates are long enough to allow for a sufficient support area for both vertical web members at the support.

The lateral supports are made at the top chord after every 600 mm by simply connecting the top flange of the chord to the c600 loading rig. First, a 500 mm piece of wood was screwed onto the top flange of the top chord and then, the pointed steel heads of the nodules in the loading pads were compressed against the wood board. The load cylinders are hinged in the plane of the structure but fixed in the plane perpendicular to that of the truss. In addition, several strings were tied loosely around the webs and the test rig in order to prevent accidents in case of sudden breakage of members.

The vertical web profiles on the supports were designed so that they lean against the bottom flange of the bottom chord and could thus directly transmit the load from the structure onto the

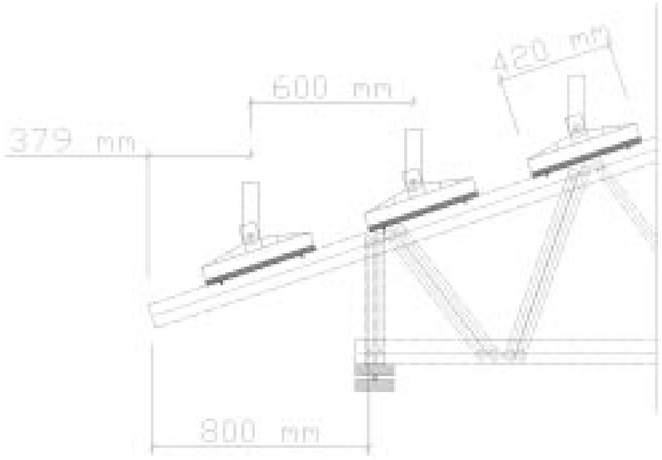

Fig. 14 Sketch of loading arrangement

support as compression, without the chord member having to support superfluous local forces which would inevitably cause strong distortion in the lower part of the chord member.

3.5.4.2. Loading of the test truss

General

The testing was performed according to the procedure described in Eurocode 3 Part 1.3 Appendix A4: Tests on Structures and Portions of Structures.

Only symmetrical evenly distributed loading was considered in this test. The load was pumped into a hydrostatic pressure cylinder using a hand pump and subsequently evenly divided between all 18 load cylinders. Each load cylinder had a 420 mm long loading pad that transmitted the load from the cylinder onto the structure (see Fig. 14). The load pad is 80 mm wide which makes it possible to place the 63 mm wide top chord profile centrally under the pad and leave a minimum space of approximately 8 mm for distortional or other deformation of the cross-section on both sides of the profile.

Loading Phases

The loading is performed in three distinct stages according to EC 3. In the first stage ('Acceptance Test'), the loading is gradually augmented to a load value corresponding approximately to the characteristic loads (Eurocode 1) the structure is designed for. Under the 'Acceptance Test' load, the structure should demonstrate substantially elastic behaviour. No significant local distortion or defects likely to make the structure unusable should remain after this stage.

This first phase of testing is then followed by a 'Strength Test' where the total test load is determined from the total design load specified for ultimate state verifications by calculation. The design load is multiplied by a load adjustment factor that takes into account the differences between nominal and measured yield stress and material thickness. Under the "Strength test" load there should be no failure by buckling or rupture in any part of the specimen and after the removal of the load, the residual deflection should be reduced by at least 20%.

A third phase of loading is then performed in order to find out the actual failure mode and load of the structure. The ultimate load carrying capacity should be taken as the value of the test load at the point at which the structure is unable to sustain any further increase in load.

Load Values for Loading Phases

The total load value determined according to the above method for the "Acceptance test" is 30.03

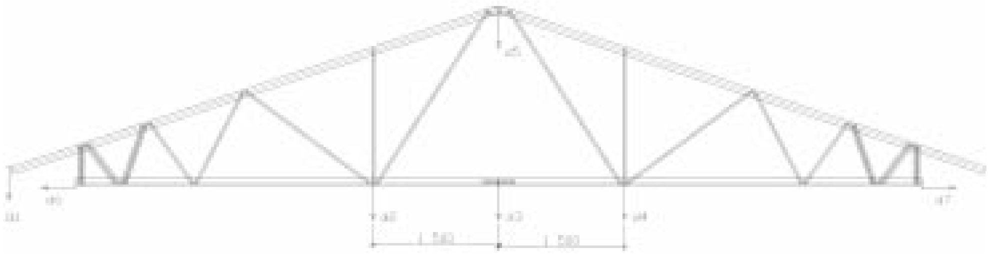

Fig. 15 Locations of displacement measurements

kN. The corresponding value for the "Strength test" is 35.16 kN.

3.5.4.3. Measurements

Vertical deflections are measured with displacement bulbs at the mid-point and the most central chord-web-connection areas of the bottom chord. Horizontal displacements at the supports are measured. There are also vertical displacement measurements at the end of one of the eaves and the top chord connection at mid-span. The locations of deflection measurement bulbs are shown in Fig. 15.

3.5.5. Progression and results of the full scale truss test

The Rosette test truss number 3 was tested on September 7, 2000 at the testing hall of the Tampere University of Technology following the procedures explained above. Fig. 16 shows the deflection at point d2 (see Fig. 15), where the maximum deflections occurred.

