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STRUCTURAL DESIGN STUDY ON A LIGHT - GAUGE STEEL PORTAL FRAME WITH COLD-FORMED SIGMA SECTIONS

by

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SUMMARY

Design study is conducted on a thin-gauge cold-formed steel portal frame with compound double (back-to-back) sigma sections in columns and rafters. Experimental study on the semi-rigid rafter-to-column connection behaviour is first carried out for two special bolted connection types based upon specific thin connection plates between the double sections. First connection plate studied was a compact corner plate bolted between double sigma sections of rafter and column end parts. Other connection plate type studied was composed of four together layered thin plates with the edges of two outer plates outwardly lipped at the free side of the corner plate. Connection behaviour in the ridge joint of the portal frame also constructed with using a connection plate between double sigma sections of rafter ends was experimentally studied in a similar way as the rafter-to-column connection behaviour. Analytical models are developed for the semi-rigid connection behaviour in the corners and in the ridge of the portal frame and these models are then used in design calculations for the whole portal frame. One design case study for a 12 m span portal frame is completed by using the newest version of Eurocode 3.

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1. INTRODUCTION

Light-gauge steel framing by using cold-formed open sections has gained during recent years an established application in light-weight industrial building frames. Portal frames in these buildings are typically constructed from cold-formed members having sigma section profile. Bolted connections in these frames can be constructed with using thin steel connection plates between compound double (back-to-back) sections both in the rafter-to-column and ridge connection. In case of longer span frames, tie bar between rafter-column connection points is necessary for stiffening and stabilizing the frame.
Connection behaviour in the joints between rafter and column and also between rafters in the portal frame ridge is semi-rigid i.e. moment - rotation behaviour is becoming non-linear at a relatively low bending load in the connection. This is due to the fact that in these bolted joints composed of thin-walled sections and plates local buckling is occuring and associating gradual loss of stiffness in the joint. In this study, experimental investigation was carried out on the rafter-to-column connection behaviour by testing two different connection types. First connection plate in the rafter-to-column joint was composed of four layered thin plates between sigma sections and second connection plate used was a compact thin plate between double sigma section. Based upon extensive loading tests on rafter-column connections, analytical models of the semi-rigid connection behaviour are developed for both connection types. These models are then used in design calculations of the whole portal frame.

In this design study, interaction between rafters of the frame and sheeting panels of the pitched roof is also examined. This diaphragm action of roof sheeting has importance in stressed skin design of the frame because lateral deflection or drift of the frame is often as main serviceability criterion also basic criterion for the whole frame design, especially when light foundation is used i.e. semi-rigid column base connection of the portal frame has moderately low stiffness.
In the ultimate limit state design of the light-gauge steel portal frame, overall instability phenomena like torsional or torsional-flexural buckling and lateral-torsional buckling are predominant. This is especially the case when the compressed flanges of sigma sections are free. When these compressed flanges have lateral restraints near the connection region buckling resistance of the frame member is governed by the interaction between bending and axial compression. If the compressed free flange has closely spaced restraints then local buckling of the compressed flange is determining the design of the frame member.
In this study, design calculations are carried out for a typical case of the portal frame by checking resistances separately for rafters and columns of the frame and also for rafter-to-column connection. These calculations are entirely based on the newest Eurocode 3 (Ref. 1.) version.

2. RAFTER-TO-COLUMN AND RIDGE CONNECTION TESTS

2.1 Overview of test program

Loading tests carried out for rafter-to-column and ridge connections of the thin gauge portal frame were basically divided in three stages of test series. In the first test series (Ref. 2.), rafter-to-column connection was loaded with axial compressive load at the rafter end cross-section (Fig. 1) with a fixed loading angle while outer column end was jointed with a steel plate bolted between sigma sections and free edge of this plate was fixed with one bolt to the anchorage floor. Sigma section depth in all these tests was 300 mm and two wall thickneesses of 2,5 mm and 3,0 mm were used. Connection plate joining double sigma sections of the rafter and the column together with bolts was a compact steel plate and three plate thicknesses of 8 mm , 10 mm and 12 mm were used. Tie bar (double flat bar section) end was bolted in these tests with three bolts to innerside free edge at the middle part of the connection plate.

In the second test series, connection plate joining rafter and column ends was composed of four layered cold-formed steel plates (each of thickness of 2,5 mm). One test was first carried out with a rafter-column connection having this layered connection plate. In the three other tests, edges of the two outer layers of this connection plate were outwardly lipped at free innerside of the corner plate. In these tests, tie bar (double L -section) end was bolted to a separate fin plate between rafter sections above the corner connection plate. Section depth of 300 mm and two wall thicknesses of 2,5 mm and 3,0 mm were also used in these tests.

Third test series was a systematic testing sequence for seven rafter-to-column connections, for four ridge connections and for one column base connection. Parameters varied in these tests were loading angle at the rafter end , sigma section height (250 mm and 400 mm) and wall thickness (2,5 mm and 3,0 mm). Compact connection plate used was of a constant thickness of 12 mm. Tie bar used in these tests was a double flat bar section bolted to innerside edge in the middle of the corner connection plate.

2.2 Connection specimens and test set-up

Cold-formed steel grade used in rafter and column sigma sections was S 350GD+Z(EN 10 002-1) with nominal yield strength of 350 MPa. Structural steel grade of compact connection plate material was S 355JO with nominal yield strength of 355 MPa. This same steel grade was also used in flat tie bar sections. Strength grade of bolts used was 8.8 with yield strength of 640 MPa and ultimate strength of 800 MPa. Bolts were tightened to 200 Nm moment. Test set-up used in the first rafter-to-column connection test series is shown in Fig. 1. Double

sigma section (back-to-back) members (depth of 300 mm and wall thicknesses of 2.5 mm and 3 mm) of the column and rafter bolted together in the corner joint with a compact steel plate (thicknesses of 8, 10 and 12 mm) are in connection region and rafter center laterally restrained to move with three tubular beams bolted over the members to the anchorage floor. In the column end, a head plate (thickness of 12 mm) is bolted between sigma sections and the free end of the plate is bolted with a 36 mm diameter bolt through the anchorage floor of the testing hall. Tie bar (double angle section L 50x50x2.5) is bolted (three M 16 bolts) to the innerside edge of the corner plate. A short I-section is bolted between sigma sections at the rafter end and the web of I-section cutted in a certain angle (78 degrees) for the end plate taking perpendicular load of the hydraulic jack at the rafter end cross-section.

Test set-up used in the second series of connection tests was similar as described before. Connection plate used in these tests was a layered compound of four cold-formed steel plates. One test with a layered corner plate of 4 x 2.5 mm was first carried out . Three other tests with the two outer plates outwardly lipped (Fig. 2) were also carried out for two cases of layer plate thicknesses of 4 x 2.5 mm and 4 x 3.0 mm. Tie bar in these tests was connected to a fin plate bolted above the corner plate (Fig. 2).

Test set-up in the third test series was basically the same as before but in rafter-to-column connection three different loading angles were applied for the hydraulic jack at the rafter end for achieving different bending moment-normal force combinations in the connection members. One rafter-to-column connection (test A5) was loaded with a small angle causing opening bending moment at the right hand corner of the frame.

Four ridge connections with using two rafter sections of depths 250 mm and 400 mm (tests A3, B3, C3 and D3) were also tested. Three different loading angles were applied at the rafter end. One basic test (test A4) was also performed for the column base connection with a S400 sigma section of length of 1 m and clamped to the anchorage floor. Loading angle at the column end was 10 degrees to the column axis.

2.3 Measurements and calculation of forces and deformations

Connection loading tests were conducted as load controlled and at the ultimate loading stage load was first applied as load controlled and after a certain load it was switched to displacement control. Load capacity of the hydraulic jack was 500 kN and the force of jack was measured with a load cell having 0 - 350 kN measuring range. The tie bar force was measured with a load cell having measuring range of 0 - 200 kN. In all test series, stress-strain states in the connection plate and also in the lower flanges of sigma sections close to the joint region were measured with strain gauges using both rousette-type and normal gauges. Lateral displacements in connection members were measured with displacement transducers (LVDT) both in rafter and column part of the connection.

