Study on the Behaviour of a New Light-Weight Steel Roof Truss

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ABSTRACT

The ROSETTE thin-walled steel truss system presents a new fully integrated premanufactured alternative to light-weight roof truss structures. The trusses will be 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 jointed together with the ROSETTE press-joining technique to form a completed structure easy to transport and install. The trusses can be case-dependently designed in order to efficiently respond to different structural forms and loading conditions. A single web section is used when sufficient and can be strengthened by double-nesting two separate sections or by using two or several lateral profiles where greater compressive axial forces are met. All system components have industrial-quality cut details. The profiles may be formed using pre-coated steel for special architectural effects when so desired.

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 are also done on small structural assemblies, e.g. the eaves section, and on actual full-scale trusses of 10 metre span. Design calculations have been performed on selected roof truss geometries based on the test results, FE-Analysis and on the Eurocode 3 [6], U.S.(AISI) [1] and Australian / New Zealand (AS) [2] design codes. The design calculations are not presented in this paper, but can be found in the diploma thesis of the author [8].

INTRODUCTION

The Rosette-joining system is a completely new press-joining method for cold-formed steel structures. Rosette joining has several advantages over other common joining methods used in 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.

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.

This paper presents the first extended test programme performed on the ROSETTE light-weight steel roof truss system. Results of tests on individual members and full scale roof trusses are presented and analysed.

THE ROSETTE - JOINT

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.1 and the finished Rosette-joint is shown in Fig.2.

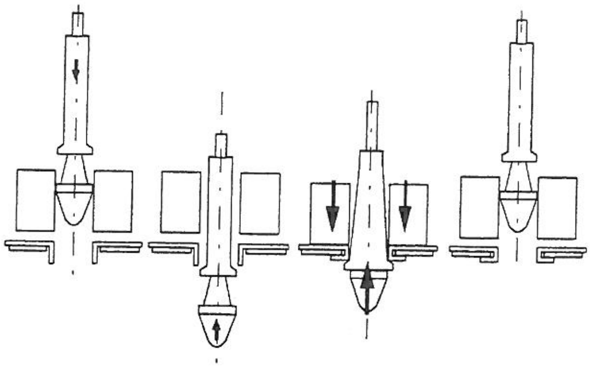

Fig.1. Rosette-Joining Process.

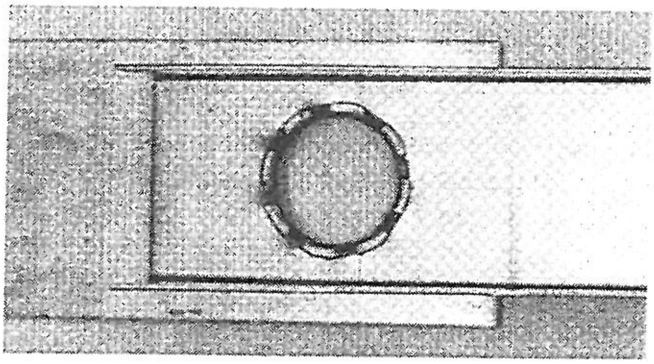

Fig.2. The Rosette-Joint.

DESCRIPTION OF THE ROSETTE - ROOF TRUSS SYSTEM

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 (Fig.3) which are jointed together with the Rosette press-joining technique to form a completed structure easy to transport and install. The profiles are manufactured in two size groups using strip coil material of various 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 (see Fig.3) and/or by using two or several lateral profiles where greater axial loads are met. At the present time, the application of the Rosette truss system is being examined in the 6 to 12 meter span range.

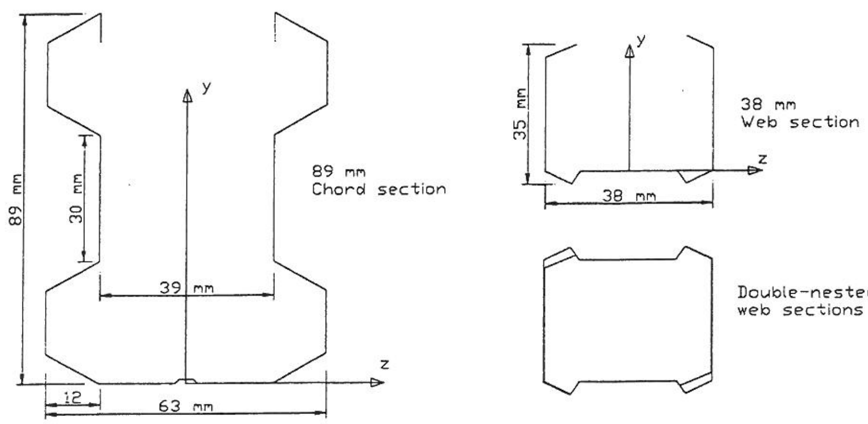

Fig.3. Cross-Sections of the 89 mm Rosette Chord and 38 mm Web Members.

