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On code formulation, testing and computer simulation of cold-formed thin-sheet steel arches

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Abstract

The aim of this paper is to gain insight on the mechanical behaviour of cold-formed thin-sheet steel arch structures subjected to gravitatory loads. The maximum vertical load that an arch of such characteristics is capable to withstand was computed. This was done by two different methodologies. The first one was analytical, for which two different finite element models of the arch were simulated. Effective properties of the cold-formed thin-sheet sections were obtained according to different structural codes. The second consisted on the mechanical testing of 10-13.3 m-span arches until failure occurred. Analytical and experimental results were compared.

Keywords: arch, cold-formed steel, finite element analysis, mechanical testing, light structures, structural codes, class 4 sections.

1. Introduction

Cold-formed thin-sheet steel arches are usually employed as light rooftops, mostly in industrial buildings. These elements must satisfy the requirements of separation and load-bearing function, withstanding exterior loads such as wind or snow.

The study of the mechanical behaviour of curved elements as arches is complex. Among other considerations, the curved geometry generates high horizontal reactions even when the arch is subjected to vertical loads, so the supports must be capable to resist or transmit them correctly to the structural elements where the arch is attached (Cudós, [1]). In spite of this, arches are able to provide higher span lengths than other "classic" structural solutions. This is due to the intrinsic, curved geometry of the arch, specially the key height/span length ratio. Moreover, this feature can be enhanced if the arch weight is lowered. This is the reason why cold-formed thin-sheet steel arches are employed.

The difficulties in the study of an arch rise if thin-sheet elements are used to build it. The reason is found in the features of its cross-section, which is denoted as *Class 4* according to Eurocode 3, part 1-3 (CEN [2]). Elements with Class 4 cross-section fail by local effects – such as flange curling or buckling – before yield stress is reached. This is the reason why reduced (or effective) dimensions of the cross-section have to be computed in order to take into account the diminished strength of elements due to the local effects. This must be done according to structural codes.

This paper summarizes the study of the maximum vertical loads that cold-formed thin-sheet steel arches are capable to withstand. This study was performed by the authors at the request of a manufacturer of such elements. Analytical calculations based on theoretical formulations and structural codes were carried out, for what arches were modelled and simulated by finite element analysis. On the other hand, several arches were tested as an attempt to validate the analytical models and check the load value obtained theoretically.

2. Code-based calculations

2.1. Arch geometry

Manufacturing process of arches is described as follows: a spool of rolled thin-sheet steel is put on a machine that rolls, hammers and curves the sheet at the same time. Rolling is needed to obtain the desired profile, or cross-section, of the arch. The number of hammerings is previously calculated to provide the desired curvature (Figure 1).

(a) (b)

Figure 1: (a) Superior and (b) inferior views of a cold-formed arch. Rolling process provides the crests and valleys in the sheeting. Hammering effect is seen in the folds.

The arch curve is circumferential, so its curvature is a function of the span length and the arch slope in the supports. In the present study, span lengths of the analyzed arches ranged from 10 to 13.3 meters. The slope in the supports corresponded to an angle of 25º for all cases.

2.2. Steel properties

Steel properties are provided by the manufacturer. In spite of arches come from different steel spools provided from a third-party, they are supposed to consist on the same steel type. Considering the values of the different spools, the following mean values were taken for all cases:

Tensile yield strength:	f_y= 275,8 N/mm^2
Compressive yield strength:	f_y= 248,2 N/mm^2
Ultimate strength:	f_u= 300,4 N/mm^2
Shearing yield stress:	f_v= 158,2 N/mm^2
Initial modulus of elasticity:	E_0 = 193.100 N/mm^2

2.3. Gross and effective cross-section properties

The cross-section of the arches in study is shown in Figure 2. The shape was always the same, although the thickness of the steel sheet varied from one arch to another.