For determining the non-linear moment-rotation relationship for the connection, calculation model shown in Fig. 3 was used. Formula (1) in Fig. 3 gives bending moment in the joint with respect to applied force, loading angle, bending stiffnesses and lengths of rafter and

column. Last rotation term in the formula can be neglected in the moment calculation. For moment-rotation relationship of the connection, rotations are calculated from two measured (LVDT measurements) displacements (S3 and S4) between rafter and column end points and between bolt group centres on rafter and column sides of the connection plate.

3. RESULTS OF CONNECTION TESTS

3.1 Moment - rotation curves of rafter-to-column connections

Fig. 4 shows the results of the first test series (U1, U2, U3, U4 and U5) as non-dimensional moment - rotation curves. Sigma section S300 was used in rafters and columns with wall thicknesses of 2.5 mm (U1 and U2) and 3.0 mm (U3 and U4) and connection plate thicknesses of 8 mm (U2 and U3), 10 mm (U4) and 12 mm (U1). In test U5, a layered connection plate of four cold-formed plates 4 x 2.5 mm was used. For comparison, boundary lines of the rigid connection behaviour in cases of unbraced and braced frames according to the Eurocode 3 are also shown in Fig. 4.

Fig. 5 shows the results of the second test series (R1, R2 and R3) as moment - rotation curves. In test R1, wall thickness of S300 sigma section was 2.5 mm and in layered connection plate of 4 x 2,5 mm two outer layer edges were outwardly lipped (80 mm and 20 mm lips) at innerside of the corner plate. In tests R2 and R3 sigma section S300 wall thickness was 3 mm and four layer connection plate similarly lipped as before was of thickness of 4 x 3 mm. In all these tests, tie bar was bolted to a fin plate above the connection plate corner.

Moment - rotation curves measured in the third test series for rafter-to-column connections (A1, A2, A5, B1, C1, D1 and D2) are shown in Figs. 6 - 12. Rotation for these curves in determined in two ways i.e. on the basis of displacement (S3) measured between loading point and column end point or between bolt group centres (S4) at rafter and column side edges in the corner plate.

3.2 Ridge and column base connection test results

Ridge connection in the portal was tested in four tests (A3, B3, C3 and D3) with two sigma section depths of 250 mm (C3 and D3) and 400 mm (A3 and B3) and wall thicknesses of 2.5 mm (B3 and D3) and of 3.0 mm (A3 and C3). Figs. 13 - 16 show the measured moment - rotation curves (based both on displacement S3 and S4) of these tests. In test D3, loading angle was chosen to cause opening (negative) moment in the ridge connection. Moment - rotation curve measured in test A4 for column base connection is shown in Fig. 17 where rotation is determined on the basis of measured displacement S1 perpendicular to the column axis.

4. FRAME DESIGN CASE STUDY

4.1 Portal frame dimensions and loadings

Dimensions of the light-gauge steel portal frame studied are shown in Fig. 18. Spacing of these 12 m span portal frames is 4 m. Both rafter and column members of the frame are of sigma section S 300 with nominal wall thickness of 3.0 mm. Both rafter and column members have spacer plates between double sigma section with 1 m spacing. Tie bar is a double L-section of 50 x 50 x 2.5. Connection plate thickness both in corner and ridge connections is 12 mm. Spacing of the roof purlins is 1.5 m.

Unfactored loadings of the frame are as follows: Dead load 0.4 kN/m2, wind load 0.5 kN/m2 and snow load 1.8 kN/m2. Load factors of 1.35 and 1.5 are applied according to Eurocode 1 for permanent loads and imposed loads, respectively. Fig. 18 shows normal force, shear force and bending moment distributions in frame at the critical loading condition.

4.2 Serviceability and ultimate limit state design results

Serviceability limit of horizontal drift of the frame was calculated for the critical loading case as 14 mm which is well under the limit value of 30 mm (H/150) according to Eurocode 3. For vertical displacement of the ridge, a value of 12 mm was calculated and this value is also under the limit value of 60 mm (L/200) of Eurocode 3.

All the basic (axial and bending) capacity values for rafter and column members of the frame were first determined according to Eurocode 3, including also checks for combined bending and axial force. In the ultimate limit state of the thin-gauge steel portal frame, overall instability phenomena like torsional and torsional-flexural buckling and lateral-torsional buckling were also considered. Especially the case of free compressed flanges in members was checked in respect to local buckling between spacer plates. When compressed flanges of frame members have lateral restraints (e.g. hat-shaped purlins) near the connection region buckling resistance of the member is governed by the interaction between bending and axial compression.

Calculated utilization or loading rates of the frame members in this design study are as follows. First, as use of load capacity rafter had 56%, corner connection 56% and column 46% rate. In case of free compressed flanges, buckling resistance of rafter and connection were of 66% and column of 54% utilized. For interaction resistance of bending and axial compression, rafter had 74%, connection 77% and column 62% rate. For this interaction covered by lateral-torsional buckling, rafter had 70%, connection 95% and column 62% rate of utilization.

ACKNOWLEDGEMENTS

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APPENDIX I - REFERENCES

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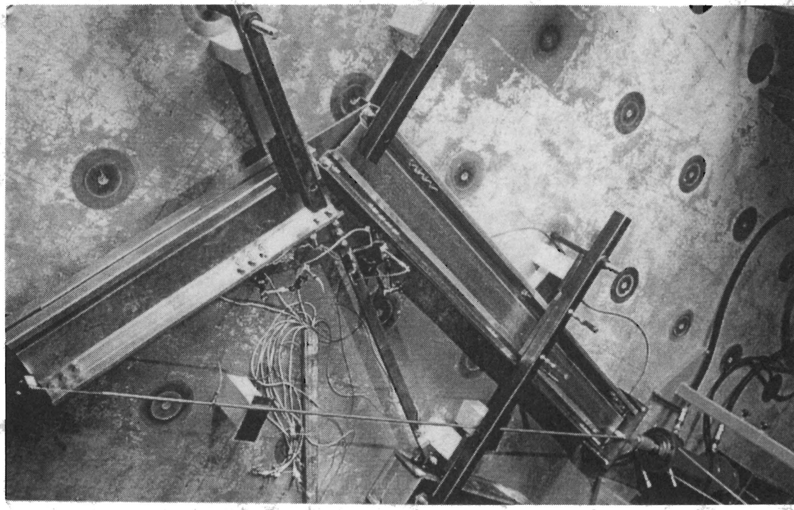


Fig. 1. Arrangements in the rafter-to-column connection tests.

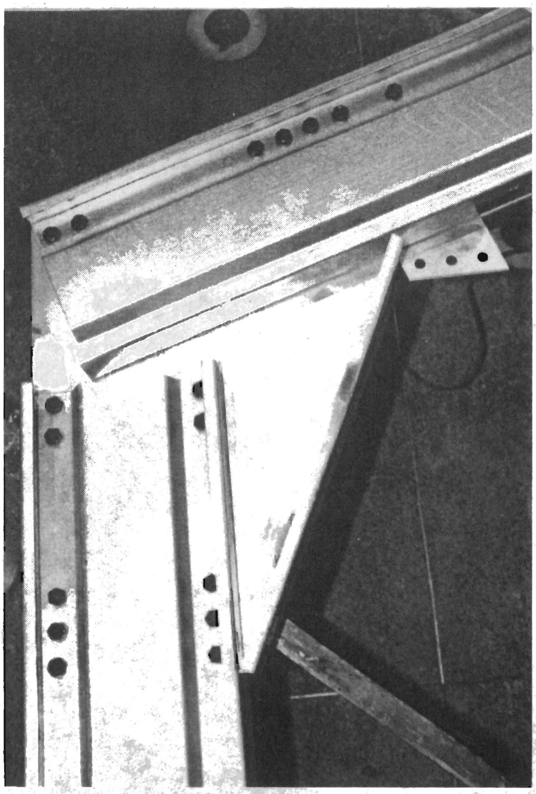


Fig. 2. Rafter-to-column connection with outwardly lipped edges at innerside of the connection plate and fin plate for tie bar above the plate corner.