After the application of a bedding-down loading, the "Acceptance test" phase was begun. The behaviour of the truss was linear and no signs of buckling or other irregularities were seen. This

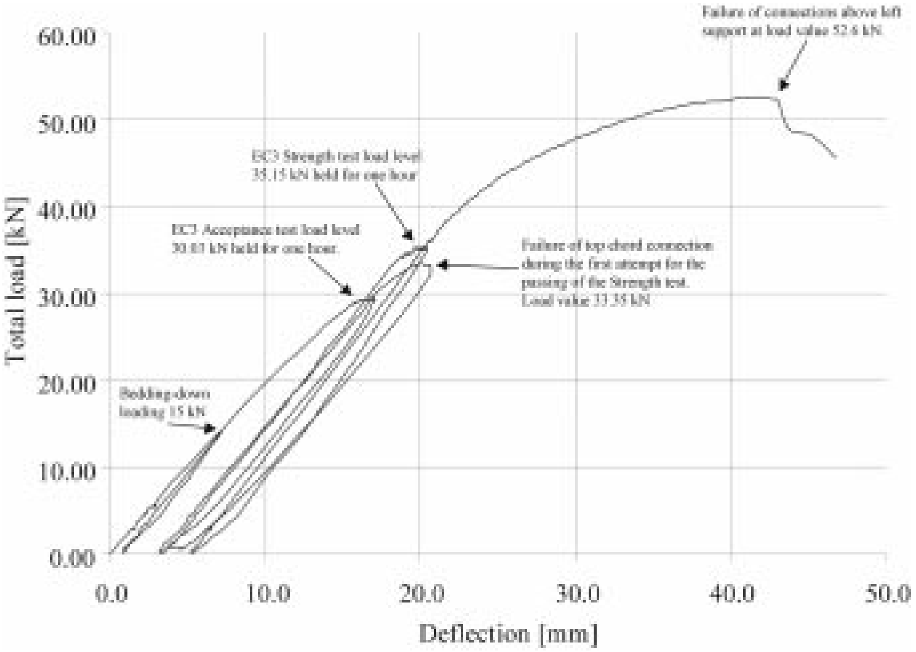

Fig. 16 Deflection at the maximum deflection point d2

first test phase was successfully completed, as the residual deflection was just under 20% of the maximum recorded.

After the completion of the “Acceptance test”, the “Strength test” phase could be begun. During this stage, the collapse of the top chord connection piece by plate buckling took place just before reaching the constant load value determined for the “Strength test”. The load was taken off and the top chord connection was fixed using a 250 mm long steel tube member (30×30×15) that was attached by a total of six 4.2×19 self-drilling screws (3 on each side) to each of the chord members.

The damage caused by the plate buckling at the top chord connection area was localised and did not cause other irregularities in the truss. Therefore it can be considered that after the connection was fixed and made secure, the truss test could be continued in the normal fashion. The fixed connection showed no more signs of instability during the remainder of the test.

After the load was taken off, the residual deflection was reduced by about 74% of the maximum recorded. The requirement being that the deflection be reduced by at least 20%, the truss passed the “Strength test” phase without problems.

The “Strength test” was thus run successfully and the final stage of the test, “Prototype failure test” was begun. The load was slowly increased until failure occurred. As was noted above, the top chord members were critical in design. They suffered noticeable deformations, as can be seen from Fig. 17 below. However, they did not prove to be critical for the load carrying capacity of the truss. The actual failure occurred at the left support area via the rupture of the critical joints, as was the case also with the previous tested trusses. The ultimate load capacity of the test truss was 52.6 kN. Fig. 18 shows the ruptured joints.

Fig. 17 Deformations of the top chord at total load value 39 kN

Fig. 18 Ruptured joints after test

3.5.6. Conclusions from the truss test

The truss passed the complete Eurocode 3 testing procedure without great difficulty. At the time of failure, the capacity of the ruptured joint had already been passed by approximately 13% in relation to the calculated nominal capacity. The most stressed top chord member had a capacity as much as 38% greater than its calculated nominal capacity.

The connection between the top chords at the mid-span of the truss did not prove to be efficient. It had to be strengthened during the test. As a replacing solution, it is suggested that a simple $t = 1.5$ mm web stud is used between the top chords. The joints of this member can be strengthened by using bolts in the joint holes and/or self-drilling screws.

4. Conclusions

This study on a new joining technique for steel sheets based on the Rosette-joint has been completed. A series of shear tests was carried out to determine the shear strength capacity values of the joint. The shear test specimens of 1.0 mm thickness failed in sheet bearing by local buckling of the compressed edge of the hole part. In addition to the local buckling, the specimen of 1.5 mm thickness also had considerable deformations in the collar part. The shear tests show that the capacity of the hole part is dependent on the supporting conditions of the hole-edge. The capacities determined using simple shear tests and the performance of the joints in roof truss applications were verified by T-joint sub-assembly tests whose results were in compliance with those of the simple shear tests. The shear capacity values of the Rosette-joint are sufficient for applications in light-weight steel-framed wall panels or light-weight steel trusses.

The capacities of special structural members for use in truss applications have been determined according to European and American design codes and verified by testing. Three full scale roof truss tests have been carried out according to the Eurocode 3 procedure. The behaviour of the trusses was linear and predictable throughout the testing procedure. The structure successfully passed the first and second stages of the EC 3 testing procedure, ‘Acceptance Test’ and ‘Strength Test’, respectively. The “Prototype failure Test” showed that the truss had reasonable and sufficient safety margins for the capacities of the members and joints when the actual failure load is compared to the predicted failure load.

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