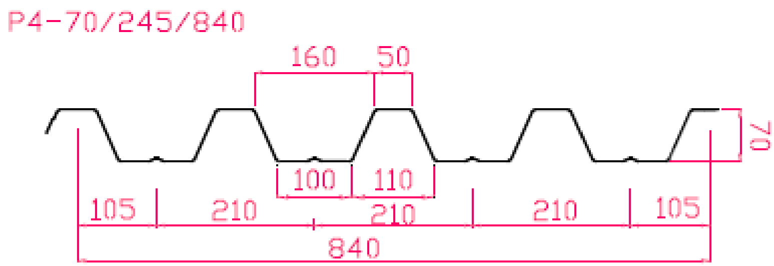

Figure 2: Shape of the cross-section of one tested arch (units in mm).

Cold-formed thin-sheet steel elements fail by local failure of the cross-section before yield strength is reached, mostly when it is subjected to compressive axial forces or negative bending moments. This effect must be taken into account when cross-section properties are computed.

This means that *effective properties* of the sections should be calculated from gross ones, the previous with lower values than the latter. Using the effective properties in the finite element model ensures that the strength of the section is diminished, and subsequently the local effects are considered.

In the present study, the effective properties of the sections were calculated following the prescriptions from (CEN [2]) and (Normenausschuss Bauwesen [3]), which are depicted in the table of Figure 3.

Thickness (mm)	Properties	Area (mm^2/m)	Inertia about flexural axis (mm^4/m)	Section modulus (mm^3/m)
0,8	Compression only	519	- - -	- - -
	Positive bending	1.085	778.881	18.404
	Negative bending	964	682.259	19.159
1,0	Compression only	776	- - -	- - -
	Positive bending	1.423	1.055.919	25.899
	Negative bending	1.256	907.436	24.498
1,2	Compression only	1061	- - -	- - -
	Positive bending	1.708	1.267.597	31.038
	Negative bending	1.555	1.136.383	29.834

Figure 3: Effective properties of the arch section as a function of sheet thickness.

3. Finite element modelling

2- and 3-dimensional finite element (FE) models of the arches were created from drafts provided by the manufacturer. FE analysis software SAP2000 8.15 (Computers and Structures, Berkeley, CA, USA) was used to perform the simulations.

3.1. 2-dimensional models

The arch was represented with 100 1-dimensional frame elements, approximating it to a polygonal line. There were assigned fixed pinned boundary conditions to the ending nodes of the model. The model was subjected to a unitary load distributed along its curve and in the direction of gravity (Figure 4), which use is explained in the next section.

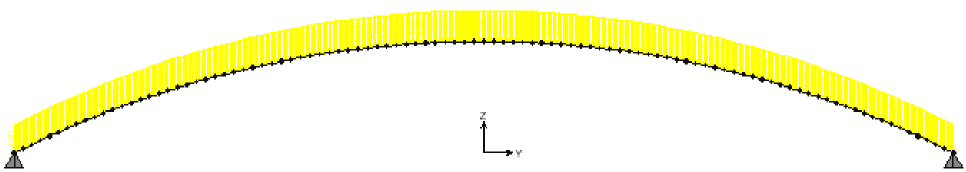

Figure 4: 2-dimensional model subjected to distributed load in the direction of gravity.

The model was submitted to a linear, elastic analysis, and the distribution of internal forces was obtained on each section of the model corresponding (Figure 5). The use of this type of analysis is justified because Class 4 cross-sections fail before yielding occurs.

(a)

(b)

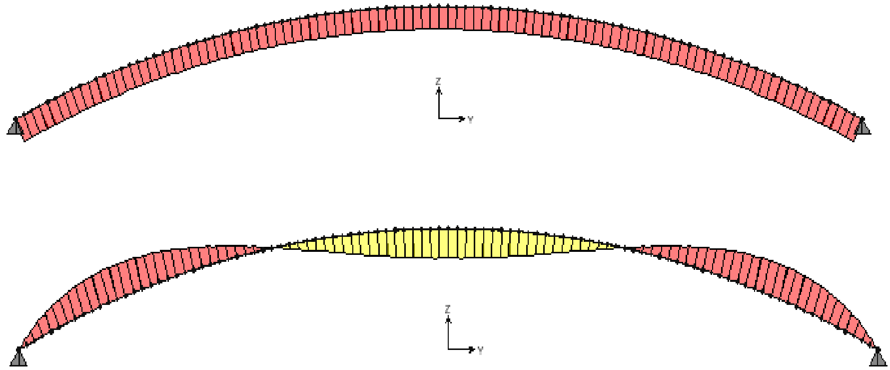

Figure 5: Internal forces diagrams in the 2-dimensional model. (a) Axial forces and (b) bending moments (yellow: positive; red: negative).