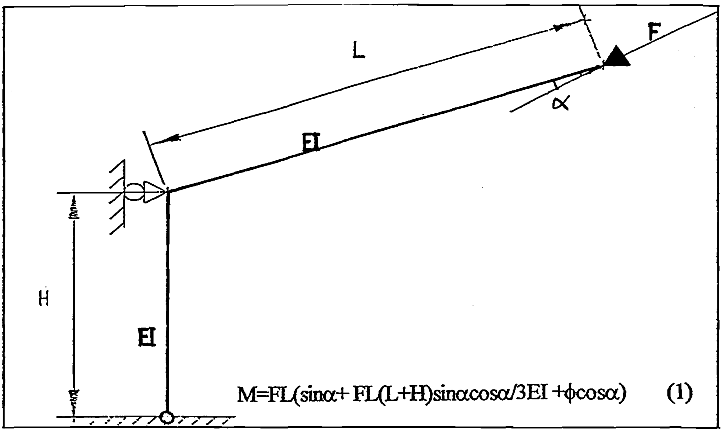


Fig. 3. Calculation model for determining bending moment in the corner joint.

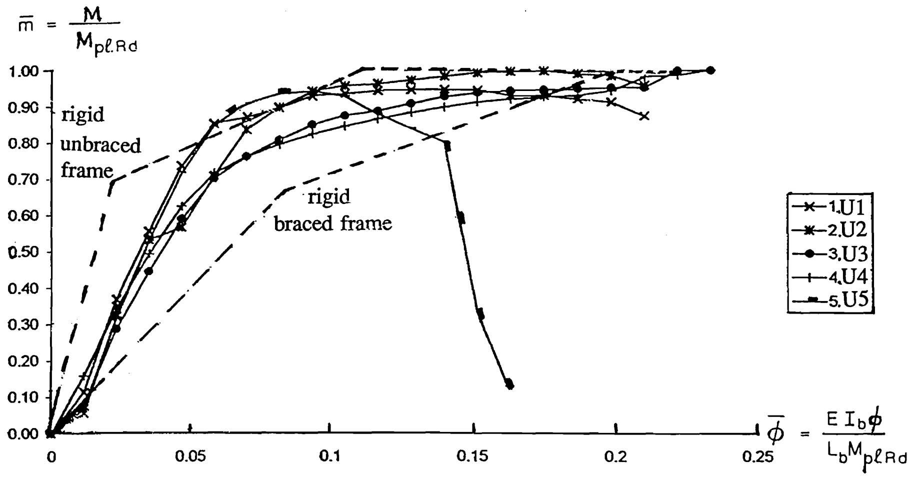


Fig. 4. Non-dimensional moment-rotation curves of rafter-to-column connection tests U1 -U5 with boundary lines of the rigid connection behaviour in cases of unbraced and braced frames.

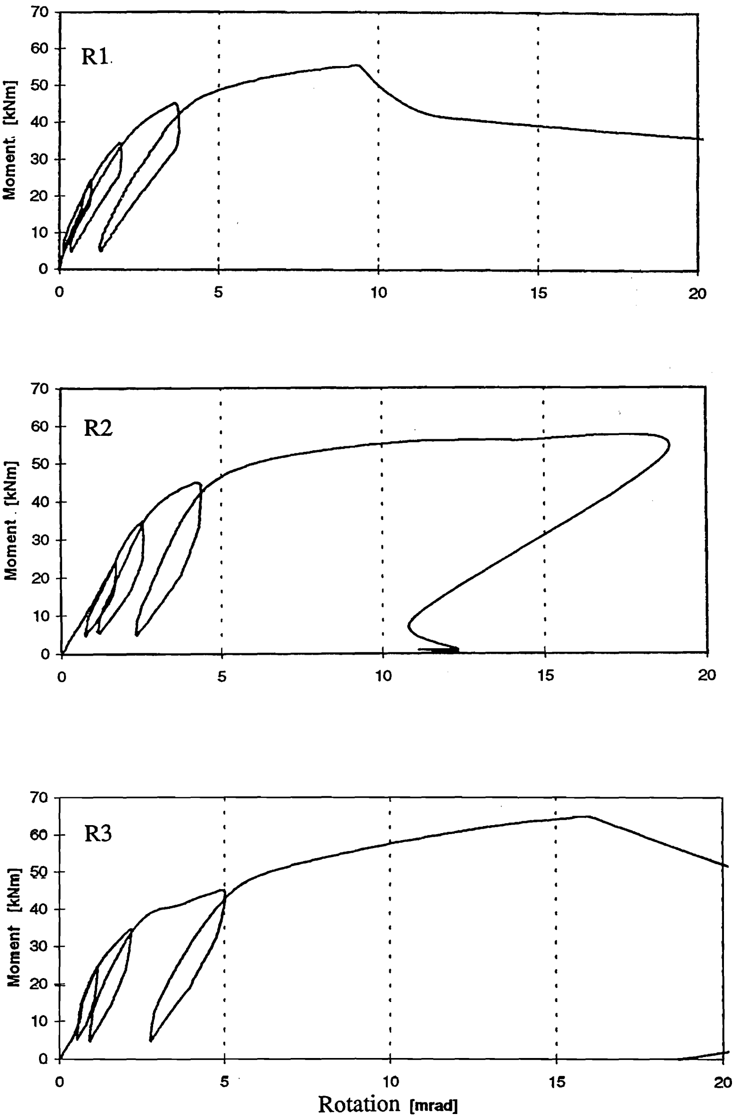


Fig. 5. Moment-rotation curves of rafter-to-column connection tests R1 - R3.

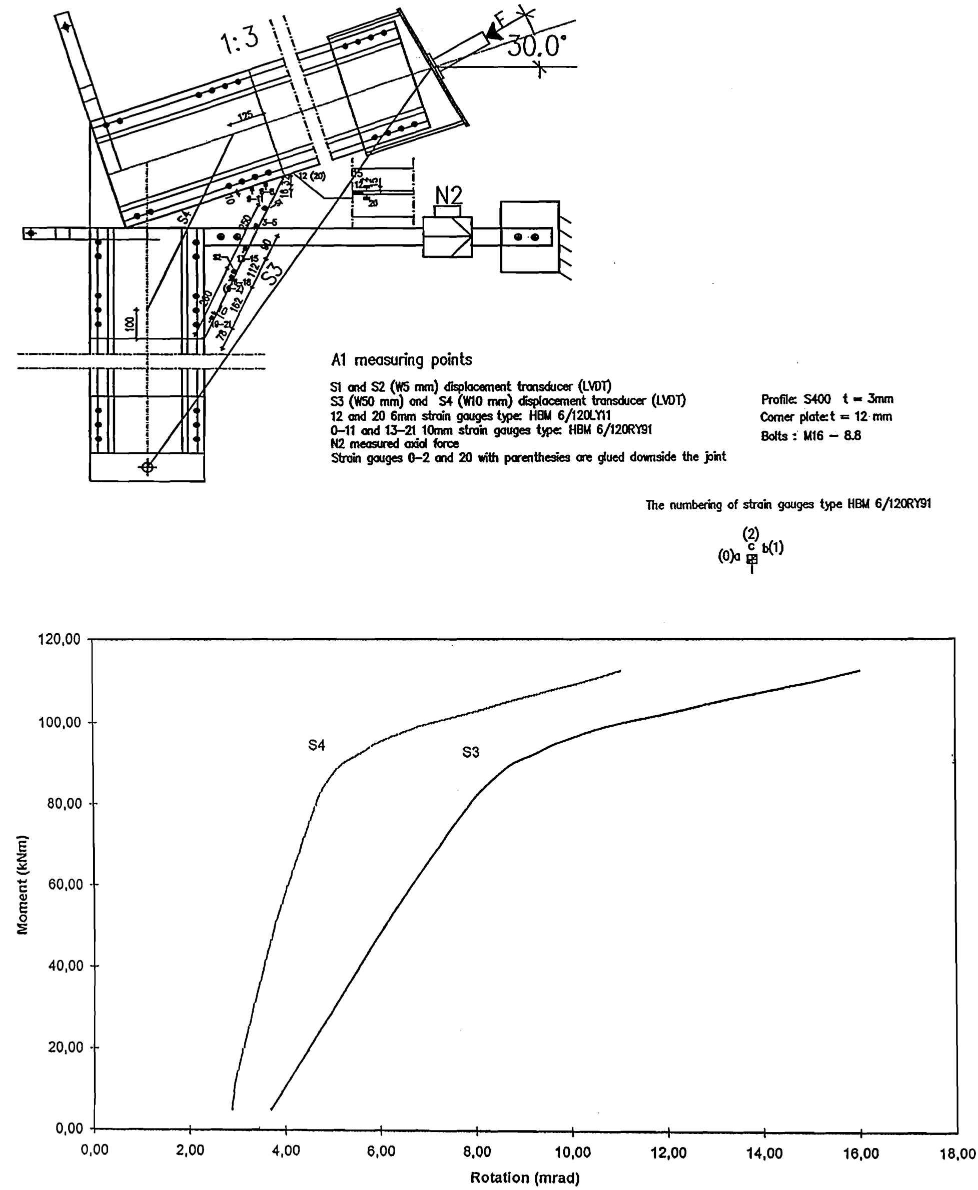
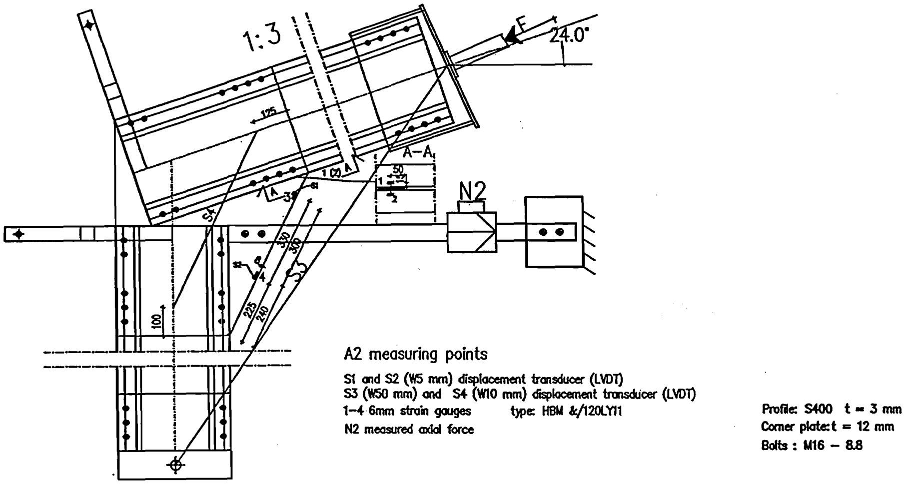


