Experimental testing of steel arches

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Experimental testing of steel arches

Preliminary investigation

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Fall 2003
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Table of contents

SUMMARY ........................................................................................................................................III
DEFINITIONS ...................................................................................................................................III

1 INTRODUCTION ....................................................................................................................................1
1.1 OBJECTIVES ..................................................................................................................................1
1.2 RESTRICTIONS ............................................................................................................................1
1.3 ASSUMPTIONS ............................................................................................................................1

2 STATE OF THE ART OF EXPERIMENTAL TESTING OF ARCHES ....................................3
2.1 GLOSSARY OF EXPERIMENTS .................................................................................................3
2.2 COMPARISON OF SECTIONS .......................................................................................................6
2.3 COMPARISON OF ARCHES ........................................................................................................7
2.4 BOUNDARY CONDITIONS ...........................................................................................................8
2.5 LOADING .......................................................................................................................................9
2.6 MATERIALS AND PRODUCTION .............................................................................................10
2.7 CONCLUSIONS .........................................................................................................................10

3 GEOMETRY ......................................................................................................................................11
3.1 SHAPES ....................................................................................................................................11
3.2 CIRCULAR ARCHES ....................................................................................................................12
3.3 IMPERFECTIONS ........................................................................................................................12

4 MECHANICS ....................................................................................................................................13
4.1 SECTIONS ..................................................................................................................................13
4.2 LOAD CASES ...............................................................................................................................15
4.3 BOUNDARY CONDITIONS ........................................................................................................15
4.4 LOADING ....................................................................................................................................18

5 CONTROLLING THE EXPERIMENTS ................................................................................21
5.1 CONTROLLING MECHANISMS ...................................................................................................21
5.2 AVAILABLE JACKS ...................................................................................................................21

6 MEASUREMENTS ............................................................................................................................23
6.1 MATERIAL PROPERTIES ............................................................................................................23
6.2 GEOMETRY ..................................................................................................................................24
6.3 LOAD, DISPLACEMENTS AND STRAINS ..................................................................................24

7 MANUFACTURING OF THE ARCHES ................................................................................27
7.1 PRODUCTION OF MODEL SECTIONS .......................................................................................27
7.2 ROLLING ARCHES ......................................................................................................................27

8 PRELIMINARY FINITE ELEMENT ANALYSIS ...........................................................................29
8.1 FE-MODEL ..................................................................................................................................29
8.2 PARAMETER STUDY ..................................................................................................................30
8.3 RESULTS .....................................................................................................................................32
8.4 DISCUSSION .............................................................................................................................34

9 RESULTING EXPERIMENTAL PROGRAM .............................................................................35
9.1 TEST PROGRAM ........................................................................................................................35
9.2 DISCUSSION ...............................................................................................................................36

10 CONCLUSIONS ............................................................................................................................39
REFERENCES .................................................................................................................................41
**Summary**

In this report an investigation is made into possibilities of experimental testing of the stability of steel arches. The relevant literature is discussed in chapter two. Chapter three discusses issues of the geometry while chapter four gives considerations of the mechanics of arches. Then the possibilities of steering the experiments are discussed in chapter five, while chapter six treats the measurements that should be taken before and during testing. In chapter seven, manufacturing methods of arches are discussed while chapter eight gives the results of preliminary finite element analyses. Finally, all these items led to a test program in chapter nine and a selection was made for the experimental testing of stability of steel arches.

**Definitions**

Curved member: Member with the shape of either a segment of a circle, catenary or parabola without horizontal restraint in the plane of loading.

Arch: Member with the shape of either a segment of a circle, catenary or parabola with horizontal restraints in the plane of loading.

Free-standing arch: An arch supported only at its ends.

Gravity load: Load that keeps its original direction (conservative loading).

Tilt load: Load that becomes inclined when the specimen deforms. This load has a fixed point at which it is directed (non-conservative loading).
1 Introduction
In this report the objectives and restrictions are written down which formed the basis for the tests that were carried out in the laboratory. The tests themselves will be described in a forthcoming report (la Poutré, 2003). The contents of the current report and the forthcoming test report will be, in part, incorporated in the PhD thesis of the author.

1.1 Objectives
The main objective of the test program is to generate data on which a finite element (FE) model can be calibrated. This data was not sufficiently available in literature, so it was decided to carry out tests. The objectives are formulated as follows:

- The spatial stability phenomenon of steel arches is to be captured. The geometry and load configuration should be such that the arch buckles laterally rather than in-plane. The shape of the arch, the exact dimensions and load configuration will be decided upon in this investigation.
- Elastic-plastic out-of-plane stability is focused on.
- Load–deformation and load-strain relations, to be able to calibrate an FE–model, are to be gathered.

1.2 Restrictions
In previous studies, mainly the elastic stability of arches was researched. In this study the elastic-plastic out-of-plane stability under bending and compression is investigated which puts restrictions on the size and shape of the arches. Furthermore, there are constraints on what is feasible in the laboratory. The restrictions are summed up as follows:

- The experiments should be configured such that the results are of practical use, i.e. the arches must not be too slender or too stocky.
- The curvature of the arch should be such that snap through buckling will not occur. This puts restrictions on the subtended angle ‘\( \alpha \)’ of the arch.
- For circular arches, the maximum subtended angle considered is 180°.
- The proportion of in-plane bending stiffness to lateral bending and torsional stiffness should be such that lateral buckling occurs before in-plane buckling.
- The arch is bound to maximum dimensions dictated by the transport possibilities (maximum rise of 3.5 meters) and by the available space in the TU/e Faculty of Architecture and Building laboratory.
- The arch must be manufactured from one continuous member. An assembled arch should be avoided because the connection of the parts could influence the structural behavior.
- The dimensions of the arch should be chosen such that failure can be reached within the capacity of the equipment available at the TU/e Faculty of Architecture and Building laboratory.

1.3 Assumptions
- The load cases should be chosen such that they resemble actual load situations.
- At the boundaries (supports and load introduction) rotations and displacements are either completely free or fully restrained.
2 State of the art of experimental testing of arches
At first an investigation was carried out to find what kind of experiments have been performed in the past. The test set-ups, boundary conditions and load configurations used in these experiments are discussed. This investigation is limited to experiments on single, free-standing arches.

2.1 Glossary of experiments

Stüssi, 1944
The elastic stability of a parabolic arch, loaded by eight equally spaced concentrated loads, was tested, see figure 1. The arch was cut from an aluminum plate and had a rectangular cross section. Loads remained vertical (gravity loading) as the arch deformed sideways. The arch was cut from a flat sheet of aluminum. The objective of these tests was to verify the method of calculating the critical load presented in the paper.

![Figure 1. Test arrangement of Stüssi (not to scale).](image1)

Godden, 1954
Elastic stability of stabbogen arches was studied, see figure 2. In a stabbogen-arch, the suspended structure (e.g. bridge deck) possesses a greater in-plane bending stiffness than the arch rib. In this structure, the arch is a pure compressive member. Load was applied in a reversed way: the supports were pushed towards each other while the wire hangers were fixed in vertical and lateral position, but free to slide along the chord between the supports. A vibrating machine was mounted on the test rig to overcome friction in the supports and wire hangers. Since the wire hangers were fixed in lateral direction, they made an angle with the original direction as the arch deformed out-of-plane. This type of loading is considered as tilt loading. The cross sections used were solid circular bars and hollow seamless elliptical steel tubes. The dimensions of the cross sections were not supplied in the paper.

The objective of the tests was “verifying the validity of the assumptions made in the theory outlined” in Godden, 1954 and in particularly “the assumption for the buckling form in each of the two boundary conditions analyzed”. See section 2.4 for the boundary conditions used in these tests. Southwell plots were used to determine the buckling loads from the load-displacement data.