3.2. 3-dimensional models

These models were more geometrically accurate compared to the 2-dimensional ones. Each one had an average number of 2212 nodes and 2132 shell elements (Figure 6).

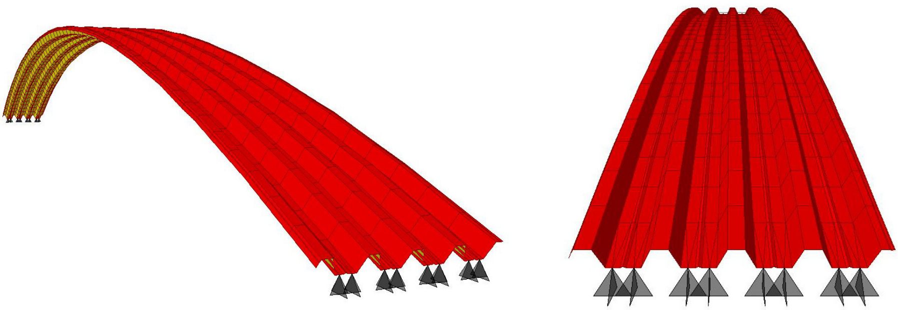

Figure 6: Views of the 3-dimensional FE model of the arch.

4. Design requirements

When an arch is subjected to gravitatory loads, it deforms and develops internal loads such compression, shearing, and bending moments, which can be positive or negative. A single section can experiment a single or a combination of this loads.

Structural codes provide expressions aimed to satisfy design requirements, in compliance with Ultimate and Serviceability Limit States. Willing to compute the maximum theoretical load an arch is able to support, the following strength conditions were checked, according to Eurocode 3, part 1-3 (CEN [2]): compression and/or bending, shear only and local transverse forces. This code doesn't consider the effect of buckling in arches, so provisions from DIN 18800 (Normenausschuss Bauwesen, [3]) were also considered.

Regarding Serviceability Limit States, it was stated that the limit condition for the deflection of the key of the arch was *L/200*, being *L* the span length. Considering linear and elastic regime in the FE model, the theoretical maximal load was the multiplying factor of the unitary load for which the levels of internal loads reached the most restrictive Limit State. The failure load corresponded to arch buckling was the most restrictive.

5. Mechanical testing

The manufacturer of the arch provided the site, personnel, specimens and equipment to perform the mechanical tests. Firstly, two steel hollow tubes were rigidly attached to the concrete slab of the ground of the testing area with HILTI® connectors, on which arch ends would be bolted. This was made in order to mimic the real situation of assembly of an arch. Distance between both supports was equal to the span length. A gantry crane was used to place the arch on these supports (Figure 7).

