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Proceedings of the METNET Seminar 2012 in Izmir

Kuldeep Virdi and Lauri Tenhunen (Editors)

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HAMK University of Applied Sciences

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PREFACE

Many European universities and research institutes cooperate actively with regional or international companies. Such cooperation, typically in the form of formal or informal networks, is essential to foster regional innovation environments. Each regional network has its own development strategy, focus and strengths. For a network to operate successfully, healthy interaction must be maintained between its participants. Through these links, companies acquire new knowledge, develop new technologies and tap new business opportunities. By maintaining information channels and accessibility of information at regional level, collaboration at international level is facilitated better.

In line with above objectives, METNET network aims to match and utilize the know-how and activities of European Research, Development and Training organizations, to develop new products, services and business for the companies in the field of metal products, to support innovation processes in involved companies and to promote knowledge and best practice transfer between companies, universities and research and training organizations in Europe. More information on METNET can be found at www.hamk.fi/metnet.

Basic ideas behind METNET are: to promote sharing of current knowledge, capabilities and resources in the sphere of metal products, especially those relating to construction, to support co-operation at European level by finding topics of current needs of member organisations, to conduct international seminars, workshops, training, consultation, analyses, design, testing and prototype manufacturing, to plan and prepare national and international research projects, to administer and coordinate these projects. METNET also cooperates in Management issues and with issues covering Industrial Economics. METNET seeks financial support from companies, European Union, World Bank etc., to carry out demanding research and development tasks.

The METNET partners contribute to new effective bilateral and/or multilateral cooperation models for technology and construction industry as well as industrial economics topics within the Triple Helix framework (companies – universities – policy makers).

Today, safety is a high first priority for any modern company or organization in the construction field. Systematic work has reduced the number of accidents substantially, but further efforts are needed to reach the "zero accident" target. This forms part of codes and standards, corporate vision and mission statements, operational plans and personnel policies. For example, building structures need to satisfy given design criteria, predicated on safety, serviceability and performance. Buildings endure conventional loads as well as the effects of changing climate and natural disasters arising from fire, wind, snow, icing, rainfall, temperature variations, and radiation, among others.

Sustainable development is aimed to fulfil the needs of the present from the points of view of social, environmental and economic aspects, without compromising the ability of future generations to meet their own needs. In this sense, sustainable development follows the guidelines of Pareto efficiency in allocating fairly the resources of the present and the future.

METNET focuses on Urban development as a process of synergetic integration and sustainable evolution among subsystems making up an urban area, namely, economic, social, physical and environmental factors. METNET endorses the terms and the broad purpose, concept and methodology of strong, safe and sustainable urban development.

Within the METNET network the participants organize annual seminars, work-shops, flexible collaboration and support, planning, preparing and administrating national and international projects as well as dissemination of know-how. The first International METNET seminar took place in Berlin in November 2006. In 2011, with the seminar taking place at Aarhus University, a precedent was set for formal presentation of quality research papers and technical notes using a peer review system and publication of the proceedings in book form.

The present book collects the material from the seventh international METNET seminar, arranged in Izmir, Turkey, during 10-11 October 2012. The materials of the seminars and workshops can also be found from the METNET web/site www.hamk.fi/metnet.

The papers in this volume cover themes of current technical research on steel structures. Three of the papers relate to material properties of steel, five papers cover the performance of steel structures and a further three papers deal with issues that concern the steel industry.

Structural quality steels with very high strength, nearly four times the strength of mild steel, are now manufactured by several steel makers. Such steels have already found applications in the automobile industry. The first paper in the proceedings deals with the material properties of interest in the design of steel beams made with ultra-high strength steels. Use of state-of-the art digital imaging technology forms the basis of a paper dealing with strain localisation in high strength steel structures. The problems associated with bolted connections are discussed in the context of modern wind turbines in another paper.

With the need to cover long spans economically, use is often made of membrane structures. Such structures inevitably use a steel skeleton to give the form envisaged by the architect. Interaction between the supporting steel structure and the non-metallic membrane is described in a paper resulting from collaboration between a university and industry.