Fig. 6. Moment-rotation curve of rafter-to-column connection test A1.



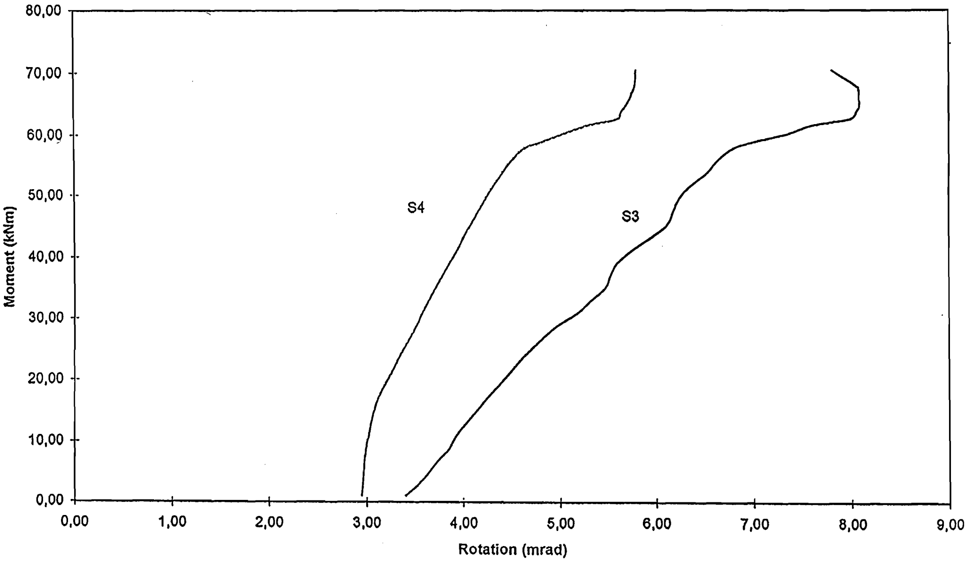
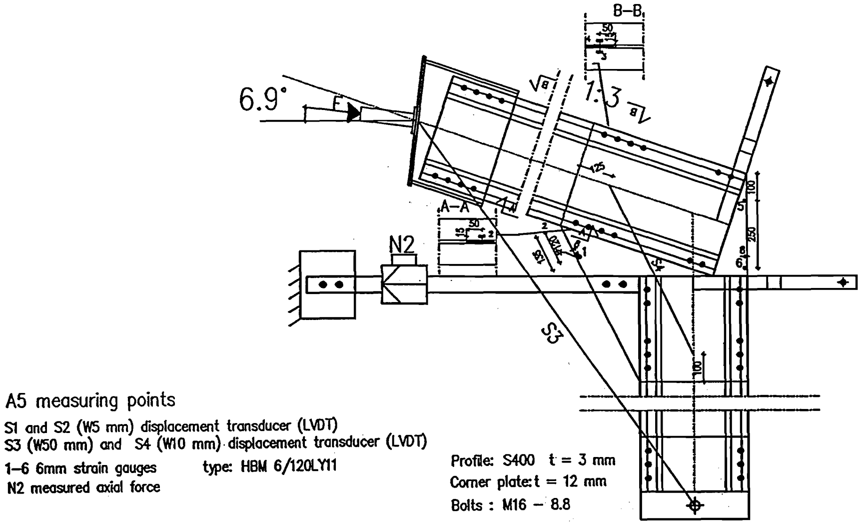


Fig. 7. Moment-rotation curve of rafter-to-column connection test A2.



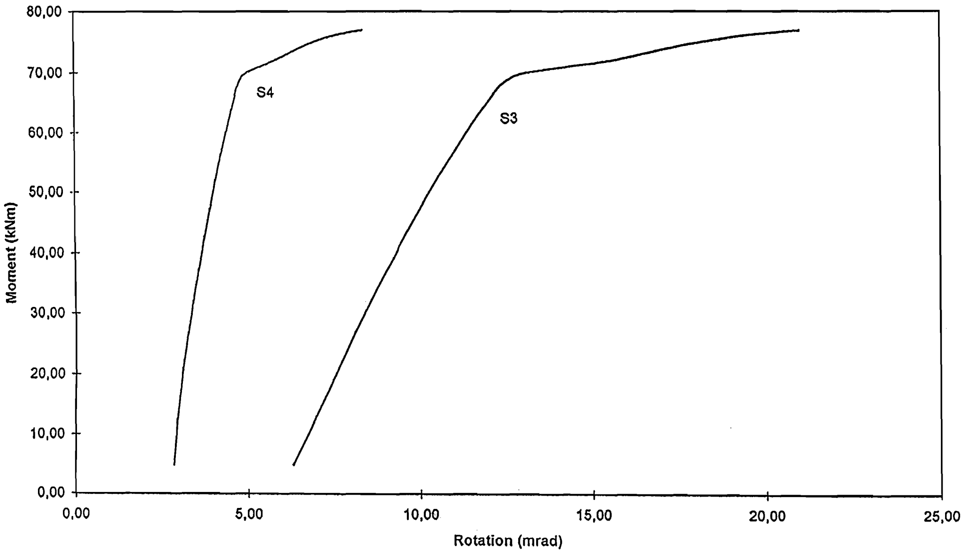
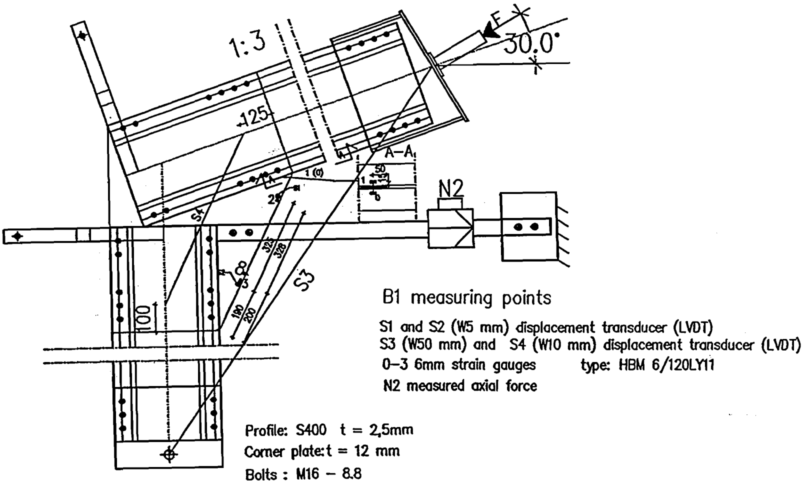


Fig. 8. Moment-rotation curve of rafter-to-column connection test A5.