TESTS ON WEB MEMBERS

Axial compression tests were carried out on four differently arranged sets of 38 mm web sections of cross-sectional thickness 0.94 mm in order to verify their actual failure mode and load. The test specimens are listed and described in the following :

1. Single-web profiles of 660 mm free length, a total of three specimens.
2. Single-web profiles of 1 060 mm free length, a total of three specimens.
3. Two web profiles nested one inside the other, of a free length of 1 060 mm, a total of two specimens.
4. Single-web profiles of 1 003 mm (free length connected with Rosette-joints to 400 mm pieces of chord members at each end, a total of three specimens.

The specimens in groups 1 to 3 were prepared for testing by casting each end in concrete, thus providing unhinged end conditions. All specimens, including group 4, were placed firmly on solid smooth surfaces and the compressive force was applied centrally on the gravitational centroid of the members.

The test results are summarized in Table 1. In test groups 1 and 2, they are quite consistent with analytical values determined according to EC 3 , Part 1.3, when rigid end conditions are assumed.

Table 1. 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**	

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 for 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 capability of the nested profiles to resist torsion 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 adjacent chord profiles in the actual structure 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 occured. 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 usual practice in roof truss structures.

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 actual structure will include continuous chord members that are connected to web members at different intervals and laterally supported by braces every 600 mm.

Table 2. 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		

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.

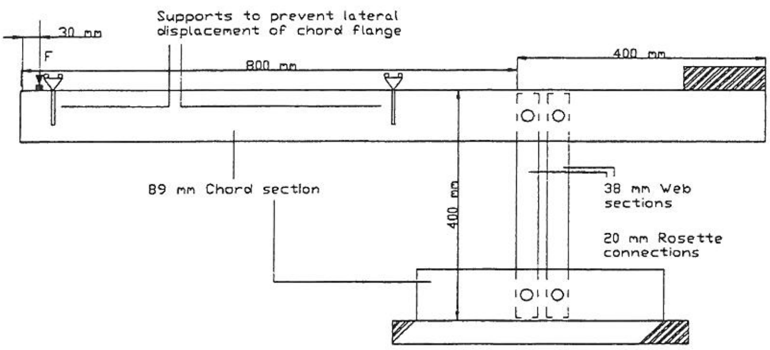

Fig.4. Eaves Chord Test Member.

In roof-truss structures, the eaves element requires special attention, because it is usually at the end support where the greatest negative bending moments occur in the top chord member under downward loading (dead load, snow, etc.). The webs of the chord profile can open up due to this negative moment. Tests were performed where an assembled eaves specimen is loaded in bending. The test arrangement is outlined in Fig.4. An 800 mm long cantilever chord section was loaded at its free end with a concentrated load.

The tests showed that the eaves member tends to buckle laterally to one side after an initial phase of light distortional deformations of the chord webs. The average maximum load obtained from the two test specimens was 2.79 kN giving a maximum moment of M_{test} = 2.79 kN $\times$ 0.800 m = 2.23 kNm. The analytical value calculated according to the Australian / New Zealand Standard (AS 3.3.3) [2] for the evaluation of critical bending moment (including distortional buckling modes) was 22.8 % larger than the test result. The principal conclusion drawn from these tests is that the analytical values are slightly conservative compared to the test results.

TESTS ON FULL-SCALE TRUSSES

The Test Truss

General. Two full scale 10 metre span trusses have been tested according to the testing procedure described in Eurocode 3: Part 1.3 Appendix A4 [6]. The first truss passed the first phase of testing, i.e. the 'Acceptance Test', but failed during the load increase phase of the next test round, i.e. the 'Strength Test'. This failure was due to manufacturing difficulties and insufficient detail design of the truss [8]. The information received from the first test was analysed and used to improve the details of the second truss while preserving the original basic geometry. The different phases and the results of the second truss test are given in the present chapter.