![Figure 2. Test set-up of Godden (not to scale).](image2)
**Kee, 1959**
The elastic stability of a parabolic tied arch (stabbogen) was tested on the test set-up of Godden. The cross section was a solid circular steel bar with a diameter of 0.246 inch. The objective of the experiments was to study the effect of wind load and initial lateral eccentricities on the buckling load. The arches were either tested with no lateral load, with one lateral load at the crown, or with five equally spaced lateral loads to initiate lateral buckling. Southwell plots were used to determine the buckling loads.

**Kee, 1961**
This study investigated inelastic stability. The same test setup was used as in the previous tests of Kee, 1959. The spans of the arches were decreased to 40 inch and just seven wire-hangers were used. The material used was commercially pure aluminum with comparatively large inelastic range and low elastic limit. As cross section a tube with 3/8 inch outer diameter was used. The tests were carried out to verify a method of computing the inelastic buckling load and to examine the validity of modified Southwell plots to predict the inelastic failure load.

**Klöppel & Protte, 1961**
The elastic stability of circular arches, loaded by horizontal loads at the supports, was tested, see figure 3. The arches had a subtended angle of either 90º or 180º. The cross sections were either a rectangular plate or an I-section. These tests were performed to verify analytical work.

![Figure 3. Test set-up of Klöpp & Protte (not to scale).](image)

**Tokarz, 1968**
The elastic stability of 35 arches was studied, of which 13 arches were free standing. Of these free standing arches, seven were tested with gravity loads and six with tilt loading. Load was applied through 14 equally spaced wire-hangers, see figure 4. The supports were either fixed or hinged. Parabolic and circular arches, made of aluminum, were tested. All spans were 59 inch with heights ranging from 12 to 24 inch. The purpose of these tests was to supply additional experimental verification “of lateral buckling solutions of curved members based upon a linear buckling theory”.

![Figure 4. Test set-up of Tokarz (not to scale).](image)
**Di Tommaso & Viola, 1976**
The elastic stability of a circular arch with built-in ends and a single load at the crown was tested, see figure 5. The arch was made of a thin sheet of aluminum. The arch had a subtended angle of 135°. The experiment was performed to compare with analytical results.

![Diagram](image)

*Figure 5. Test set-up of Di Tommaso (not to scale).*

**Papangelis & Trahair, 1987**
The elastic stability of a circular arch with a single concentrated load at the crown was tested, see figure 6. The cross section was an aluminum I-section. Both supports were rollers, which made the construction behave like a curved beam rather than an arch.
The experiments were set-up to evaluate the difference between older analytical solutions by Timoshenko & Gere, 1961, Vacharajittiphan & Trahair, 1975, and Vlasov, 1963, with, at that time, more recent studies by Yoo, 1982, Yoo & Pfeiffer, 1983, and Yoo & Pfeiffer, 1984. The results from the tests rebutted the work of Yoo & Pfeiffer. The results were also compared to an FE-model of Papangelis.

![Diagram](image)

*Figure 6. Test set-up of Papangelis & Trahair (not to scale).*
2.2 Comparison of sections

Figure 7 shows the sections used in the previously discussed experiments. It is clear that these sections are all quite different.

To be able to better compare the sections, they were all scaled to a depth arbitrarily chosen as one hundred millimeters, which is shown in figure 8. Underneath the cross section the scale factor is given; dividing by this factor gives the actual depth (e.g. the section of Di Tommaso has a depth of $100/1.25 = 80$ mm). From these pictures it emerges that Di Tommaso and Klöppel used rather thin rectangular sections compared to Stüssi and Tokarz. As for the I-sections, Klöppel uses a much thinner section than Papangelis. The solid bar of Godden cannot be compared to the ones used by Kee, since the dimensions of his section were not mentioned in his paper.
2.3 Comparison of arches

In this comparison, only the circular arches were used. The comparison made is that of absolute and relative size. Figure 9 shows all circular arches used in experiments to scale. For each arch the radius and subtended angle is given. Figure 10 shows the arches scaled to an equal, but arbitrary radius of 1 m. The number under each arch is the scale factor; dividing by this factor gives the actual radius (e.g. one of the 180° arches of Klöppel has a radius of 1000/5 = 200 mm).

From figure 9 it becomes clear that all tested circular arches were rather small (model testing). Figure 10 shows clearly that some arches had larger section depths than others. For example Klöppel tested three semicircular arches which were difficult to compare to each other. Once these were scaled to the same radius, the difference in section depth became visible.

![Figure 9. Comparison of circular arches in experiments.](image)

![Figure 10. Arches scaled to equal radii.](image)
2.4 **Boundary conditions**

2.4.1 **Supports**

In the experiments several different support conditions were used. Stüssi used in-plane hinges with out-of-plane fixity. Godden used two support conditions: pin-ended and out of plane fixed, but torsionally free. Tokarz used in-plane and out-of-plane fixed in some of his experiments, while in other experiments the arch rib was fixed for out-of-plane rotation, see figure 11. The support condition of Papangelis allowed in-plane and out-of-plane rotation while the entire support was free to translate along the chord of the arch, see figure 12 (a). The functioning of the support is shown in figure 12 (b) and was described in detail in Put et al., 1999. Klöppel used a similar support to that of Papangelis, but instead of the support being free to translate, load was applied by pulling the support along the test rig, see figure 13. Friction in this support was overcome by manually vibrating the set-up.

![Support conditions in tests Tokarz](image11)

(a) out-of-plane hinged

(b) out-of-plane fixed support

**Figure 11. Support conditions in tests Tokarz.**

![Support condition in tests Papangelis](image12)

(a) support in arch test (after Singer et al., 1998, p. 445).

(b) drawing of same support used in tests Put et al.

**Figure 12. Support condition in tests Papangelis.**

![Support condition in test of Klöppel](image13)

**Figure 13. Support condition in test of Klöppel.**
2.4.2 Load introduction

In this section an overview is given of the different ways of introducing load. Tokarz made special arrangements to introduce load exactly at the centroid of the cross section. As the arch rib twists, load will remain to act at the centroid, see figure 14. Papangelis introduced load on the top flange with a knife-edge fitting, see figure 15. As the arch rib twists, the load at the top flange will amplify the twisting. Godden introduced load by inserting wire hangers vertically through the tube and soldering them in place. Kee introduced load by wrapping the wire around the tubes and twining them together underneath. In these cases the load will act at the bottom of the tube and will diminish twisting (see also section 4.3.2).

![Figure 14. Load introduction at centroid in experiments of Tokarz.](image)

![Figure 15. Load introduction at top flange by Papangelis (after Singer et al., 1998, p. 444).](image)

2.5 Loading

Different ways of applying load were used. While most experiments were loaded with dead weight (Stüssi, Klöppel, Tokarz, Di Tommaso, and Papangelis), some were loaded by screw spindles (Godden and Kee). Loading with spindles is displacement controlled and one can slowly approach the buckling load and stop before actual buckling. Southwell plots were then used to extrapolate the buckling load.

Loading with dead weight can be done in two ways: load control and buoyancy control. In load control, the arch is loaded by suspending a platform from it, on which dead weight is piled up. The load is increased until the ultimate load is reached and the arch will fail suddenly. Buoyancy control was used in the experiments of Tokarz, 1968, and Di Tommaso & Viola, 1976. Canisters were suspended from the arch in a water tank, see figure 16 (a). The load of the cable and canister $F_{\text{canister}}$ was in equilibrium with the buoyancy force $B_1$. Then, weights were placed in the canisters, which sank into the water tank. The weights exerted a force on the arch but also increased the buoyancy force $B_2$ on the canisters, see figure 16 (b). Adding weights was continued until the lateral deflections became disproportional to the weight increase. At this point the water level in the tank was lowered which reduced the buoyancy of the canisters. Reducing the buoyancy increased the load that the canisters exerted on the arch, without adding weights to the canisters, see figure 16 (c). With this method the load could be precisely regulated. By controlling the water level, one controls the buoyancy and indirectly the stroke.
2.6 Materials and production

Materials. In the experiments two types of material were used: steel and aluminum. Since most studies were aimed at elastic stability either aluminum or steel could be chosen. Often aluminum was chosen because it was easy to machine into arches. The material type and properties used in the experiments discussed, are listed in table 1. Stüssi determined the E-modulus of his material by a bending test while Godden did not supply any information on the type of steel used. Kee, 1961, used commercially pure aluminum with a comparatively large inelastic range because he tested inelastic buckling of arches. Papangelis used aluminum with a comparatively large elastic range since he was testing elastic stability. The E-modulus was determined from bending tests. He also made an analysis of the residual stresses and concluded that these were small in the flange compared to the web and would have very little effect on the elastic stability.