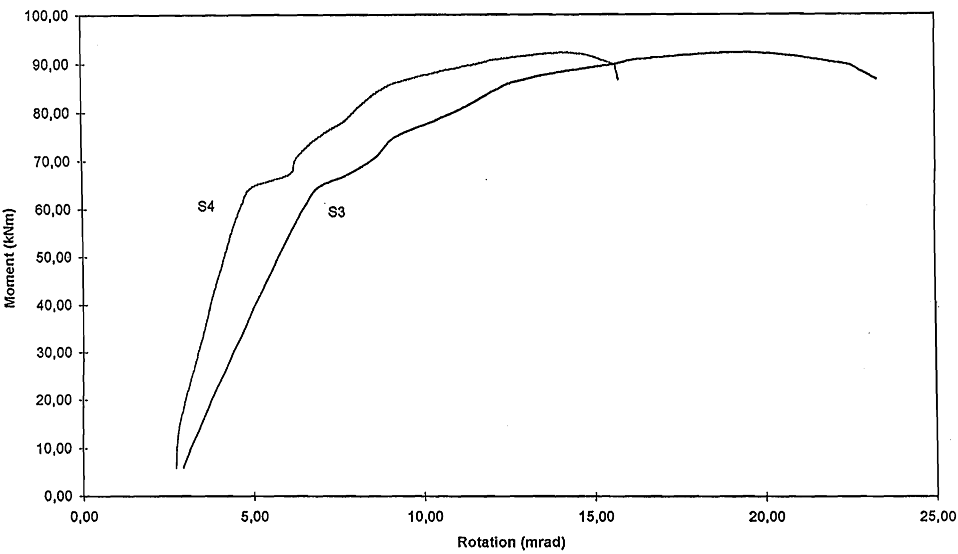
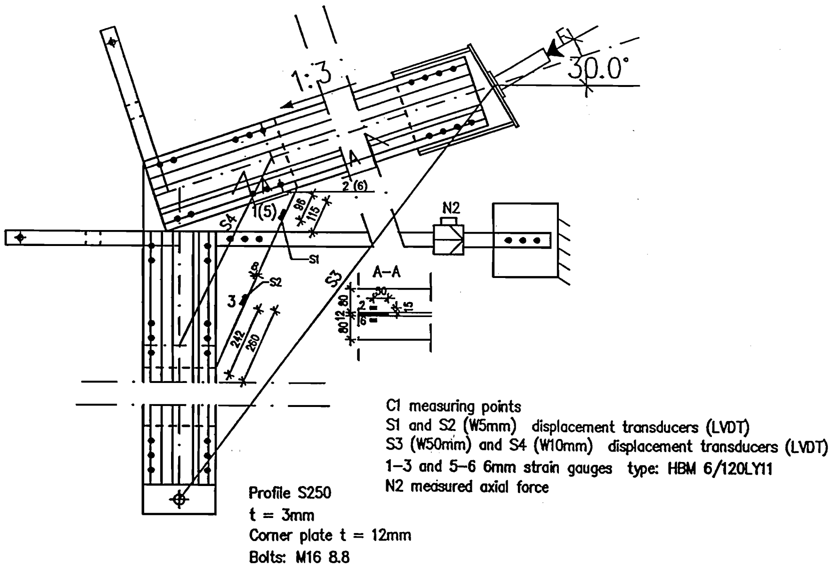


Fig. 9. Moment-rotation curve of rafter-to-column connection test B1.



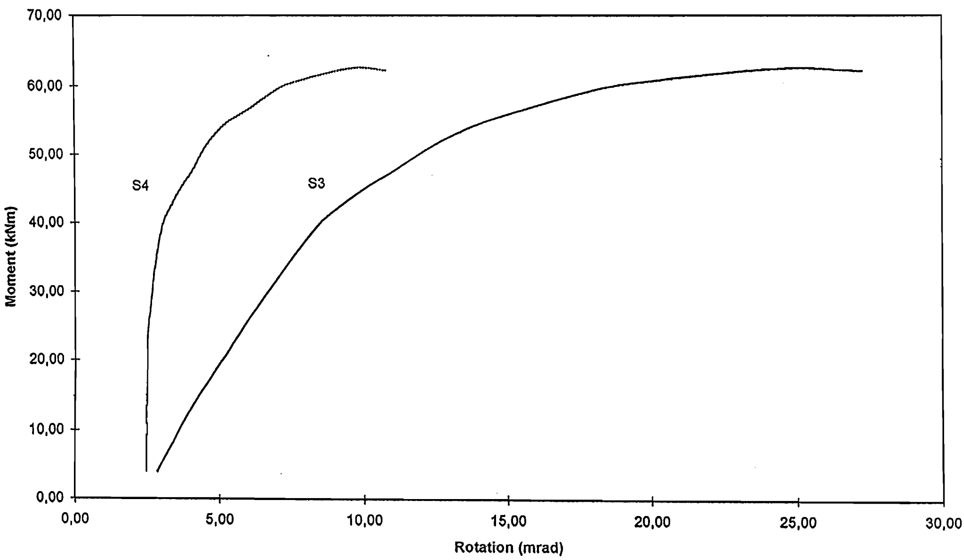
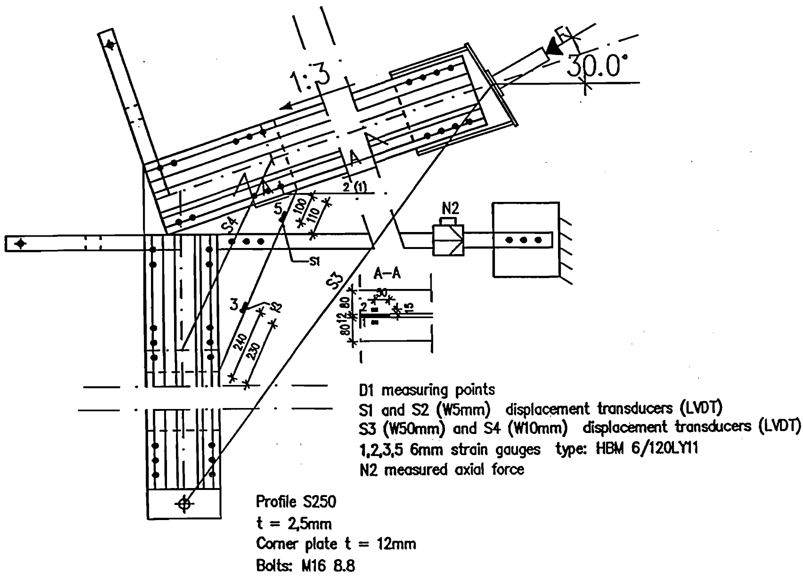


Fig. 10. Moment-rotation curve of rafter-to-column connection test C1.



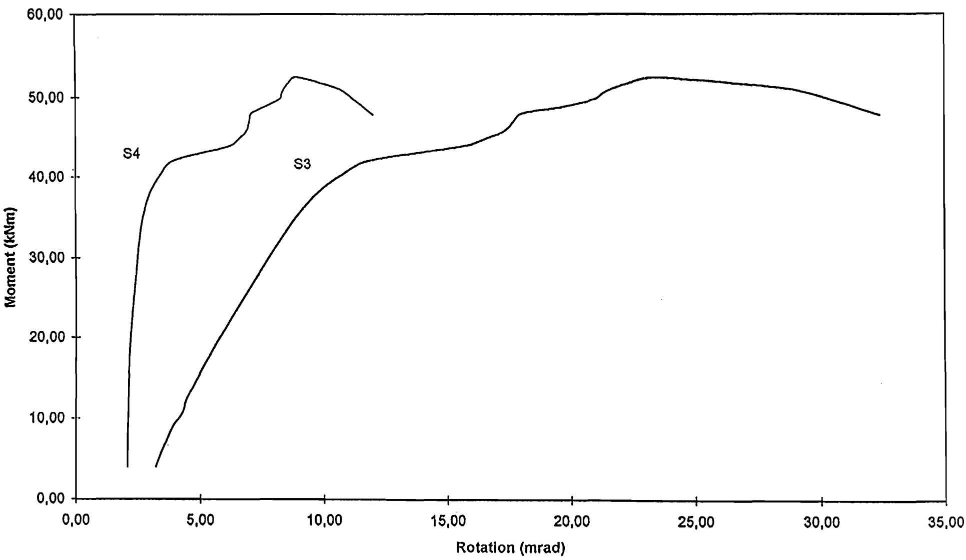


Fig. 11. Moment-rotation curve of rafter-to-column connection test D1.