Material data. The test truss was manufactured from steel plate with cross-sectional wall thickness t_{obs} = 0.95 mm (+ zinc coating), yield stress $f_{y,obs}$ = 368 N/mm^2, and elastic modulus E = 189 430 N/mm^2 (all values taken for steel in the direction of cold-forming).

Geometry Data. The profiles used were a modified 89 mm chord and a new 29 mm web profile (cf. Fig.5). 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 support as compression, without the chord member having to support unneccessary shear force which would cause strong distortion in the lower part of the chord member, as observed in the tests on eaves members.

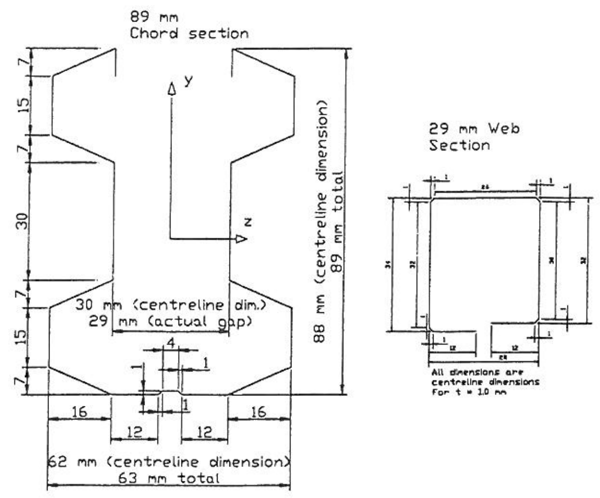

Fig.5. The Profiles used in the Second Truss Test.

The nominal geometry of the tested truss is outlined in Fig.6. The truss is symmetrical about its center line with a top chord inclination of 18 degrees. The height at the support is approximately 490 mm, which gives the truss a total height of about 2100 mm. The top chords are connected to each other at mid-span using a short web member and specially manufactured jointing plates. The total mass of the actual truss was 75.5 kg.

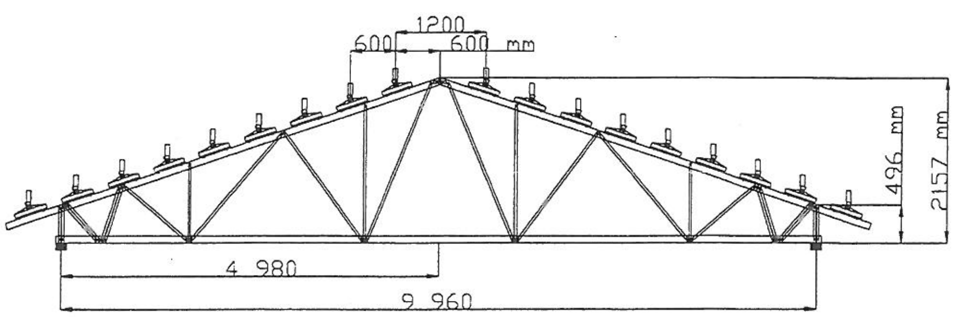

Fig.6. Nominal Geometry of Test Truss with Load Cylinders.

Support Conditions. The truss is supported at the ends of the bottom chord with pinned supports. All horizontal displacements are prevented at the lefthand support and free in the plane of the structure at the righthand support. The support plates are long enough to allow for a sufficient support area for both web members at the support. The lateral supports are made at the top chord every 600 mm by simply bolting the top flange of the chord to the c600 loading rig. The load cylinders are hinged in the plane of the structure but fixed in the plane perpendicular to that of the truss.

Imperfections of the Actual Tested Truss. The dimensions of the actual truss differed quite little from the nominal values. The actual dimensions of the manufactured profiles differed from the nominal cross-sections by less than 5 %. The production of the joints was done successfully this time without the problems that occurred in the manufacturing of the first test truss.

Outline of Test Procedure

Loading. The testing was performed according to the procedure described in Eurocode 3 Part 1.3 Appendix A4 : Tests on Structures and Portions of Structures [6]. This method includes three distinct phases, an 'Acceptance Test', a 'Strength Test' and a 'Prototype Failure Test'. The course of the test is explained further below.