Table 1. Materials used in previous tests.

<table>
<thead>
<tr>
<th>material</th>
<th>specifications</th>
<th>E-modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stüssi</td>
<td>aluminum</td>
<td>E = 700 t/cm² ≈ 70,000 N/mm²</td>
</tr>
<tr>
<td>Godden</td>
<td>steel</td>
<td>E = 11.24·10⁶ [psi] ≈ 79,200 [N/mm²]</td>
</tr>
<tr>
<td>Kee 1961</td>
<td>aluminum, comm. pure alu.</td>
<td>E = 10.7·10⁶ [psi] ≈ 75,400 [N/mm²]</td>
</tr>
<tr>
<td>Klöppel</td>
<td>steel</td>
<td>E = 72,600 [N/mm²]</td>
</tr>
<tr>
<td>Tokarz</td>
<td>aluminum, alloy 2024-T3</td>
<td>E ≈ 11·10⁶ [N/mm²]</td>
</tr>
<tr>
<td>Di Tommaso</td>
<td>aluminum</td>
<td>E ≈ 63,000 [N/mm²] (f₀₂ = 124 [N/mm²])</td>
</tr>
</tbody>
</table>

Production of arches. Most arches with rectangular cross sections were cut from flat plates (Stüssi, Tokarz, Di Tommaso). The tubes used by Godden and Kee were bent into a parabolic shape over a line drawing. The webs of the arches of Klöppel with I-sections were cut form a flat plate after which the flanges, also cut from flat plates, were bent and soldered to the web. The I-section arches of Papangelis were roller bent into shape. See chapter seven for a discussion of bending techniques.

2.7 Conclusions

From the study of previous tests on arches, it emerges that all tests were model tests and that most researchers studied elastic stability. All tests were initiated to prove analytical theoretical work while the most recent tests (Papangelis) were also compared to FEA. The inelastic tests of Kee, 1961, were also setup to verify analytical work as well as the possibility of extending Southwell plots to inelastic stability. In all tests, Southwell plots were used to extrapolate failure loads which means that no post buckling behavior of arches was available.

From the comparison of cross sections, it became clear that most cross sections were very slender plates, which are rather susceptible to out-of-plane buckling. These types of cross sections do not seem very realistic in practice. Most arches were produced by cutting the shape out of a flat plate, which gave residual stress-free specimens. A few arches were roller bent into shape, which gave specimens with residual stresses.

The rather unpractical cross sections, the absence of full-scale and inelastic stability tests has led to setting up a test program for the inelastic stability of full-scale steel arches. In the next chapters, this program will be elaborated.
3 Geometry

Before treating the mechanics of an arch, the geometry is investigated. Different shapes of arches and imperfections are discussed. All these issues influence the mechanics which will be treated in the next chapter.

3.1 Shapes

Several shapes of arches are conceivable, such as circular, parabolic and catenary shapes. In figure 17 all these shapes have been plotted. The functions of these plots are given by equations 3-1 to 3-3, with $-1 < x < 1$.

\[
\text{circle}(x) = \sqrt{1 - x^2} \quad (3-1)
\]

\[
\text{hyperbola}(x) = 1 - x^2 \quad (3-2)
\]

\[
\text{catenary}(x) = 2.84 - 1.84 \cdot \cosh(x) \quad (3-3)
\]

Each of these shapes has their specific advantage: Pure compressions emerges for a radial load in a circle, for an equally distributed vertical load in a parabola and pure tension emerges for the own weight of a suspended chain in a catenary. Furthermore the circle is simplest to roll because the curvature is constant and it is assumed that it is used most often in practice for arches made out of a continuous member (non assembled). Therefore, the circle is selected for testing.

![Figure 17. Shapes of arches.](image-url)
3.2 Circular arches

The dimensions of a circular arch can be described by five parameters, see figure 18. However, only two parameters are needed to fix all dimensions.

Dimensions of center line of arch:
- \( L \) = length of the arch [mm]
- \( S \) = Span [mm]
- \( R \) = radius [mm]
- \( H \) = rise of the arch [mm]
- \( \alpha \) = subtended angle [rad] or [°]

Dimensions of the cross section:
- \( h \) = depth of the section [mm]

Figure 18. Dimensions of a circular arch.

3.3 Imperfections

A construction cannot be considered to be either completely straight or vertical. Therefore certain imperfections need to be modeled. Sway imperfections were modeled analogous to a frame. In section 5.2.4 of Eurocode 3 (ENV 1993-1-1, 92) provisions are made for frame imperfections. An arch could resemble a single story frame, thus the imperfection angle ‘\( \phi \)’ amounts to 1/200\(^{th}\) of the rise of the arch ‘\( H \)’, see figure 19.

If one considers the arch as a member, a common lateral imperfection ‘\( w_0 \)’ of 1/1000\(^{th}\) of the span ‘\( S \)’ can be imposed. For arches with a low rise in relation to the span, the member imperfection is larger than the sway imperfection and should be modeled.

In the preliminary FEA (chapter eight) the sway imperfection has been slightly reduced to \( \phi = 1/250 \cdot H \). The imperfection modeled will be the largest of the sway imperfection or the member imperfection.

In the experimental program, the actual imperfection (out-of-plane, in-plane and twist) will be measured for each test and is afterwards used to simulate that test.

Figure 19. Imperfections to be modeled.
4 Mechanics

4.1 Sections

A number of sections are feasible to use for the arch rib, see figure 20. Each of these sections has specific advantages and disadvantages. A short discussion:

- In case of the tube the moment of inertia is equal about any axis passing though the centroid. Torsion is resisted solely by Saint Venant-torsion since warping does not occur in tubes. By measuring strains on the exterior of the tube, the bending moment and the torsional moment can be determined since no warping occurs. Also, no special attention needs to be paid to warping at the supports, since it does not occur.

- The rectangular hollow section (RHS) has such a geometry that some warping can occur. However, a torsional moment is mainly resisted by Saint Venant-torsion, a small part will be resisted by warping restraint-torsion. The warping restraint-torsion might be negligible. If not, special attention must be paid to the design of the supports in regard with warping.

- The I-section has a small torsional rigidity and a warping constant of intermediate value. Furthermore the minor moment of inertia is less than one tenth of the major moment of inertia. Therefore this section is highly susceptible to failure through lateral torsional buckling.

- In wide flange beams the relation between the minor and major moment of inertia is not as small as in the I-sections. The warping constant is larger than the one of the I-section. This makes the section less susceptible to lateral torsional buckling.

<table>
<thead>
<tr>
<th>Tube</th>
<th>Rectangular hollow section (RHS)</th>
<th>I-section</th>
<th>Wide flange section</th>
</tr>
</thead>
<tbody>
<tr>
<td>I/(I_t) = 1</td>
<td>(I_{z}/I_{y}) ≤ 1</td>
<td>(I_{z}/I_{y}) &lt; 0.1</td>
<td>(I_{/I_{y}}) &lt; 0.3</td>
</tr>
<tr>
<td>(I_{x} = 0)</td>
<td>(I_{x} = \text{small})</td>
<td>(I_{x} = \text{intermediate})</td>
<td>(I_{x} = \text{large})</td>
</tr>
<tr>
<td>(I_{t} = \text{large})</td>
<td>(I_{t} = \text{large})</td>
<td>(I_{t} = \text{small})</td>
<td>(I_{t} = \text{intermediate})</td>
</tr>
</tbody>
</table>

In making a selection from these types of sections, the discussed mechanical properties need to be considered. The tests are aimed at calibrating an FE-model. This model needs to be able to describe bi-axial bending, compression and mixed torsion. The tube and RHS are not selected because no, or very little, mixed torsion occurs. Then the choice comes down to an I-section or a wide flange beam. The differences between these types of sections are discussed next.