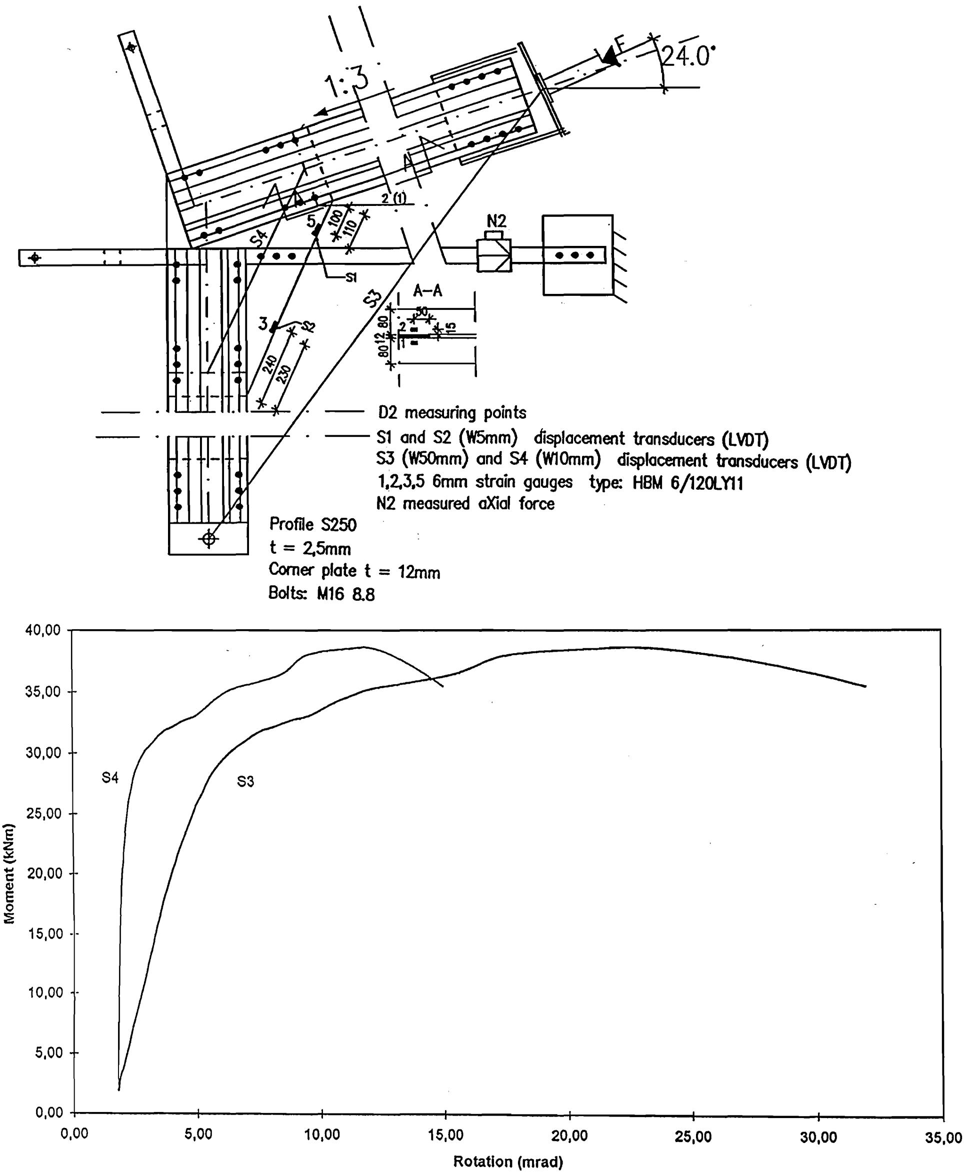
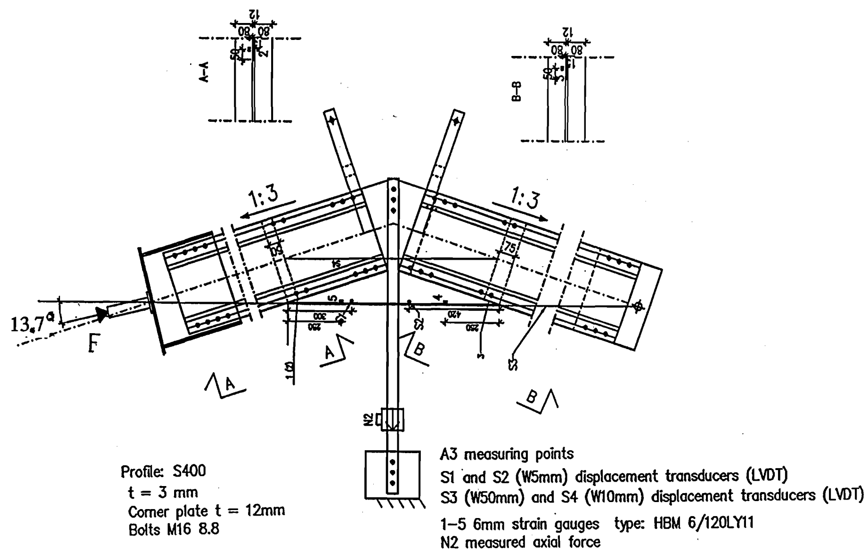


Fig. 12. Moment-rotation curve of rafter-to-column connection test D2.



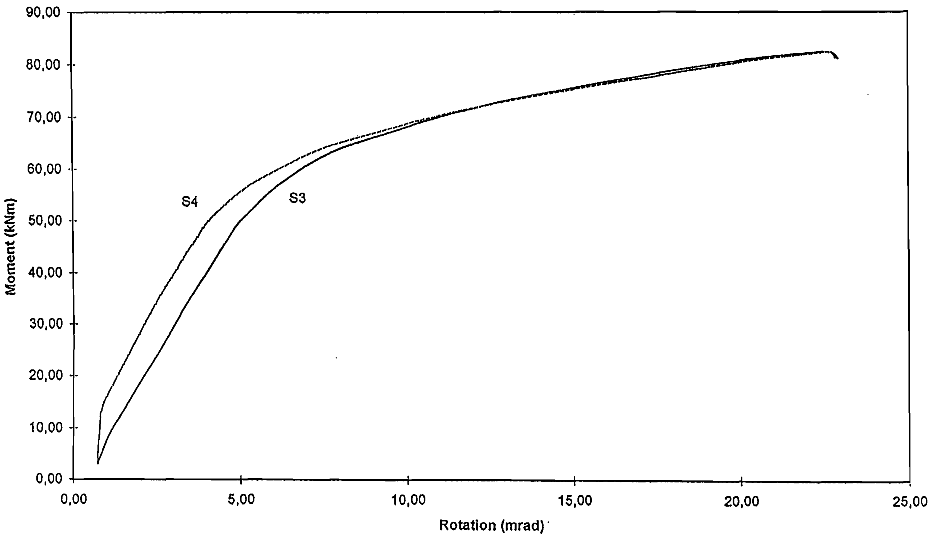
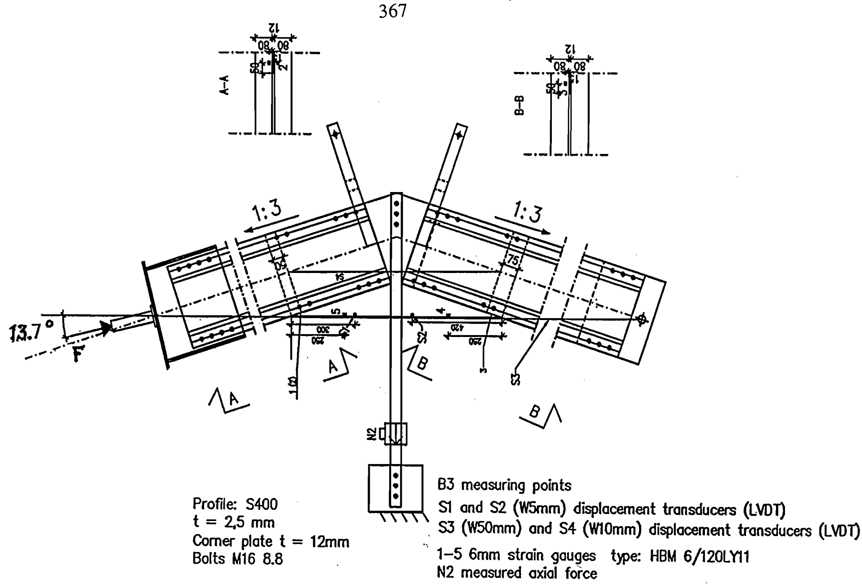


Fig. 13. Moment-rotation curve of ridge connection test A3.