With ever increasing strengths of steels and the resulting use of more slender structural elements, the problem of interaction between local and global buckling becomes of great concern. This topic is dealt with in two papers. One of the papers adopts the device of using load-deflection characteristics of plate elements, adapted as effective stress-strain characteristics, in determining the ultimate strength of stiffened plate structures. Another paper develops the elastic stiffness of thin-walled bar elements subject to local buckling for use in overall structural analysis.

Optimum design of structures, until recently, has been considered to be elusive in the light of the computational effort required. The paper dealing with multi-criteria optimisation brings in simultaneous consideration of normal loading as well as exceptional loading and other aspects such as energy consumption, environmental impact and also customer preference. The paper uses genetic algorithms in an innovative manner.

Sustainable refurbishment of residential buildings with steel elements as the basis of solution are described in a paper, which also considers life cycle costing, carbon footprint, and other factors with alternative design scenarios.

Issues of wider concern to the steel industry are covered in three papers in the proceedings. In one of the papers, the author considers the issue of optimising the price of a tender for a construction project. The skills relating to safety, from a broad perspective, needed in the future within the construction industry are dealt with in another paper. The final paper in the proceedings considers an evolutionary approach towards developing new products, an issue of great concern to steel industry.

The editors hope that this volume will contribute significantly to the understanding of technical as well as commercial issues relating to the use of steel in the construction industry. This takes on much greater importance as advanced steels with very high strength appear in the market at very competitive costs.

The editors wish to record their gratitude to the sponsors of the seminar, namely Rautaruukki Oyj, HAMK University of Applied Sciences and the European Regional Development Fund.

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NEW STEELS AND BOLTED CONNECTIONS

BENDING PROPERTIES OF SOME ULTRA-HIGH-STRENGTH STEELS

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Ruukki Metals Oy

ABSTRACT

Some ultra-high-strength steels (UHSS) were studied using bending tests with various punch radii and diameters of the V-die. The thicknesses of the steels were either 8 or 10 mm or both. The test samples were bent to a 90-degree angle using a hydraulic press. During the tests, bending forces and spring backs were measured. After the tests, cross-sections of some bends were examined by means of hardness measurements which were performed for cross-sections of some of the bends in order to define the location of the neutral axis. The ratio of the location of the neutral axis to the thickness of the material was calculated from the results for all specimens. The maximum forces applied in the bending tests were compared with the forces predicted by known equations which usually consider parameters such as bending width, the strength of the steel, the thickness of the sheet and the diameter of the V-die. The results showed that the punch radius also has an influence on the force, especially when the radius is large, which is usually the case with the bending of UHS steels. One modified equation which deals with the punch radius gave a fairly good correlation between the test results and the equation. Spring-back increased with the strength of the steel but also with increased punch radius and diameter of the Vdie. Overall, the spring-back was found to be very high, especially with the highest strength, which must be borne in mind when the tools and processes are being designed. When the punch radius was reduced, in most cases the neutral axis moved from close to the centre line towards the inner surface of the bend, meaning that the k-factor was reduced.

INTRODUCTION

Ultra-high-strength steels (UHSS) are usually considered to be steels with yield strength of more than 550 N/mm^2 and an ultimate tensile strength of more than 700 N/mm^2. Optim®700 MC Plus steel is thermo-mechanically hot-rolled with accelerated cooling, Optim®700 QL steel is quenched and tempered, and Optim®960 QC, Raex® 400, and Raex®500 steels are manufactured by controlled rolling and subsequent direct quenching. The steels have a fine-grained martensitic or bainitic-martensitic microstructure and good strength and toughness properties. Because of their high strength, which can be utilised in the design of structures of lighter weight, the use of UHS steels has and the demand for them has been increasing. At the same time, challenges in product manufacturing have also increased.