In figure 21 the ratio of minor to major moment of inertia was plotted on the horizontal axis while the normalized beam parameter for torsion was plotted on the vertical axis. The beam parameter for torsion was normalized by multiplying it by the depth of the section, see equation 4-1.

\[
\sqrt{\frac{I_{y}}{I_{w}}} \cdot h
\]  

(4-1)

European wide flange beams have a width equal to the depth up to depths of 300 mm. After that, the width remains constant at 300 mm. The effect of this change becomes apparent in figure 21. In this figure the ratio \(I_{y}/I_{y}\) is almost equal for all sections below 300 mm depth. After 300 mm depth, the ratio reduces rapidly. For the torsion parameter, the opposite is true: for increasing depths, up to 300 mm, it reduces rapidly, while it remains equal or increases slightly for section depths beyond 300 mm. For the upcoming experiments, a selection was made for
wide flange beams because generally they have a larger ratio of $I_z/I_y$, which makes them stiffer for out-of-plane bending. Furthermore, these beams are commonly available and can be bent easily into arches.

![Graph showing ratio minor-major moment of inertia versus torsion parameter]

Figure 21. Cross sectional properties of wide flange beams and I-beams in relation to each other.

Out of the range of available wide flange beams, a selection was made of three different cross sections. An HEA 100 section was selected because it represents the outer extreme of the combination ratio $I_z/I_y$ and torsion parameter. Then an HEB 300 section was selected because at that depth the ratio $I_z/I_y$ and beam parameter change considerably. Finally an HEB 600 section was selected to have a section near the other extreme of the graph, see figure 22.

![Selected sections for full-scale testing]

Figure 22. Selected sections for full-scale testing.

Since these sections are quite different in size, they would each demand their own supports and load introduction fittings if they were tested on full-scale. This was not feasible and therefore it was decided to scale the larger sections to the depth of the HEA 100, see figure 23. See chapter seven for details about the scaling techniques. Scaling has an additional benefit that exactly the same quality of steel can be used for all arches.

![Selected sections for experimental testing]

Figure 23. Selected sections for experimental testing.
4.2 Load cases

In a previous study by Delrue, 1998, two load cases have been investigated: pure bending and pure compression, see figure 24. For these load cases it was possible to analytically determine the elastic critical load (eigenvalue).

However, the boundary conditions in these load case are not used in practice, and therefore they should not be used for the experiments (see next section for a discussion of boundary conditions). Furthermore, equally distributed loads that can freely move with the lateral deformations of the arch cannot be generated in the experiments, ruling out pure compression. A selection must be made from one or more concentrated loads, see figure 25. In practice these loads will predominately be vertical in direction rather than radial. For the experiments a single point load at the crown was selected because it introduces the maximum combination of bending moment and compressive forces in the arch.

4.3 Boundary conditions

4.3.1 Supports

In the previous section, some boundary conditions were already suggested. In this section they are investigated further. First a choice was made for a co-ordinate system, see figure 26 (a). The global Cartesian co-ordinate system determines the location of the arch in space and is indicated by capitals. The y- and z-axis of the local co-ordinate system coincide with the principle axes of the cross section, and the x-axis is the tangent to the arch in the considered cross section. Figure 26 (b) gives the positive signs for displacements and rotations in the co-ordinate systems.
Table 2. Feasible boundary conditions in local Cartesian co-ordinates.

<table>
<thead>
<tr>
<th></th>
<th>left support</th>
<th>right support</th>
</tr>
</thead>
<tbody>
<tr>
<td>minimum restraints: *)</td>
<td>( u = v = \phi = 0 )</td>
<td>( u = v = w = \phi = 0 )</td>
</tr>
<tr>
<td>two axis pinned ends:</td>
<td>( u = v = w = \phi = 0 )</td>
<td>as left support</td>
</tr>
<tr>
<td>one axis pinned ends:</td>
<td>( u = v = w = \phi = \psi = 0 )</td>
<td>as left support</td>
</tr>
<tr>
<td>fixed ends:</td>
<td>( u = v = w = \phi = 0 = \psi = 0 )</td>
<td>as left support</td>
</tr>
</tbody>
</table>

*) does not function for arches with a subtended angle of 180°.

The displacements or rotations were considered either free or fully restrained. A combination of restraints must be chosen such that the arch is at least statically determinate. These restraints were used in Delrue, 1998 and are referred to as ‘minimum’ in table 2. These minimum restraints do not make use of the specific advantages of an arch that the stiffness is improved by restraining the radial displacement ‘\( w \)’.

For the experiments, two in-plane hinges were selected. Warping of the cross section will be prevented at the supports. For out-of-plane rotations the arch rib will be fixed, see figure 27. The hinges are fixed in place, thus full benefit of the arch stiffness can be used. The supports should be designed such that the out-of-plane bending moment \( M_x \), see figure 27 (b), does not impair the in-plane hinge function. The actual design will be treated in la Poutré, 2003.

4.3.2 Load introduction

Location of load. Three locations for load introduction were considered: at the top flange, at the centroid, and at the bottom flange of the section. Initially, load at any of these locations will just produce bending and shear in the arch. However, upon buckling the loads at the top and bottom flange will either amplify or diminish torsion in the cross section, see figure 28. This additional effect should be avoided in the experiments, therefore load will be applied at the centroid of the section.

Types of load introduction. Three types of load introduction at the centroid were considered. The first is to use a steel wire, similar to the experiments by Tokarz, see figure 14. This solutions has as advantage that it is rather simple to realize. The disadvantage is that the sections in the current research all have bottom flanges. These may come in contact with the wire
hangers once the section buckles. While Tokarz used many points of load application along the arch, in the present research only one is used. Also, the elastic-plastic stability is studied of stronger sections than Tokarz. These two items result in a much higher load on the single wire hanger, which might cause such high stresses at the hole in the web, that reinforcement becomes necessary. Therefore this option was not selected.

The second option is to use a rigid shaft, which is placed through a hole in the web, see figure 29. This setup was successfully used in stability experiments on channel sections (la Poutré et al., 2000). However, in these experiments the loads remained below 50 kN but already showed some plastic deformation around the holes in the web. In the experiments on arches, loads twice as high are expected. Reinforcement of the web will then become necessary. Advantages of this method are that it is simple to realize and one can reduce the stresses in the web by selecting a wider shaft and making a deeper notch. The disadvantages are the high stress concentration and having to cut a hole in the web of considerable dimensions at the most heavily stressed cross section.

The third option was to use a spherical bearing on top of the section. The centroid of the sphere coincides with the centroid of the cross section. However which way the section twists, the applied load will always act at the centroid, figure 30. This solution was used in stability experiments by Maljaars et al., 2002.

Type of bearings. There are three working principles for bearings: rubbing contact, rolling elements and fluid films (O'Donoghue et al., 1970). In the rubbing contact bearing, two surfaces slide over each other. This is the simplest bearing but it has, relative to the other bearings, enormous friction to overcome. In the rolling bearings, rolling elements allow for the relative motion of the two load carrying surfaces. In these bearings the concentrated stress at the contact with the rolling elements, is the limiting factor. Of the fluid film bearings two types exist: hydrodynamic and hydrostatic bearings. In the hydrostatic bearing, the load carrying capacity is governed by the supply pressure of the lubricant, while in the hydrodynamic bearing load carrying capacity increases with the turning speed (which distributes the lubricant). A selection criterion for one of these bearings is the starting torque. Figure 31 gives the speed-torque characteristic for fluid film bearings and rolling bearings. For the tests under
consideration, only torque at zero speed and the onset of turning is of interest. Since a hydrostatic bearings has zero starting torque, it is the most suitable bearing, and it was selected for testing. A detailed account of the design of such a bearing will be given in la Poutré, 2003. The need for zero starting torque is elaborated next.