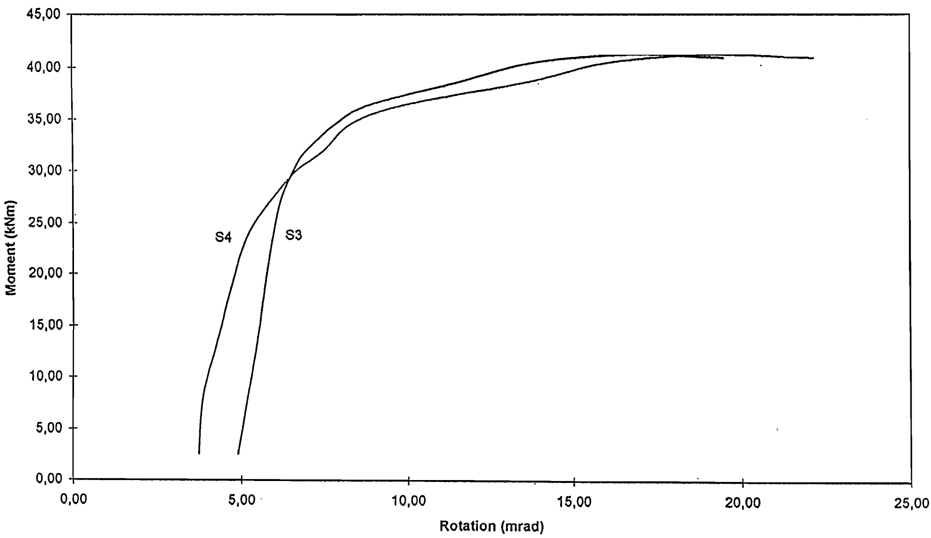


Fig. 14. Moment-rotation curve of ridge connection test B3.

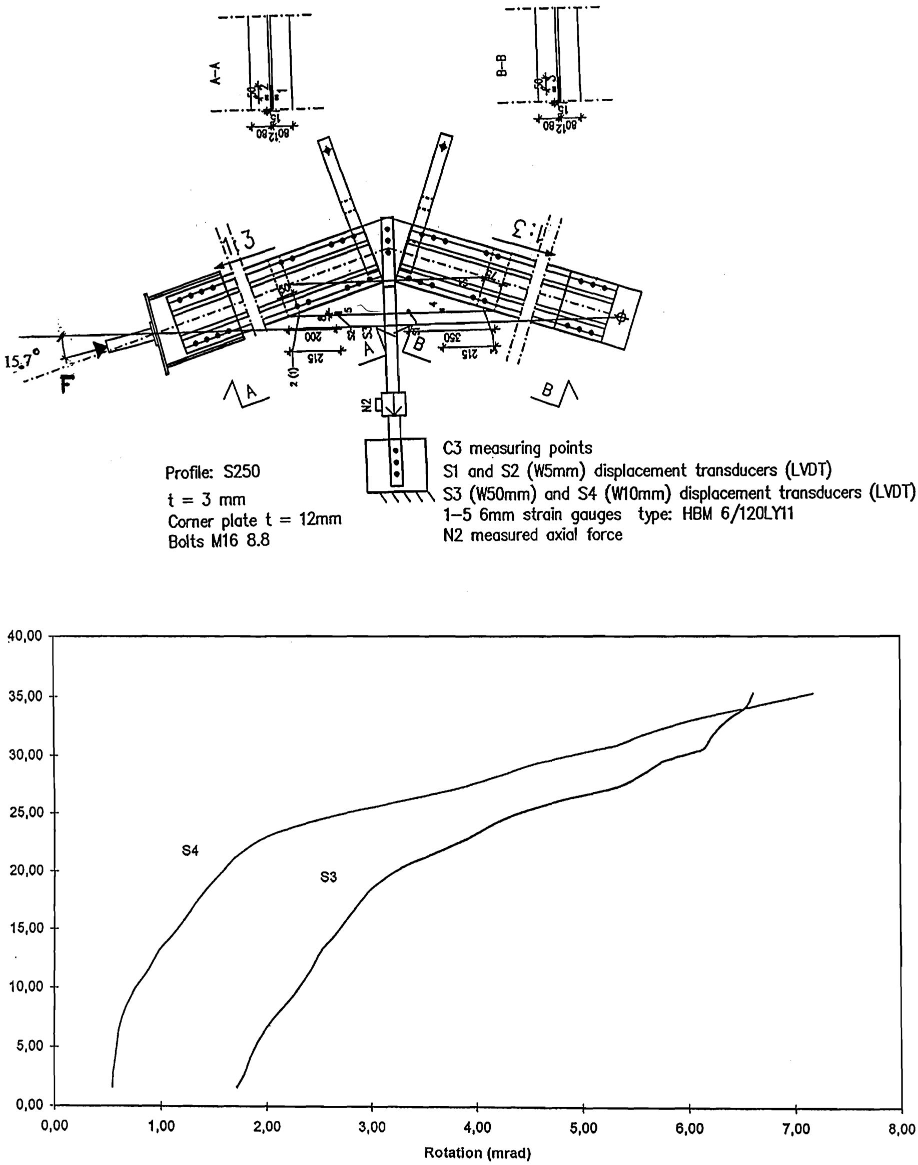
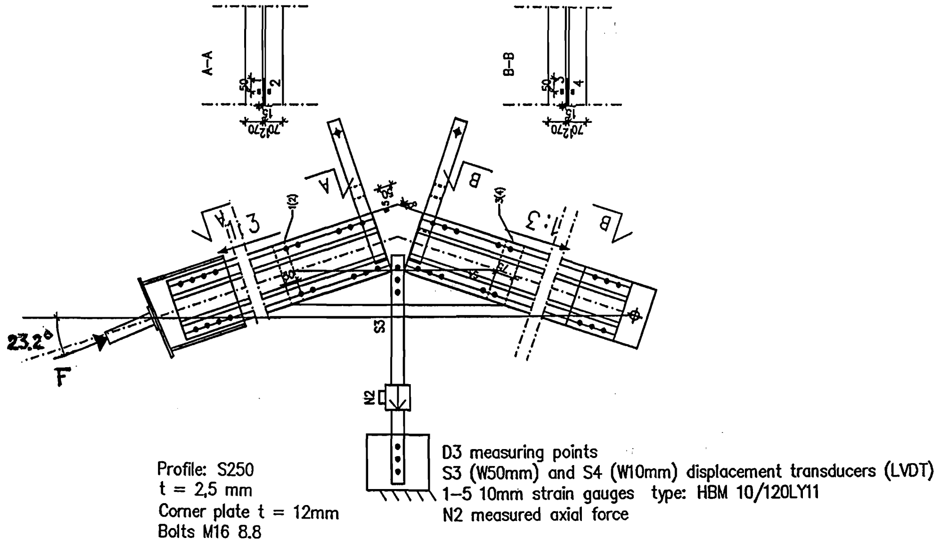


Fig. 15. Moment-rotation curve of ridge connection test C3.