A typical method for manufacturing products from UHS steels is forming with a hydraulic bending press. When the strength of the steels increases, the required force increases. This leads to higher demands being imposed on the bending machines and the need for evaluation of the bending force. The manufacturers of steel and bending machines have developed equations that are intended to predict the bending force F:

$$F = C\,x\,\frac{R_m \times b \times t^2}{W} \quad \text{[N]} \qquad \text{(Ruukki)} \qquad (1)$$

$$F = \frac{b \times t^2 \times R_m}{W} \quad \text{[N]} \quad \text{for } (W/t \geq 10) \qquad \text{(Schuler)} \qquad (2)$$

$$F = \left(1 + \frac{4 \times t}{W}\right) x \frac{b \times t^2 \times R_m}{W} \quad \text{[N]} \quad \text{for } (W/t < 10) \qquad \text{(Schuler)} \qquad (3)$$

$$F = \frac{1.33 \times b \times R_m \times t^2}{W - (2 \times cos45° \times R_p)} \quad \text{[N]} \qquad \text{(Trumpf)} \qquad (4)$$

where: C is a coefficient 1.2...1.5 (Eq. 1)
R_m is the ultimate tensile strength (N/mm2)
b is the bending width (mm)
t is the sheet thickness (mm)
W is the diameter of the V-die (mm)
R_p is the punch radius(mm) (Eq. 4)

A bent sheet has areas where the stress is not high enough to cause plastic deformation. The deformation remains elastic, causing internal stress, and the sheet tends to return to its original shape after the force is released. This is called spring-back. The material strength, thickness, and the tools have an effect on the magnitude of the phenomenon. Referring to Figure 1, the spring-back angle β is given by:

$$\beta = \alpha_2 - \alpha_1 \qquad (5)$$

where: α_1 is the bending angle
α_2 is the bending angle after spring-back

Another way to express the magnitude of the spring-back is the spring-back ratio K. Referring to Figure 1, the spring-back ratio K is given by:

$$K = \frac{\phi_2}{\phi_1} \qquad (6)$$

where: ϕ_1 is the arc angle
ϕ_2 is the arc angle after spring-back

When the sheet is bent, the inside surface of the bend is compressed and the outer surface is stretched, but somewhere within the thickness of the metal lies its neutral axis, which is a line in the metal that is neither compressed nor stretched. The location of the neutral axis varies depending on the material itself, the radius of the bend, the direction of the material grain, and the method by which it is being bent, etc. The location of this neutral axis is referred to as the k-factor, which represents the location of the neutral axis relative to the material thickness.

Referring to Figure 1, the k-factor is given by:

$$k = \frac{t}{T} \tag{7}$$

where: T is the thickness
t is the location of the neutral axis

The k-factor is used when estimating the flat length of the sheet, which is important in the design of products manufactured by bending.

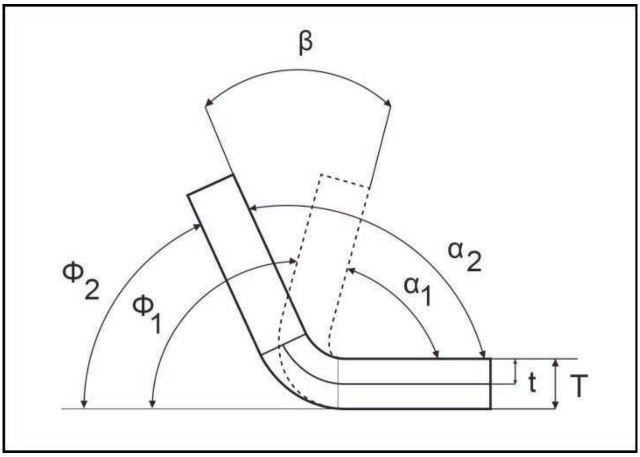

Figure 1. Spring-back phenomenon and neutral axis.

EXPERIMENTS

Five different UHS steels were used as test materials. The steel thicknesses were either 8 or 10 mm or both. The test materials, with their nominal thicknesses, typical compositions, and mechanical properties are shown in Table 1.

Table 1. Test materials and their properties

Test material	Nominal thickness (mm)	C Max	Si Max	Mn Max	P Max	S Max	B Max	Cr Max	Ni Max	Cu Max	Mo Max	Ti Max	Al Min	$R_{p0.2}$ min N/mm^2	Rm min N/mm^2	A5 min %
Optim 700 MC Plus	10	0.10	0.50	2.10	0.020	0.010							0.015	700	750-930	15
Optim 700 QL	8 / 10	0.20	0.80	1.70	0.020	0.010	0.005	1.50		0.50	0.70			690	770-940	14
Optim 960 QC	8	0.11	0.25	1.20	0.020	0.010						0.070		960	1000	7
Raex 400	8 / 10	0.25	0.80	1.70	0.025	0.015	0.005	1.50	1.00		0.50			1000	1250	10
Raex 500	10	0.30	0.80	1.70	0.025	0.015	0.005	1.00	1.00		0.50			1250	1600	8

For the tests, 285 mm wide pieces were cut from the sheets. For each test piece, one edge was painted white in order to enhance its visibility. Each test piece was bent to a 90-degree angle using different punch radii and diameters of the V-die. The bending line was parallel to the rolling direction. The force of the press was measured during the tests with a pressure sensor mounted on a hydraulic cylinder of the machine.