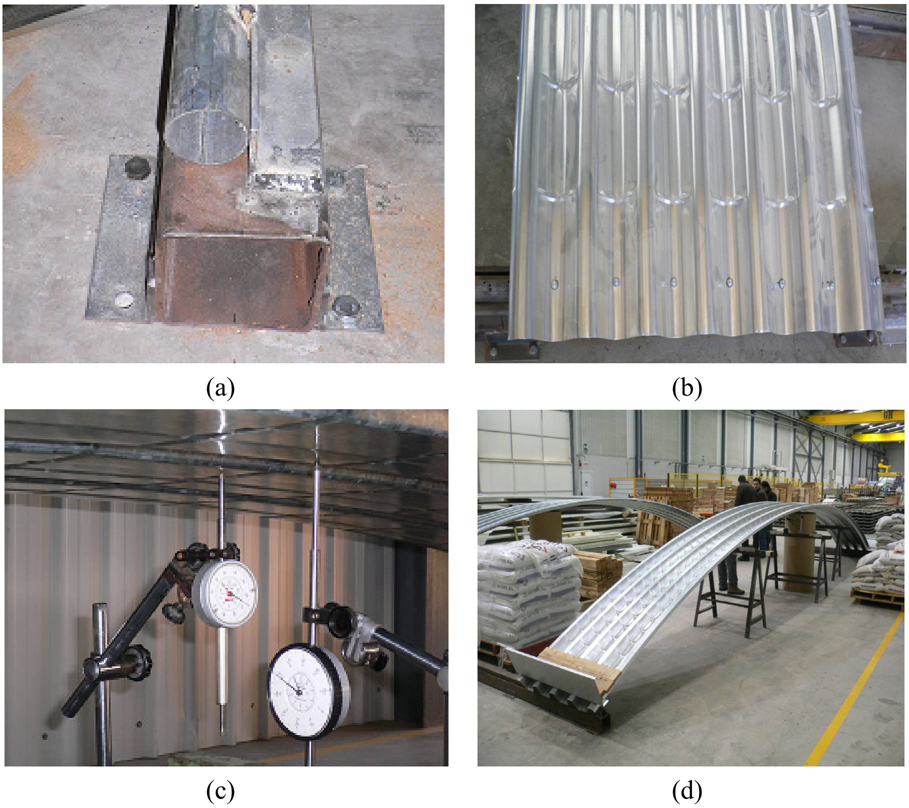

Figure 7: (a) Arch support. (b) Bolted support. (c) Dial indicator with directional feeler, used to control key deflection. (d) First stage of arch loading (the inferior elements were for personal security).

Load protocol consisted on placing an entire layer of wooden bars over the top surface of the arch. The bars were placed two at once symmetrically, starting from both ends and finishing on the key, and placed adjoining one to another to create a distributed load.

When the first layer was complete, and the arch reached again an equilibrium situation, the deflection of the key of the arch was measured using a dial indicator with directional feeler. This process was repeated by superimposing layers of wooden bars (Figure 8) until failure occurred (Figure 9). The last layer had half the number of wooden bars than the previous one, in order to obtain the most accurately possible value of the total weight which causes the failure.

Figure 8: Two stages of arch loading.

Figure 9: Arch failure by buckling.

The entire process was carried out carefully, ensuring that the arch was always subjected to static forces, and avoiding impacts that would result in undesirable effects on the structure.

A total of 45 arches were tested. For several reasons (i.e., unexpected failure, bad manufacturing process…) 16 tests were rejected.

5. Comparison and discussion of results

Failure loads obtained by analytical methods are shown in the table of Figure 10. Formulations from the work of Cudós (Cudós, [1]) were also used to contrast the code-based and FE model results. It can be seen from these values that FE results and those from Cudós theory do not differ too much. This leads to think that the FE models have been properly constructed, taking into account that linear and elastic analyses have been performed.

Thickness (mm)	Analytical method	Span length (m)				
		9	10	11	12	13,3
0,8	Code-based	252,6	190,1	146,8	115,4	86,8
	2-D FE Model	805,2	571,8	423,9	339,6	241,1
	3-D FE Model	834,5	610,0	461,7	352,7	256,4
	Theoretical [1]	769,9	579,0	448,9	333,6	243,4
1,0	Code-based	385,4	290,2	224,3	176,4	132,7
	2-D FE Model	1078,2	773,3	579,4	460,8	328,4
	3-D FE Model	1054,3	770,0	580,8	446,4	326,5
	Theoretical [1]	1036,4	779,0	603,9	449,0	328,6
1,2	Code-based	535,5	403,5	312,0	245,5	184,8
	2-D FE Model	1346,3	965,8	724,0	576,2	411,6
	3-D FE Model	1272,2	928,0	699,4	537,0	392,4
	Theoretical [1]	1366,5	974,0	689,1	512,2	443,5

Figure 10: Failure loads (daN/m^2) predicted with different analytical methods.