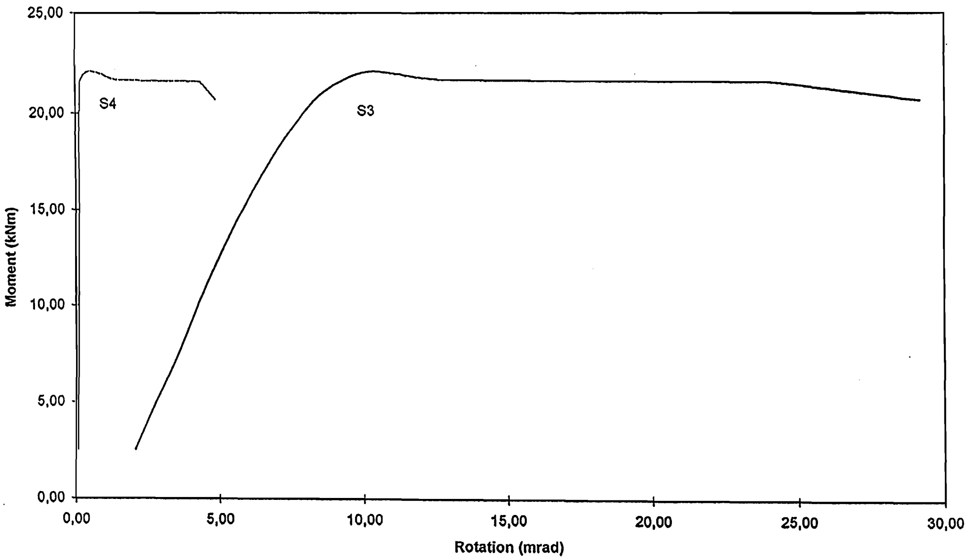
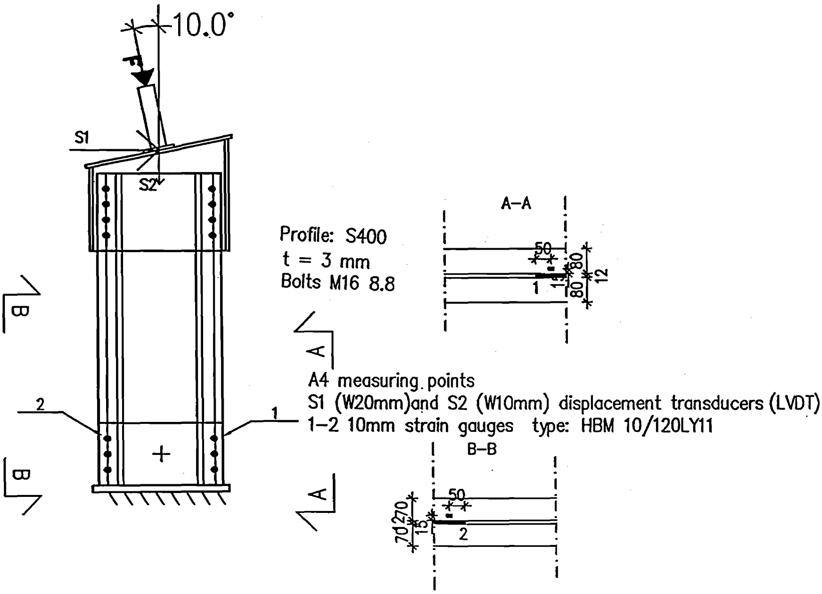


Fig. 16. Moment-rotation curve of ridge connection test D3.



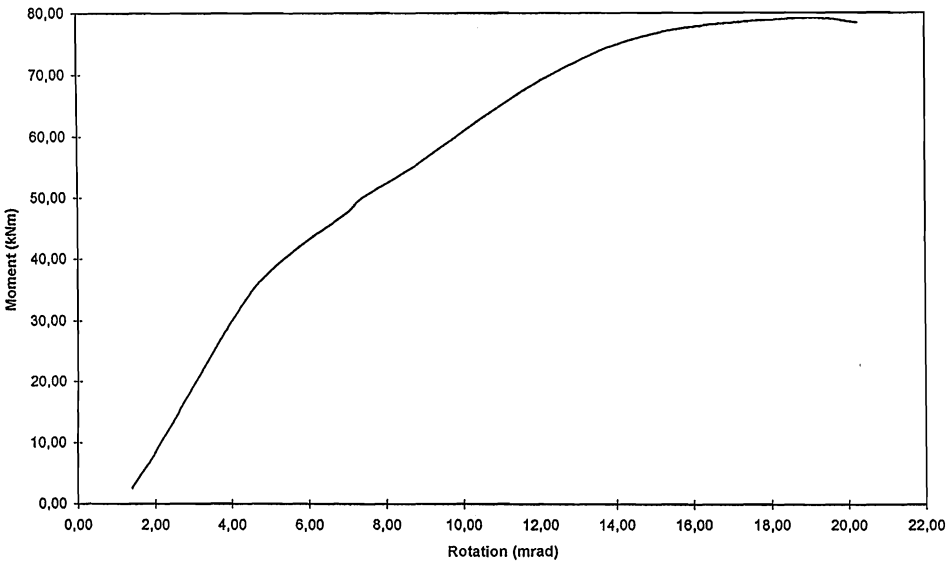


Fig. 17. Moment-rotation curve of column base connection test A4.

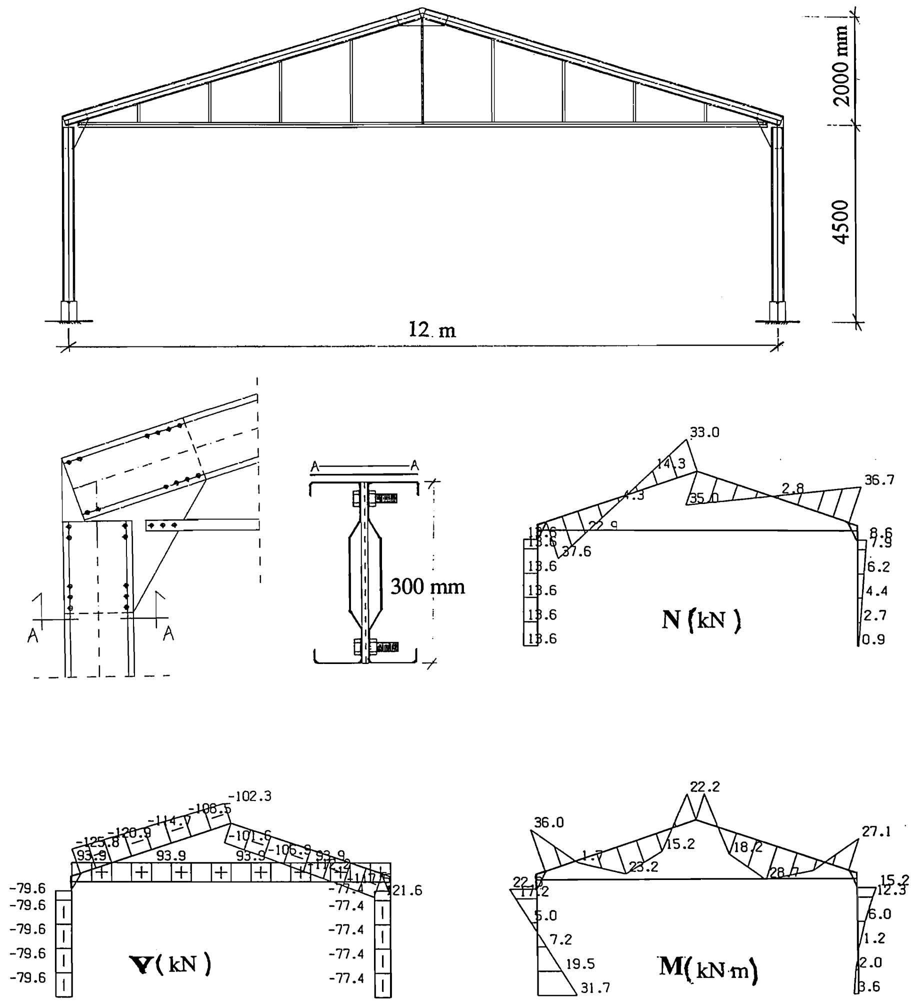


Fig. 18. Steel portal frame of design case study with normal force, shear force and bending moment distributions.