The spring-back angle of the test piece was measured by machine vision. The bending procedure was photographed and the measuring method that was developed was used to measure the spring-back and bending angles. The measuring method was based on automatic edge finding and line fitting. The outer edges of the test piece were found and the lines were fitted to the outer edges. The bending and spring-back angles were defined with these fitted lines. The measuring principle and the machine vision system are shown in Figure 2.

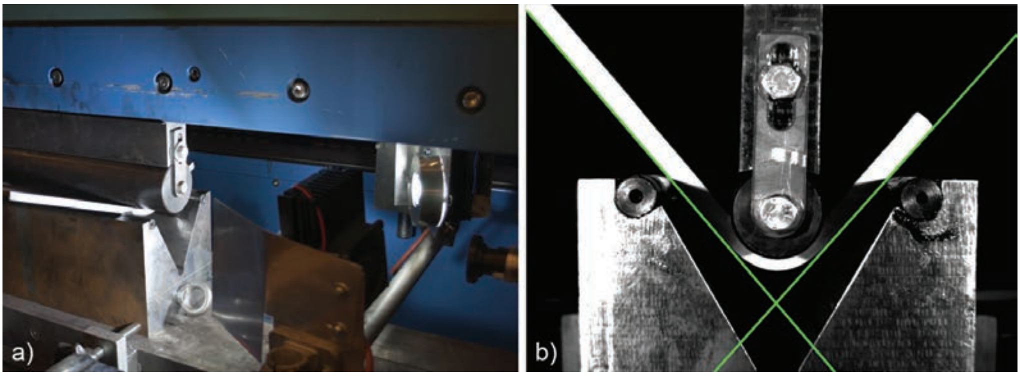

Figure 2. a) The spring-back machine vision system,
b) The measuring principle of bending angle measurements.
The green lines are the fitted measurement lines.

Samples were cut from the bent test pieces and cross-sections of the bends were examined. For the determination of the neutral axis, seven radial lines were selected and hardness (HV5) was measured along the lines from the outer surface to the inner surface. The distance between the individual measurements was 0.25 mm.

RESULTS

The influence of the punch radius on the force either measured or calculated using the above-mentioned equations is illustrated in Figure 3 a. The measured force increases when the punch radius increases. With a small radius, the measured and the calculated forces are close to each other, but when the radius is large, the difference increases. Ruukki's and Shuler's equations do not contain the punch radius as a variable, which means that the force remains constant when calculated using these equations. When the radius is large, the best equation is clearly Trumpf's. However, the measured forces are still lower than those that are calculated. The correlation between the measured forces (all tests) and the calculated forces using Trumpf's equation is shown in Figure 3 b. It can be seen that the calculated forces are greater than the measured ones in each case.