The loading was applied at eighteen distinct points (nine on each side of the truss's midline) with c600 mm space between them so, that at mid-point there was no load and thus the space between the two middle load cylinders was 1 200 mm (cf. Fig.6). This way, it was possible to get loading also onto the eaves parts of the top chords, which wasn't possible with the first tested truss [8].

Only symmetrical evenly distributed loading was considered in this test. The load was pumped into a hydrostatic pressure cylinder using a handpump and subsequently evenly divided between all 18 load cylinders. Each load cylinder had a 420 mm long loading pad which transmitted the load from the cylinder onto the structure. The loading pad is 80 mm wide which made 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.

Deflections. Vertical deflections were measured with displacement bulbs at the mid-point and the quarter points of the bottom chord, and at the ends and the mid-point of the top chords. Horizontal displacement of the supports was also measured.

Computer Model of Test Truss

A STAAD III-analysis was performed for the design of the truss. The material values used in the computer analysis were as follows:

- wall thickness $t = 0.96$ mm

- yield strength $f_y = f_{yb} = 350$ N/mm^2
- modulus of elasticity $E = 210\,000$ N/mm^2

The connection (i.e. two joints) capacity used in the analysis was taken as $F_{c,conn} = 10.8$ kN.

Progression and Results of the Full Scale Truss Test

The second test truss successfully passed all phases of testing and the maximum load reached was 48.5 kN.

The course of the test can be most simply explained with the aid of a diagram showing the deflection of the truss at mid-span measured from the bottom chord (cf. Fig.7). The graph is complemented with numbers showing the different phases of testing, which are explained in the following:

1. The test was begun at zero load and the load was steadily increased up to 25.16 kN, where it was held for one hour. The unlinearities in the curve during load increase were caused by the movement in the joints due to production tolerances. Point 1 marks the beginning of the one hour period. During load increase or decrease, displacement values were taken at 5 second intervals. During the constant load phases, they were recorded every 30 seconds.

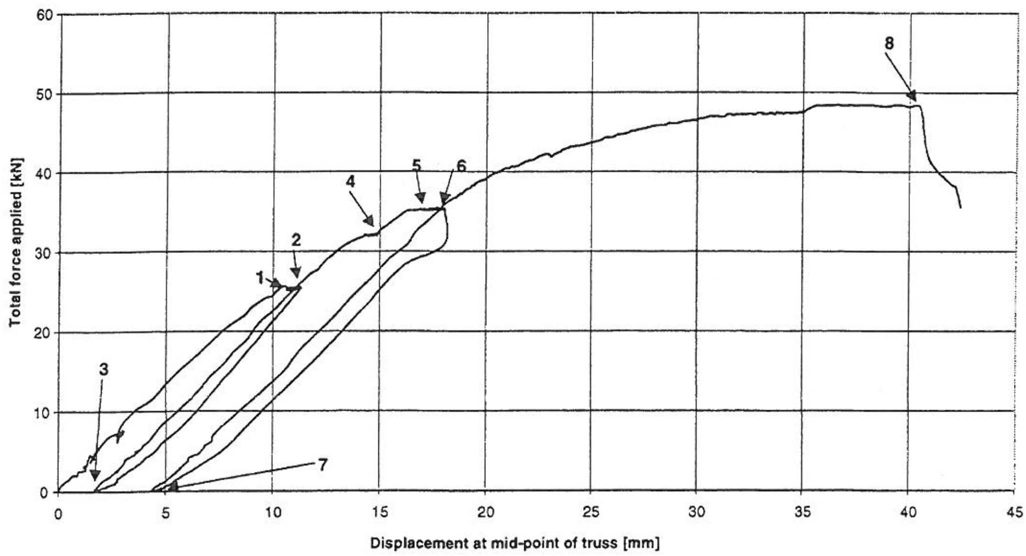

Fig.7. Deflection at Mid-Span of Truss (see text for explanations).