![Figure 31. Torque-speed characteristics of bearings (after O'Donoghue, 1970).](image)

Figure 31 shows the characteristic of bifurcational buckling. At the critical load, the section suddenly buckles out-of-plane and twists. If any support is given to the section, an unstable path of equilibrium of continued in-plane bending will be followed. If the bearing has any friction, a starting torque needs to be overcome to start to turn. This starting torque will support the cross section against twisting and thus support the arch in the unstable equilibrium path. Eventually buckling will occur, but might falsely be identified as the first bifurcation point. Using a hydrostatic bearing can avoid this.

**4.4 Loading**

Two types of load are distinguished: gravity load (conservative load) and load with a fixed origin (non-conservative load). Gravity loading could be compared to arch bridges with very little lateral stiffness of the bridge deck, while tilt loading occurs in bridges with a bridge deck with high lateral stiffness. How to apply these two types of load and their advantages and disadvantages are discussed briefly in this section.
4.4.1 Gravity load

In gravity loading, the direction of the load remains vertical as the member deforms. One possible way to create such a load is to measure the lateral displacement on the test specimen. This displacement is coupled with a horizontally placed jack which pushes the principal jack sideways by the same displacement, see figure 33 (a). Thus the line of action remains vertical. This is an actively controlled system and will cost some time to setup and fine tune.

A second possibility is a gravity load simulator, see figure 33 (b), which has been described in detail in Yarimci et al., 1967. The advantage of such a system is that no complex controlling system has to be set up and tuned.

A third system is an approximation to a gravity load. By choosing a very long cable in relation to the lateral displacement, the angle $\beta$ of the inclined load becomes negligible, see figure 33 (c). Experience with this type of loading has been obtained in the TU/e Faculty of Architecture and Building laboratory on earlier tests (la Poutré et al., 2000). This system has again the advantage that a complex controlling system is not needed while it has the disadvantage that a considerable height is needed in the laboratory.

---

4.4.2 Tilt load

In the case of tilt loading, the jack is fixed to the test rig. As the section deforms laterally, the load will become inclined with the original direction. At the specimen, this load can be factored in a vertical (original direction) and a horizontal component, see figure 34. This horizontal component might work favorably for the stability. Most conceivable arch structures have a fixed origin of the load, for example the bridge deck of an arched bridge. Because the experiments should resemble real arch structures as much as possible, the tilt loading seems to be in best agreement and was selected for the experiments.

Nevertheless, gravity loading was investigated also in the FEA (chapter eight) to get a feeling of the difference between the two types of loading.
5 Controlling the experiments

5.1 Controlling mechanisms

When testing the stability of a structure, typical load-deformation graphs can be of a shape as shown in figure 35. The ultimate load is reached after which the load decreases with continued deformations. The post ultimate part of the graph reveals the ductility of the system. When setting up a criterion to check the stability of a structure both the ultimate load and the post ultimate behavior need to be considered. Therefore the experiments need to be controlled such that the post ultimate behavior can be captured. Two methods were considered: loading by jacks and loading by dead weight.

![Load deformation graphs](image)

Figure 35. Load deformation graphs.

**Jacks.** In Rasmussen, 2000, three possibilities in controlling experiments loaded by jacks were distinguished:

1. Load control
2. Stroke control
3. External signal control

In the first possibility, the load monotonically increases and equilibrium will be lost at the ultimate load. Therefore this control mechanism can not be used to capture the post-ultimate behavior. This rules out the use of all hand controlled jacks listed in table 3. The second possibility of controlling the experiments has the capability of capturing the post-ultimate response. If the load decreases sharply, dynamic jumps may occur (Rasmussen, 2000) and should be avoided. It needs to be investigated if this can occur in these experiments, if so, stroke control cannot be used. The third option of controlling the experiments on an external signal has the best capability of capturing the post-ultimate response and is not susceptible to dynamic jumps provided that the external signal is monotonically increasing.

**Dead weight.** Using dead weight has as advantage that gravity loads can be easily generated. However, it has as disadvantage that a considerable amount of dead weight is needed to generate large forces. Therefore, it is not appropriate for full-scale tests.

From the possibilities outlined above, it was decided to use a jack with stroke control. Unless the FEA shows that external signal control is needed to overcome sharp decreases in load.

5.2 Available jacks

The at the TU/e Faculty of Architecture and Building laboratory available jacks are listed in table 3. There are three demands for selecting a jack:

1. Maximum load
2. Stroke
3. Control mechanism

The first two demands can be quantified after FEM-simulations have been carried out. Table 3 shows that the maximum applicable load is either 5 MN in compression or 1.4 MN in tension, with a stroke of 200 mm. The last demand, the control mechanism, is a result of the selected control mechanism, which will be discussed in a following chapter.
Table 3. Available jacks at the Faculty of Architecture and Building laboratory.

<table>
<thead>
<tr>
<th>Manufacturer</th>
<th>control</th>
<th>number of avail. jacks</th>
<th>maximum static load</th>
<th>stroke</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>compression [kN]</td>
<td>tension [kN]</td>
</tr>
<tr>
<td>Hydraudyne servo</td>
<td>2</td>
<td>87</td>
<td>-</td>
<td>200</td>
</tr>
<tr>
<td>CTD servo</td>
<td>4</td>
<td>150 150 100</td>
<td>600 600 200</td>
<td>200</td>
</tr>
<tr>
<td>Hydraudyne servo</td>
<td>4</td>
<td>2000 2000 200</td>
<td>5000 5000 200</td>
<td>200</td>
</tr>
<tr>
<td>Holmatro hand</td>
<td>2</td>
<td>100 50 150</td>
<td>240 110 150</td>
<td>150</td>
</tr>
<tr>
<td>Holmatro hand</td>
<td>2</td>
<td>244 110 300</td>
<td>1018 700 100</td>
<td>300</td>
</tr>
</tbody>
</table>

From the above discussion it can be concluded that the use of hand-controlled jacks is ruled out. Since tilt load is being used with a fixed point on the chord between the supports, a tension jack was needed, see figure 36. From table 3 it follows that servo controlled jacks were available from 87 kN up to 5,000 kN. The decision was made to use the 150 kN jack with stroke control and a stroke of 100 mm. All tests need to be selected to fit within these constraints.

Figure 36. Placing of the jack.
6 Measurements

6.1 Material properties

6.1.1 Yield strength

From the original member, before it is bent into the arch, a test coupon will be cut to determine the yield strength. This process will be carried out according to the specifications of several codes: the location where to cut the coupon from and its dimensions is taken from EN 10025, 90, and the course of the test is guided by EN 10 002-1, 90. When steel is ordered the quality indication, such as S235JR, gives a lower estimate of the yield strength, the actual yield strength is usually much higher. Figure 37 shows a typical distribution for the mentioned quality of steel. In the FEM-simulations, carried out before execution of the experiments, the actual yield strength is unknown. Three standard steel grades (s235,S275 and S355) were used. The actual yield strength of S235 might be close to the nominal yield strength of S275. In evaluating the FE results, the failure loads corresponding to S275 might be a good estimate for arches of steel grade S235.

The steel order for the experiments will be of quality S235JR. This has been decided upon the fact that the actual yield strength is higher.

![Figure 37. Possible yield strength distribution for steel S235JR.](image)

6.1.2 Young’s modulus of elasticity

The tests used to determined the yield strength can also be used to determine Young’s modulus of elasticity. An extensometer will be clamped on to the test coupon and the increase in length is measured. Generally the E-modulus is taken as $2.1 \times 10^5$ N/mm². Small deviations of this value might be possible. However, in the preliminary FEA, the nominal value of E was used. For the purpose of FEM-simulations, after the experiments are accomplished, an accurate value of the E-modulus is extremely important. Previous tensile tests (la Poutré, 1999) have shown that measuring the E-modulus precisely can be challenging. Using strain gauges on the test coupons can increases the reliability of the measurements of the E-modulus.