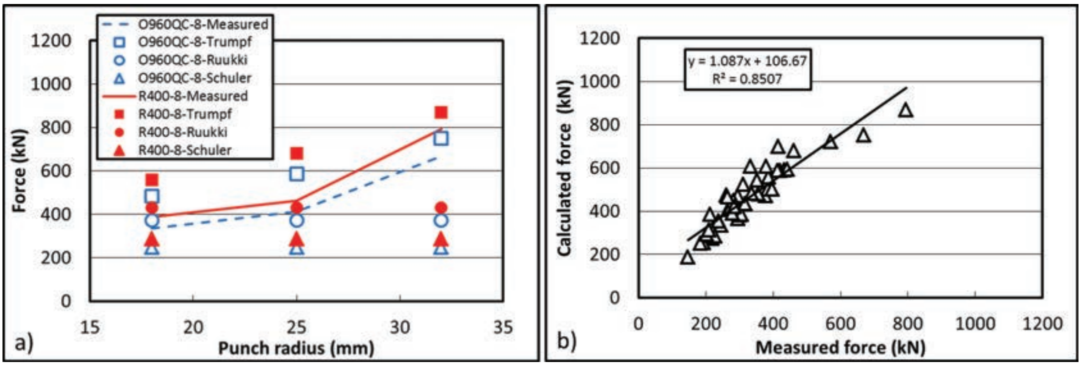

Figure 3. a) Influence of the punch radius on the measured and calculated force on 8 mm Optim 960 QC and 8 mm Raex 400 steels bent with an 80 mm V-die, b) Correlation between measured (all tests) and calculated forces using Trumpf's equation.

In order to improve the compatibility of the measured and calculated values, a modification of Trumpf's equation was examined. In the equation the coefficients 1.33 and 2 were changed to 0.8 and 2.5 respectively and the forces were recalculated. The modified equation is shown below.

$$F = \frac{0.8 \times b \times R_m \times t^2}{W - (2.5 \times cos45° \times R_p)} \qquad (8)$$

The correlation between the forces measured and recalculated using the modification of Trumpf's equation is shown in Figure 4a. Now the correlation is clearly better than previously. The influence of the punch radius on the force, whether measured or calculated, using the above-mentioned modification of Trumpf's equation, is illustrated in Figure 4b. Now it can be seen that the values are very close to each other.

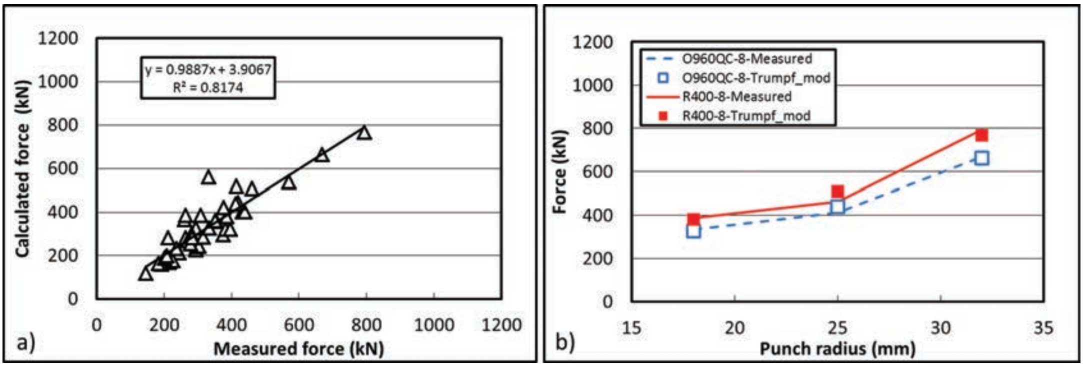

Figure 4. a) Correlation between measured and calculated force using the modification of Trumpf's equation, b) Influence of the punch radius on the measured and calculated force on 8 mm Optim 960 QC and 8 mm Raex 400 steels bent with an 80 mm V-die.

The results of the spring-back angle measurements are shown in Figure 5 a. The corresponding spring-back ratios are shown in Figure 5 b. The figures show that in addition to an increase in the material strength, an increase in the punch radius and diameter of the V-die also increases the amount of spring-back. On the contrary, an increase in the sheet thickness slightly reduces the amount of the spring-back when 8- and 10 mm Optim 700 QL are compared. Overall, when the material strength and thickness increase, a higher punch radius and diameter of the V-die must be used. This results in very high spring-back values. As can be seen, the spring-back angle can even be 40 degrees with10 mm Raex 500.

The results of the hardness measurements of 8 mm Optim 700 QL bent with two different punch radii are shown in Figures 6 and 7. The points with the lowest hardness and the corresponding HV5 values are marked in red and by arrows. A line drawn through the points could be understood as the neutral axis. The distances from the inner and outer surfaces of the bend to the neutral axis, the separation of the plate (A),and the subsequent reduction of the radius of the bend compared to the punch radius (R_{min} and R_p) are also measured and shown in the figures.