On the other hand, the results of the admitted tests are summarized in the table of Figure 11. Fold depth is introduced in order to illustrate that the manufacturing process plays a crucial role in the mechanical behaviour of an arch. In fact, it was observed that folds introduced by hammering manage to weaken the arch. Moreover, failure always initiates in the folds.

Comparisons between analytical and testing methods are shown in the graphs of Figure 12. The differences between maximum loads obtained by both methods are due to the difficulties arisen in mimicking the real test conditions in the analytical models. These are magnified as a consequence from the manufacturing process. Nevertheless, the results obtained from code-based formulations were close to the results from mechanical tests. The reason is that such formulations are based on tests, and local effects which cause failure are considered properly compared to the analytical models.

Test	Thickness (mm)	Span length (m)	Fold depth (mm)	Failure load (daN/m^2)
1	0,8	10	7	186
2	0,8	10	7	205
3	0,8	10	7	225
5	0,8	10	4,5	280
7	0,8	10	4,5	172
9	0,8	10	4,5	168
11	0,8	12	7	120
12	0,8	12	7	114
13	0,8	12	7	116
14	1	12	4	212
15	1	12	4	217
16	1	12	4	221
17	1	12	4	227
22	1	13,3	5	116
24	1	13,3	5	117
25	1	13,3	4	187
26	1	13,3	4	169
27	1	13,3	4	161
28	1,2	12	3,5	261
29	1,2	12	3,5	321
30	1,2	12	3,5	351
31	1,2	12	3,5	340
32	1,2	12	3,5	350
33	1,2	13,3	3,5	158
34	1,2	13,3	3,5	179
37	1,2	13,3	3,5	175
38	1,2	13,3	3,5	146
39	1,2	13,3	3,5	239
42	1,2	13,3	2,5	264

Figure 11: Test results. Missing test numbers correspond to rejected tests. Note that the manufacturing process doesn't cover all the arch combinations span length-thickness.