6.1.3 Residual stresses

Two types of residual stresses can be distinguished: stresses as a result of the hot-rolling process and stresses as a result of the arch-bending process at ambient temperatures. The first kind is well documented for most kind of sections and needs no further investigation. The second kind is assumed to be much larger and needs to be experimentally determined.

Three methods are available to determine residual stresses:
1. non-destructive through ultrasonic and X-ray methods;
2. semi-destructive through hole-drilling;
3. destructive through sectioning.

The first method is preferred since the actual specimen for testing can be used, however this method has practical limitations. The second method is suitable since the holes to be drilled are
so small that they will not alter the section properties. The last method is fully destructive and can therefore only be used on an additional piece of the arch. Recently, it has been investigated which one of the semi- and fully destructive methods can be used in the TU/e Faculty of Architecture and Building laboratory (Boon, 2001). This study concluded that sectioning was the most reliable method. However, this study was conducted on straight beams and it is not certain that this method can be used on arches. Therefore the hole-drilling method seems the most viable method for the arches.

Papangelis & Trahair, 1987, and Delrue, 1998, found that residual stresses do not have a great influence on the stability failure load but might have an influence on the deformations at failure. Therefore, they are mainly needed in calibrating the FE-model to the experiments.

6.2 Geometry
Deviations from the nominal geometry will be measured. Two types of deviations are distinguished: deviations in cross-sectional dimensions and deviations in the shape of the arch. Values of these deviations will be needed for fine-tuning of the FE-model after execution of the experiments.

6.3 Load, displacements and strains
The in-plane bending moment distribution for a semicircular arch is shown in Figure 2 and given by Equation 6-1, in which $R$ = radius; $F$ = applied load; $\phi$ = subtended angle of a segment of the arch. In this equation, the arch is assumed to be inextensible, which is an acceptable assumption for deep arches. The maximum bending moment occurs at the crown; the second largest bending moments occur at angles of 32.5° (for semicircular arches) from the supports.

$$M(\phi) = F \cdot R \left( \frac{1}{2} - \cos \frac{\phi}{2} - \sin \frac{\phi}{\pi} \right)$$ (6-1)

![Figure 38. Moment distribution for a semicircular arch under a single point load.](image)

The in-plane and out-of-plane displacements and twist will be measured near the crown, where the maximum bending moment occurs. It cannot be measured exactly at the crown because the hydrostatic bearing doesn’t leave enough space for the displacement transducers to be connected. The displacements and twists will also be measured at the location of the second largest bending moment. All displacements and twists will be measured from an independent measuring frame. The rigid body displacements of the supports will be measured to correct any in-span measurements, if necessary.

Strains will be measured at top and bottom flange at the second largest bending moments and near the crown. Again, they cannot be measured directly at the crown because the hydrostatic bearing is placed on top of the top flange. From the strains measured, the bending moment can be calculated.

At the supports, four strain gauges will be placed at the flange tips. Any out-of-plane (flange) bending will be recorded this way, see figure 39.
Figure 39. Location of strain gauges on a semicircular arch.
7 Manufacturing of the arches

7.1 Production of model sections

To produce model cross sections a number of techniques are feasible:

1. welding/soldering plates together
2. planing down larger sections
3. milling shapes out of rectangular bars
4. electric discharge eroding of shapes out of solid rectangular bars

The first technique was used by Klöppel & Protte, 1961, where sections were soldered together, and by Bartels & Bos, 1973 where they were welded. This technique has as advantage that no costly milling or planing is needed. The drawback is that the fillets cannot be formed exactly, that the plates for web and flanges might have different yield strengths and that a heat affected zone (HAZ) will be created at the web-flange junction.

In the second technique, a planing bench is used to plane down a larger section to a model section. This is a costly operation but has as benefit that the exact geometry can be approached, including the fillets and that the same material is used (for testing of both full-scale and model arches).

The third and forth technique are variations to the second technique. Generally, it can be stated that milling is more expensive than planing. Electric discharge eroding is a good option since it is a very exact technique and the model members could be produced within strict size margins and because it introduces virtually no residual surface stresses in the model member. However the specimens produced in this fashion will be limited in size.

At the production facilities of the TU Eindhoven, a small sample was made with the planing technique in which an HEB 100 member was planed down to an HEB 400 member, scale one to four. Figure 40 (a) shows the wide flange beam where half of the cross section is already planed down while the other half still needs to be worked. Figure 40 (b) shows how the model cross section fits in the original cross section. This technique proved to be applicable and was selected to be used to produce the model cross sections. The maximum length that could be worked was 4 m.

![Figure 40. Sample for planing down a wide flange beam.]

7.2 Rolling arches

The production of arches for structural purposes is predominately done by one of two techniques: roller bending or induction bending (King & Brown, 2001). A variation to roller bending is known under its brand name Diamond Curve. These three methods will be discussed in this section. More methods are available but aren’t typically used for structural purposes. These methods are not further explored here, but a discussion of techniques is given by Sillekens, 1999.

- Induction bending. In this method a member is pushed through a heating coil in which it is heated between 700°C and 1050°C over a length of 50 mm. Directly after this zone, it is cooled down. At the exit of the coil an arm, fixed on one side and with a specific length, guides the
section into the shape of a circle. The benefits of this method are the absence of residual stresses after bending and the tight radii that can be achieved.

- **Roller bending.** In roller bending, the member is passed through three rollers, the outer ones on one side and the middle one on the other. After each pass, the middle roller is moved inwards until the desired curvature is obtained, see figure 41. In this process the outer flange is plastically elongated and the inner one plastically shortened, causing residual stresses. This process is performed at ambient temperatures thus cold working the material, which changes the stress-strain diagram. The ends of the member cannot pass through the machine, leaving them more or less straight. Depending on the curvature, they need to be cut off afterwards and become waste. For the members under consideration in this project (HEA 100, HEB 300* and HEB 600*) the size of the cut off ends is about 750 mm at each side. The maximum length of scaled sections was 4 m. After taking off the scrap ends, the arches that remain will have a developed length of 2.5 m.

The benefits of this technique are that it is widely available and relatively easy to apply. The drawbacks are that the material properties are altered due to cold working and loosing material at the end of each member.

![Figure 41. Illustration of rolling arches.](http://www.marksmetal.com/diamondcurve.htm)

- **Diamond curve technology**[^1]. This technology is a trademark of Marks Metal Company. In this method rollers squeeze one flange such that it extends longitudinally. This makes the member curve. In this method only the material properties of the squeezed flange are altered while the rest of the cross section retains its original properties. However, it does produce residual stresses.

The selected technology for the arches in this project is the roller bending process because it is commonly available, and it will be representative from most structural arches.

[^1]: http://www.marksmetal.com/diamondcurve.htm
8 Preliminary Finite Element Analysis

In this chapter, preliminary FEA are discussed from which the final selection for the experiments is made. Some parts of the work discussed here, has been published in la Poutré, 2002 and la Poutré et al., 2003. Therefore, the description of the FE-model will be brief.

8.1 FE-model

8.1.1 Cross section and arch

The nominal cross section, shown in figure 42 (a), was modeled in Ansys with 8-node quadrilateral shell elements (Shell 93) with four elements per flange and four in the web, see figure 42 (b). With shell elements, the fillets cannot be modeled exactly and ignoring the fillets would greatly underestimate the Saint Venant-torsional rigidity. To solve this problem, boxed beam elements were placed at the web-flange junction which possessed both the lacking torsional rigidity and the lacking area of the fillets, see figure 43 (a). How this was worked out exactly is described in la Poutré et al., 2003, p. 287. These elements had the same material law, including plasticity, as the rest of the cross section. Parts of the boxed beam elements stick out above the flanges, which might cause earlier yielding in these elements than in the extreme fibers of the flanges. Along the circumference a minimum of forty elements were used. This number was increased as the developed length of the arches increased. Figure 43 (b) and (c) show how the elements were distributed along the circumference.