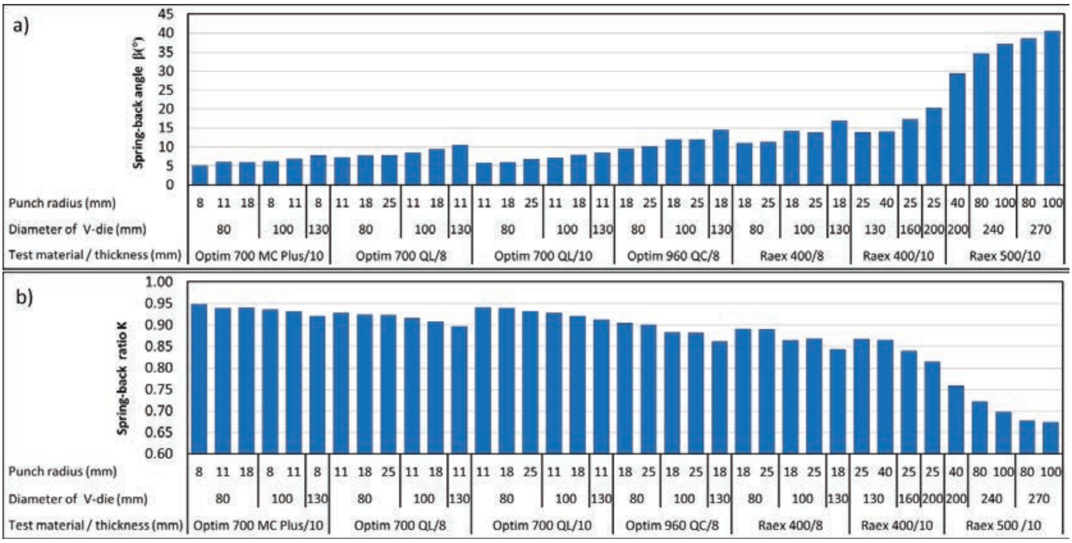

Figure 5. Influence of the punch radius and diameter of the V-die on a) the spring-back angle and b) the spring-back ratio.

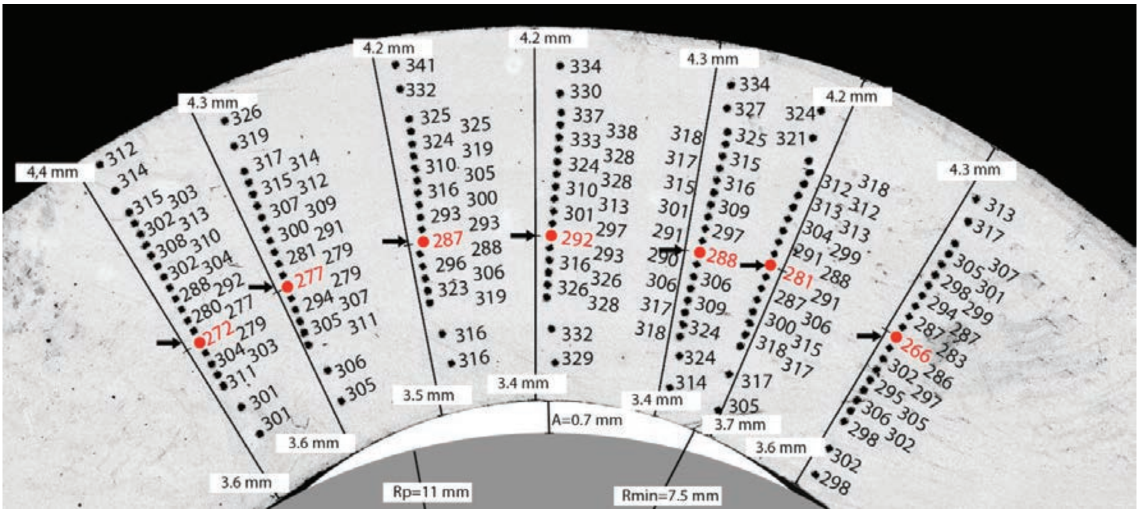

Figure 6. Hardness (HV5) measurements of 8 mm Optim 700 QL. Punch radius 11 mm, diameter of the V-die 80 mm.

The results are summarised in Table 2. The values of t and T are the average of the measured values seen in the figures. The table shows that in each case the minimum radius of the bend (R_{min}) is smaller than the punch radius (R_p). This is a consequence of the fact that the plate tends to separate from the punch like the values of A in the Table show. However, the separation is reduced when the punch radius is increased, with the result that the difference between the punch radius and minimum radius of the bend is reduced.