2. Point 2 marks the end of the one hour period. The maximum deflection at this stage was 11.28 mm or $L / 850$. The load was then gradually taken off.
3. The residual deflection after the 'Acceptance Test' phase was 1.74 mm (15 % of the maximum recorded). The allowable value is 20 %, so the truss passed this first phase successfully. Because the vertical webs were initially extended all the way to the bottom of the chord on the support, no local deformation of any importance occurred during this phase. The behaviour of the truss was very good during this first phase.
4. The test load was initially evaluated as 32.0 kN due to a miscalculation. Therefore a quick decision was made at the beginning of the one hour period of this second phase of testing, to increase the test load by 10 % up to 35.2 kN. Point 4 marks the small escalation caused by this mistake before the 10 % increase.
5. Line 5 shows the beginning of the one hour period of the 'Strength Test' phase at load value 35.2 kN.
6. Point 6 marks the end of this one hour period. The maximum deflection recorded at this stage was 18.06 mm or $L / 550$.
7. Point 7 marks the residual deflection at mid-span after the removal of the load. This total residual deflection was 4.51 mm, i.e. the deflection was decreased by 75 %, much more than the 20 % needed at this stage. No actual tear was observed, but a slight beginning of local deformations could be seen in the chord members in the area of the most heavily loaded joints, i.e. beginning shear deformations like the ones portrayed in Fig.8 were starting to appear, but in a much smaller scale than in the photographs.

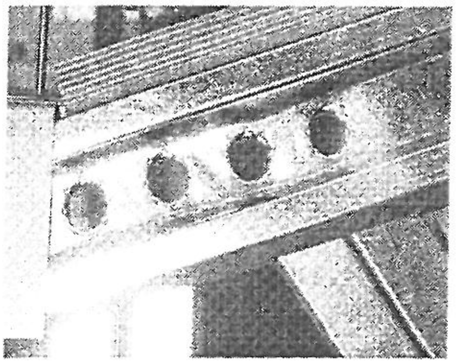
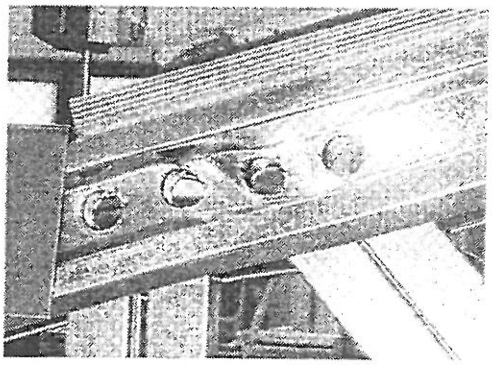

Fig.8. Deformations at the left side Support Area of the Top Chord just before Failure (left) and after Failure (right).

The free edges of the top chord deformed into slight sine-shaped curves under loading, as expected, but this deformation was elastic and the original shape of the members was regained after the removal of the load. The deformation happened in such a way, that consecutive portions separated by web members were deformed in opposite directions, i.e. the first one towards the inside, the second one towards the outside etc. A similar deformation occurred in the bottom chord, although this part of the structure should primarily be under tensile stress. The effect of bending moment caused the deformation of the free edges of the bottom chord profiles. The individual web members did not show indication of insufficiency.

8. After the truss had satisfactorily passed the 'Strength Test'-phase, the last stage with loading up to failure was begun. During the increase of the load, the longer webs were considerably deformed in torsion and flexure. Nevertheless, the final failure did not occur directly due to this but to the joints in the first tension webs counting from outside, as expected from the computer analysis. The failure load was 48.5 kN, although it can be argued that the functional capacity of the truss was reached around a total load value of 46 kN, because of the strong torsional-flexural deformations of the longer web members.

CONCLUSIONS

This paper presentes the general results of the first analysis including a test programme on the Rosette - steel roof truss system and individual members.

The behaviour of the truss 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 manufacturing of the truss was carried out with a much better standard of quality than in the first test, where several imperfections caused the truss's early failure [8].

The individual members acted well in this test. There was no significant plastic deformation before the last stages prior to failure.

The safety factors for the joints are considerably larger than those used for the members ($\gamma = 1.25$ compared with $\gamma = 1.1$, respectively). Therefore it is not surprising that it is the joints that tend to become critical in truss design. Furthermore, because the chord members did not cause any problems in this test, it might be concluded that the chord profile has unnecessary extra capacity and reasons for reducing the chord profile in size might exist. However, it is perhaps too early to draw such a conclusion, since the effects of this type of change need to be examined on the level of a complete structure.

The connection technique used to join together the top chords at mid-span should be studied and designed in a more efficient manner with an analysis extending to the effect of a suggested solution on the behaviour of the complete structure.

The truss passed the requirements set by the European design standard. Further optimization and more detailed design is needed for the application of the Rosette system to high-quantity production, but a strong confidence in the abilities of the system can be justified by this test.

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