Load was introduced with a link element (no bending moments) that extended from the centroid of the section to the chord between the supports. At the chord, displacements were prevented in the horizontal plane while a vertical displacement was imposed on the link, see figure 43 (b).
8.1.2 Initial imperfections and material law

**Initial imperfections.** The initial lateral imperfection was modeled with a sinusoidal variation. The maximum imperfection \( w_0 \) was placed at the crown and was equal to \( 1/250 \)th of the rise of the arch (see section 3.3). The magnitude of the lateral imperfection \( w(\phi) \) at a subtended angle \( \phi \) was equal to the sine of that angle times the maximum imperfection \( w_0 \) at the crown, see figure 44 (a) and (b).

**Material law.** Since the yield strength of the material was not known beforehand, three different yield strengths were used in the simulations. If steel grade S235 will be used for the experiments, the lower bound for the yield strength is 235 N/mm\(^2\). However, the actual yield strength will almost certainly be higher, but probably not exceed that of steel grade S355. Thus, steel grade S355 was taken as upper bound and the analyses were carried out for steel grade S235, S275 and S355.

The material was modeled with the bi-linear material law with kinematic hardening (BKIN in Ansys) and the von Mises yield criterion. In this law, the behavior is linear elastic up to the yield point from where plasticity starts. In the analyses, the plastic branch had zero rigidity, see figure 44 (c).

![Figure 44. Imperfections modeled.](image)

8.2 Parameter study

Three series of simulations were carried out, two on full-scale arches and one on model arches. The full-scale simulations were divided into one series with gravity loading and one with tilt loading. All model arches had tilt loading.
8.2.1 Full-scale arches

Full-scale arches with gravity loading: Arches with an HEB 100 cross section and a constant radius were used. The subtended angle was varied from 180º to 90º and the developed length from 6 m to 3 m, see figure 45. In this analysis the load remained vertical (gravity load) upon failure of the arch. This exercise has been described more closely in la Poutré, 2002.

![Figure 45. Simulation of full-scale HEB 100 arches (variation of subtended angle and developed length).](image)

Full-scale arches with tilt loading: A second analysis was performed on arches with an HEA 100 cross section. In this analysis, the developed length was kept constant at 6 m while the subtended angle was varied between 180º and 90º. In these analyses tilt loading was applied.

![Figure 46. Simulation of full-scale HEA 100 arches (variation of subtended angle and radius).](image)

8.2.2 Model arches

Model arches with tilt loading: In this analysis scaled arches were investigated. The cross sections that were used were: HEB 300*, HEB 400*, HEB 600*, and HEB 800* sections, where * denotes that these were scaled sections to a depth of 96 mm. All model arches had a developed length of 2.5 m (figure 47), which corresponded to the maximum feasible length for scaled sections, see chapter seven.

![Figure 47. Simulation of model arches (variation of subtended angle and radius).](image)
8.3 Results

8.3.1 Full-scale arches

Full-scale arches with gravity loading: Figure 48 shows the stability behavior of a semicircular HEB 100 arch under gravity loading. Table 4 gives the failure loads of the HEB 100 arches.

Stability behavior of semicircular arch with an HEB 100 cross section and several yield strengths

Figure 48. Load - displacement and load - twist diagrams for one arch with three different yield strengths.

<table>
<thead>
<tr>
<th>subtended angle</th>
<th>90°</th>
<th>110°</th>
<th>135°</th>
<th>160°</th>
<th>180°</th>
</tr>
</thead>
<tbody>
<tr>
<td>radius [mm]</td>
<td>1910</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>dev. length [mm]</td>
<td>3000</td>
<td>3667</td>
<td>4500</td>
<td>5334</td>
<td>6000</td>
</tr>
<tr>
<td>( F_{\text{ult},S235} ) [kN]</td>
<td>160</td>
<td>146</td>
<td>121</td>
<td>93</td>
<td>71</td>
</tr>
<tr>
<td>( F_{\text{ult},S275} ) [kN]</td>
<td>185</td>
<td>169</td>
<td>136</td>
<td>100</td>
<td>74</td>
</tr>
<tr>
<td>( F_{\text{ult},S355} ) [kN]</td>
<td>235</td>
<td>208</td>
<td>156</td>
<td>108</td>
<td>78</td>
</tr>
</tbody>
</table>

Full-scale arches with tilt loading: Figure 49 shows the results of the analysis of the HEA 100 arches under tilt loading. In table 5 the failure loads are listed. All failure loads of this series were within the maximum applied load of 150 kN (the capacity of the selected jack).

Figure 49. Failure loads of full-scale arches under tilt loading.

<table>
<thead>
<tr>
<th>subtended angle</th>
<th>90°</th>
<th>110°</th>
<th>135°</th>
<th>160°</th>
<th>180°</th>
</tr>
</thead>
<tbody>
<tr>
<td>radius [mm]</td>
<td>3820</td>
<td>3125</td>
<td>2564</td>
<td>2149</td>
<td>1910</td>
</tr>
<tr>
<td>dev. length [mm]</td>
<td>6000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( F_{\text{ult},S235} ) [kN]</td>
<td>75.6</td>
<td>77.7</td>
<td>77.8</td>
<td>78.1</td>
<td>77.3</td>
</tr>
<tr>
<td>( F_{\text{ult},S275} ) [kN]</td>
<td>87.1</td>
<td>89.2</td>
<td>90.1</td>
<td>89.8</td>
<td>89.0</td>
</tr>
<tr>
<td>( F_{\text{ult},S355} ) [kN]</td>
<td>107.2</td>
<td>110.0</td>
<td>111.7</td>
<td>111.6</td>
<td>111.0</td>
</tr>
</tbody>
</table>
8.3.2 Model Arches

Model arches with tilt loading: The results of the model arches are given in Table 6 and Figure 50. One can see that the failure loads rapidly reduce with more slender sections. The failure loads of model arch HEB 300* and HEB 400* are even higher than those of the full-scale HEA 100 arch, which is due to the smaller developed length of the model arches. However, they do remain within the limit of 150 kN of the selected jack.

Table 6. Failure loads of model arches under tilt loading.

<table>
<thead>
<tr>
<th>subtended angle [°]</th>
<th>90°</th>
<th>110°</th>
<th>135°</th>
<th>160°</th>
<th>180°</th>
</tr>
</thead>
<tbody>
<tr>
<td>radius [mm]</td>
<td>1592</td>
<td>1302</td>
<td>1061</td>
<td>895</td>
<td>796</td>
</tr>
<tr>
<td>dev. length [mm]</td>
<td>2500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F_{ult,S235} [kN]</td>
<td>94.2</td>
<td>94.9</td>
<td>95.4</td>
<td>95.7</td>
<td>95.9</td>
</tr>
<tr>
<td>F_{ult,S275} [kN]</td>
<td>109.2</td>
<td>110.2</td>
<td>110.8</td>
<td>111.1</td>
<td>111.3</td>
</tr>
<tr>
<td>F_{ult,S355} [kN]</td>
<td>138.5</td>
<td>140.1</td>
<td>141.1</td>
<td>141.6</td>
<td>141.7</td>
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<tr>
<td>HEB 300*</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>F_{ult,S235} [kN]</td>
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<td>92.4</td>
<td>93.9</td>
<td>94.2</td>
<td>94.3</td>
</tr>
<tr>
<td>F_{ult,S275} [kN]</td>
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<td>107.3</td>
<td>109.1</td>
<td>109.6</td>
<td>109.6</td>
</tr>
<tr>
<td>F_{ult,S355} [kN]</td>
<td>132.0</td>
<td>136.5</td>
<td>138.9</td>
<td>139.8</td>
<td>139.8</td>
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<tr>
<td>HEB 400*</td>
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<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>F_{ult,S235} [kN]</td>
<td>54.3</td>
<td>57.2</td>
<td>59.3</td>
<td>60.2</td>
<td>60.2</td>
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<tr>
<td>F_{ult,S275} [kN]</td>
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<td>75.5</td>
<td>79.2</td>
<td>72.9</td>
<td>73.1</td>
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<tr>
<td>F_{ult,S355} [kN]</td>
<td>31.9</td>
<td>33.8</td>
<td>35.2</td>
<td>35.9</td>
<td>35.9</td>
</tr>
<tr>
<td>HEB 600*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F_{ult,S235} [kN]</td>
<td>34.4</td>
<td>36.7</td>
<td>38.6</td>
<td>39.3</td>
<td>39.2</td>
</tr>
<tr>
<td>F_{ult,S275} [kN]</td>
<td>38.1</td>
<td>40.9</td>
<td>43.2</td>
<td>44.5</td>
<td>44.8</td>
</tr>
<tr>
<td>F_{ult,S355} [kN]</td>
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<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>HEB 800*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F_{ult,S235} [kN]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F_{ult,S275} [kN]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F_{ult,S355} [kN]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 50. Simulated failure loads of scaled arches.
8.4 Discussion