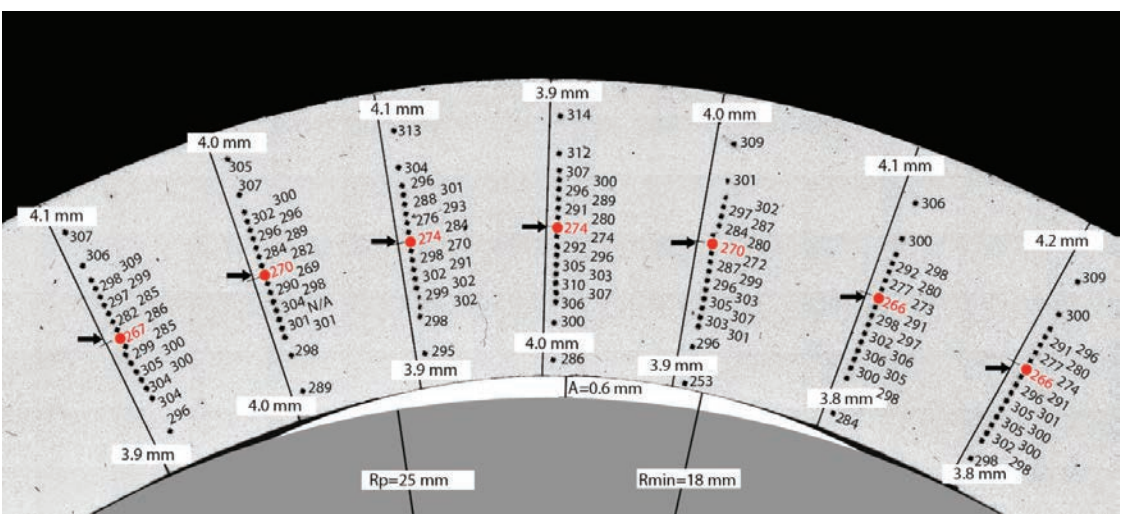

Figure 7.Hardness (HV5) measurements of 8 mm Optim 700 QL. Punch radius 25 mm, diameter of the V-die 80 mm.

Table 2.Results of the cross-sectional examinations carried out for some bends

Test material/nominal thickness	Thickness	R_p (mm)	W (mm)	R_{min} (mm)	A (mm)	t (mm)	T (mm)	k
Optim 700 MC Plus / 10 mm	10.1	8	80	6	0.3	4.5	9.6	0.42
		18	80	13.5	0.3	4.3	9.7	0.44
Optim 700 QL / 8 mm	8.2	11	80	7.5	0.7	3.5	7.8	0.45
		25	80	18	0.6	3.9	8.0	0.49
Optim 700 QL / 10 mm	10.1	11	80	8	0.3	4.3	9.7	0.44
		25	80	20	0.3	4.5	9.6	0.47
Optim 960 QC / 8 mm	7.9	18	80	11.5	0.7	3.6	7.7	0.47
		32	80	23	0.4	3.6	7.6	0.47
Raex 400 / 8 mm	8.1	18	80	13.5	0.5	3.7	7.7	0.47
		32	80	25	0.2	3.7	7.7	0.49
Raex 400 / 10 mm	10.3	25	130	17	0.8	4.7	9.9	0.47
		40	130	32	0.3	4.8	10.0	0.48

The thicknesses of the sheets are reduced by 0.2 to 0.5 mm compared to the original thicknesses of the unbent sheets (T vs. Thickness). The calculated values of the k-factor are between 0.42 and 0.49, which indicates that in each case the neutral axis is located between the centreline and the inner surface of the bend. It is evident that in most cases the values of the k-factors decreased when the punch radius also decreased. This means that the neutral axis has shifted towards the inner surface of the bend.