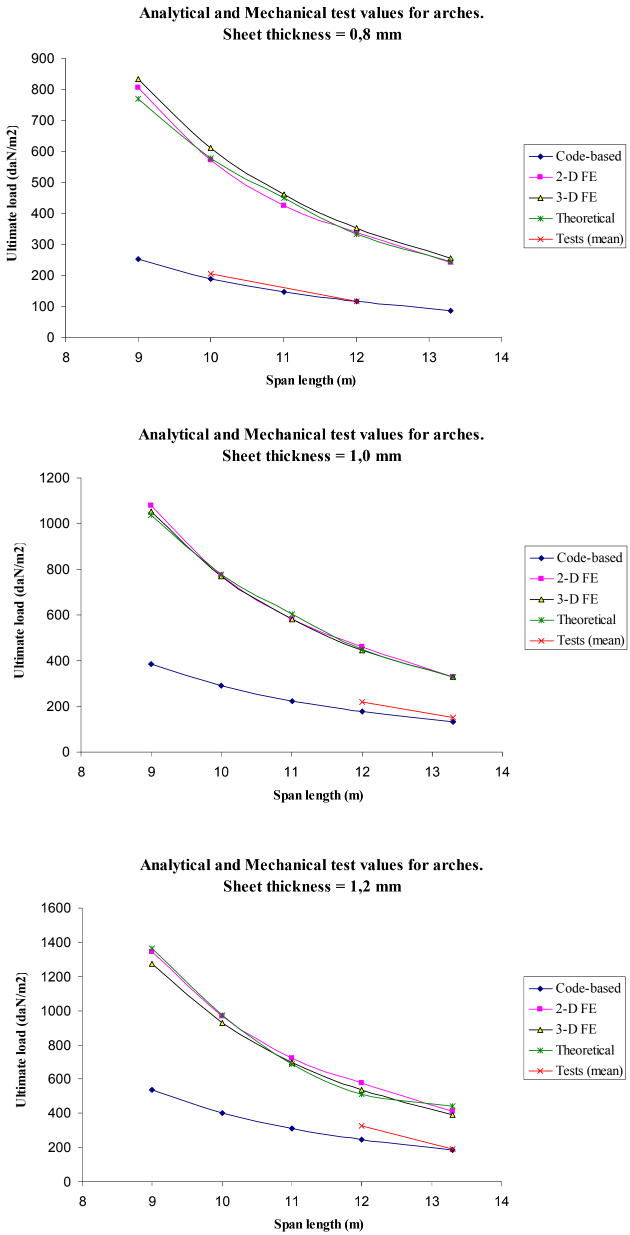

Figure 12: Comparisons between analytical and testing methods.

6. Conclusions

In the present paper, the mechanical performance of cold-formed thin-sheet steel arches was studied. The maximum load an arch is able to withstand was obtained from two different approaches. The first one was analytical, for which two FE models of the arch were constructed. The second method consisted on mechanical tests of arches subjected to gravitatory loads until failure occurred.

To the knowledge of the authors, it is virtually impossible to test mechanically wind loads on the arch (which act perpendicularly to its curve), so mechanical tests must be limited to gravitatory loads. Cold-formed thin-sheet steel arches are light structures, and they are very vulnerable to wind effect, so the results obtained in this work have to be interpreted carefully by the manufacturers of these elements.

Regarding results obtained for gravitatory loads, there were some important differences obtained with both methodologies that must be taken into account. The first one was found in the geometry of the modelled arch compared to the real one. The effects of manufacturing had not been considered in the "ideal" geometry of the FE models, such as the residual stressed in the steel sheet due to hammering and the differences between the ideal and the real arch curves. Another crucial aspect was the placement of an arch on its supports in the mechanical tests. In order to maintain the theoretical ratio height/span length before loading, the arch was somehow "forced" while it was bolted at its ends. This generated a counter-deflection not considered in the theoretical models. Mechanical behaviour of arches is very sensitive to small variations of such features, so they must be taken into account in further FE-based studies.

In a similar way, real support conditions should be considered more accurately than the ideal pinned conditions imposed in the FE model. The fact is that during mechanical tests, steel sheet crushes against the bolts, and subsequently a progressive tearing effect is observed (Figure 13).

Figure 13: Tears caused by crushing of the steel sheet against the bolts in the supports.

Tearing effect is not negligible because it implies a change in the stiffness of the union. The initial slope of the arch in the supports changes, and so the ratio height/span length does. Subsequently, internal loads are modified with regard to the theoretical values, originating remarkable differences between the predicted and the tested maximum loads. This effect can be amplified in the practice, leading to unexpected results. Arches are usually placed *in situ* with fewer bolts than the used in mechanical tests, depending on the installer technicians. Manufacturers should specify the necessary number of bolts in their catalogues.

Concerning the prescriptions from structural codes, the authors have considered several ones in order to cover properly the mechanical performance of the arches. Formulations from different codes are mostly based on tests, which are performed in different organisms in different countries. This may be another source of errors. Moreover, while section properties are different for positive and negative bending moments, it is not easy to know the extension of each zone where those properties have to be assigned in order to match the analytical results with the ones obtained from mechanical testing.

It has to be considered another important aspect: time. Constructing a 3-dimensional FE model as the one described previously is a tedious and time-consuming process, which dramatically increases if non-linear behaviour wants to be included for a more accurate analysis. This is undesirable for small arch manufacturers, who want to know the load-bearing capability of their products in a quantity of time compatible with the demands of the market. This is the reason why simplified models that incorporate certain degree of simplifications are of special interest in this paper, and its good performance has been demonstrated.

The authors believe that the analytical and testing methods presented are complementary, in spite of the aforementioned drawbacks and the excessive simplicity of the FE models. We encourage the local manufacturers to perform mechanical tests on their products, which design conditions must agree with those prescribed in the current structural codes.

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