8.4.1 Full scale arches
It can be observed that for tilt loading (HEA 100) the failure loads remain nearly the same for subtended angles between 90° and 180°. For gravity loading (HEB 100) the failure loads reduce considerably from 90° to 180°. Of these two analyses, only the semicircular arches have equal dimensions (radius and developed length). The HEB 100 section is heavier than the HEA 100 section. Gravity loading produces lower failure loads on the heavier section than tilt loading on the lighter section (compare table 4 with table 5). Which illustrates the positive effect tilt loading has on the failure load.
For the experimental program it was decided to use an equal developed length of six meters for all arches, while the subtended angle and radius were varied.

8.4.2 Model arches
The analyses showed that the earlier selected cross sections (HEA 100, HEB 300* and HEB 600* in chapter four) can be used in stability experiments. The minimum subtended angle was determined at 90° to stay away from snap through buckling and more lateral torsional buckling type of failure, which was one of the restrictions of this project.
9 Resulting experimental program

9.1 Test program

In the previous chapters (three to eight) several parameters have been discussed and were selected for the experimental program. Table 7 shows the restrictions that have been selected up to now in a chronological order.

For the shape of the arch a circular shape was selected. The geometry of this arch will not be perfect. Therefore the actual imperfections of the tests specimens will be measured to be used in FE-simulations after the tests, while in the preliminary FEA certain limits will be used for the lateral imperfections (chapter three). The cross sections were selected based upon their ratio of in-plane to out-of-plane bending as well as their torsional properties. The load case to be used in the test was chosen to be a single point load at the crown acting at the centroid of the cross section. The type of loading that will be used is tilt loading. The supports were in-plane hinges while out-of-plane rotations were prevented as well as warping of the section (chapter four). The experiments will be stroke controlled and load will be applied by a servo-controlled 150 kN jack with a stroke of 100 mm (chapter five). The steel grade was selected to be S235JR. Of the delivered steel, a number of measurements will be taken, such as tensile tests, to determine the E-modulus and the yield strength. Besides the hot-rolling of the sections, the bending process will introduce residual stresses which will be measured as well. Furthermore, a plan for measuring lateral imperfections, deformations and strains was set up (chapter six). To create model cross sections, planing was selected, and to create arches, the roller bending technique was chosen (chapter seven). Finally, an FEA parameter analysis determined the dimensions of the arches to be tested (chapter eigth).

Table 7. selected parameters.

<table>
<thead>
<tr>
<th>section</th>
<th>item</th>
<th>restriction</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>shape of arch</td>
<td>circle</td>
</tr>
<tr>
<td>3.3</td>
<td>imperfection measuring actual imperfections</td>
<td></td>
</tr>
<tr>
<td>4.1</td>
<td>section wide flange beams HEA100, HEB300/600</td>
<td></td>
</tr>
<tr>
<td>4.2</td>
<td>load case single concentrated load at the crown</td>
<td></td>
</tr>
<tr>
<td>4.3.1</td>
<td>supports in-plane pinned ends, out-of-plane fixed, warping fixed</td>
<td></td>
</tr>
<tr>
<td>4.3.2</td>
<td>load introduction load acting at the centroid</td>
<td></td>
</tr>
<tr>
<td>4.5</td>
<td>type of load tilt load</td>
<td></td>
</tr>
<tr>
<td>5.1</td>
<td>control of the experiments stroke control</td>
<td></td>
</tr>
<tr>
<td>5.2</td>
<td>type of jack servo controlled, 150 kN, 100 mm stroke</td>
<td></td>
</tr>
<tr>
<td>6.1.1</td>
<td>yield strength tensile testing S235JR (f_y = 235 N/mm²)</td>
<td></td>
</tr>
<tr>
<td>6.1.2</td>
<td>Young’s modulus appl. of strain gauges to measure E-mod</td>
<td></td>
</tr>
<tr>
<td>6.1.3</td>
<td>residual stresses center hole drilling method</td>
<td></td>
</tr>
<tr>
<td>7.1</td>
<td>model sections planing</td>
<td></td>
</tr>
<tr>
<td>7.2</td>
<td>bending arches roller bending</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>FEA</td>
<td>HEA100 arches with L = 6 m and 90° &lt; α &lt; 180°</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HEB300/600 arches with L = 2.5 m and 90° &lt; α &lt; 180°</td>
</tr>
</tbody>
</table>
Table 8 gives the total program, including the number of times a test is repeated. The full-scale tests on 180° and 90° arches were repeated five times. This was done to get familiar with the type of testing and to have spare material if any tests went wrong. The remaining full-scale tests were only repeated once. Of the model arches only one test was planned because of the amount of work that goes into scaling the cross section. Also, not all intermediate angles between 180° and 90° will be tested, but only 135°.

The gray cells in table 5 and table 6 give the expected failure loads for the experimental program (see section 6.1.1).

### Table 8. Test program.

<table>
<thead>
<tr>
<th>Subtended angle</th>
<th>HEA 100 L = 6 m</th>
<th>HEB 300* L = 2.5 m</th>
<th>HEB 600* L = 2.5 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>90°</td>
<td>5</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>110°</td>
<td>2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>135°</td>
<td>2</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>160°</td>
<td>2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>180°</td>
<td>5</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

*) scaled to the depth of HEA 100

### 9.2 Discussion

To compare the selected cross section to the earlier used cross sections, they are shown in relative scale in figure 51. From this figure it emerges that the webs of the selected sections are thinner than those of the sections of Klöppel and Papangelis, but the flanges are (much) wider, which gives larger lateral bending stiffness.

![Cross sections for current tests compared to previous tests.](image_url)

The dimensions of the arches are compared to previous tests in figure 52 and 53. Figure 52 shows the arches in absolute scale and it becomes clear that all specimens for the current test program, even the model arches, are larger than those earlier tested. Figure 53 gives the arches in relative scale and makes it possible to compare the slenderness of the arches. The arches used by Klöppel are the stockiest while those of Papangelis and the full-scale arches are rather slender.
Figure 52. Selected arches for testing in relative scale to arches tested previously.

Figure 53. Selected arches for testing and arches tested previously scaled to equal radii.
10 Conclusions

This report gives the backgrounds to the tests carried out in the laboratory. Based on a study of literature, a test program has been conceived to extend the set of experiments known in literature. The objective of the experimental program is to provide data on which an FE-model can be calibrated.

The investigation showed the need for elastic-plastic full-scale stability experiments. In this report a feasible test program has been devised. In this program, the majority of the arches are full-scale, only the largest sections were scaled down. This was done to be able to use the same supports and load introduction as well as having the same quality of steel.

All arches will be loaded beyond the buckling load to produce load-displacement graphs, while in literature only failure loads, or extrapolated failure loads were issued. The arches will have realistic cross sections selected from commonly used European wide flange beams. The selected bending process is commonly used for bending structural elements.

This test program was conceived to calibrate an FE-model and not to compare the results to analytical equations for the buckling load, since these are not available for the load case under investigation.
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