DISCUSSION AND CONCLUSIONS

According to the results, the punch radius has an influence on the force, which increases with increasing punch radius. Therefore it should be taken into account when the required force is being calculated. Among the equations examined, only Trumpf's equation meets this condition and after modification, relatively good correlation between the measured and the calculated forces is achieved within the tests carried out in this work. However, the width of the test piece was not varied in the tests. In the equations the influence of the width of the test piece is assumed to be linear. However, the edges of the sheets obviously reduce the force. The narrower the sheet, the greater the probable influence of the edges on the force, which results in the influence of the width being more or less nonlinear below a certain width. The width of the test piece was 285 mm in each case, which is relatively low compared to the sheet widths commonly used in the sheet metal industry. Whether the width is in the linear range or not is uncertain. Therefore, the modified equation may not be valid within a width range which is very different from the one studied here. To get more information, tests in which the bending width is varied need to be carried out.

The amount of the spring-back is relatively high with the UHS steels studied in this work. With Raex 500, the spring-back angle can be up to 40 degrees, with the spring-back ratio being as low as 0.65. This, however, is the situation in the case where the diameter of the V-die is as high as 270 mm and the punch radius reaches 100 mm. These values are compatible with the guidelines given by the steel producer. However, the tests were also carried out with a smaller punch radius (40 mm) and diameter of the V-die (200 mm), as shown in the results. Even in these cases there were no problems with the quality of the bends, but the spring-back was reduced from 40 to 30 degrees. This indicates that if high spring-back is causing problems, Raex 500 could possibly, in certain cases, be bent with a smaller punch radius and diameter of the V-die compared to the guidelines. This preferably requires tests prior to the full-scale process and assistance from the steel manufacturer.

Hardness measurements performed for the cross-sections of the bends showed that it is possible to determine the location of the neutral axis by this method. It can be seen from the results that in most cases the values of the k-factor decrease with decreasing punch radius. The thickness of the sheet is reduced when the sheet is bent but the phenomenon is more intensive when the bending radius decreases. This probably also explains the reduction of the values of the k-factor in this case.

It can be concluded that the amount of the spring-back increases with an increased diameter of the V-die, which makes the selection of the correct diameter of the V-die very important. An increase in the punch radius also increases the spring-back but on the other hand a reduction of the punch radius is limited by the forming capability, i.e. the quality of the bend. The separation of the plate also increases when the punch radius decreases but probably also when the diameter of the V-die increases. These facts should be considered when the punch radius and the diameter of the V-die are selected for the bending process and the recommendations of the steel manufacturer should be noted carefully.

ACKNOWLEDGEMENTS

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INVESTIGATION OF STRAIN LOCALIZATION OF HIGH STRENGTH STEEL WITH THE HELP OF DIGITAL IMAGE CORRELATION

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ABSTRACT

During different metal forming operations such as bending, deep drawing and stretch forming, the limiting phenomenon is necking. Necking is, in effect, localization of deformation. Necking can be divided into two parts: diffuse necking and localized necking. Both of these could be used to evaluate the formability of the material in question. The tested steel sheet material for the following study was thermo-mechanically rolled, cold formable, and structural steel with the nominal yield strength of 650 N/mm2. The base material was tested according to the current standard SFS-EN ISO 6892-1 with a Zwick / Roell 250 kN tensile test machine. The GOM ARAMIS digital image correlation (DIC) system was used to measure the strain field over the whole visible area of the specimen during the tensile test. From that information two distinct types of necking can be observed at the end phase of the test, in effect, after the point of the uniform elongation. The onset of the diffuse necking is analysed from the relation of the surface strain information. The latter necking type, the localized necking, is analysed from the thickness reduction information, or more precisely from its derivative curve. The stress value from the tensile test machine needs correction after the necking due to the change of the specimen cross section dimensions. The dimension-corrected stress was constructed by a procedure based on DIC deformation and a strain field analysis. The corrected stress strain data provides a basis for the construction of an improved material model for FEM-simulations at higher strains.

Keywords: high strength steel, digital image correlation, tensile test, ARAMIS

INTRODUCTION

Modern requirements for increasing safety as well as demand for lower fuel consumption and carbon emissions steer the material selection towards higher strengths than regular construction steels in nearly all branches of industry. With proper use of high strength steels it is possible to manufacture lighter yet stronger structures for load bearing applications (Vierelä 2012).

The most common ways of making formed high strength steel parts are by bending, stamping and deep drawing. In these forming methods, the deformation is closely related to stretching or other stretch related strain states. The thinning of the material during a forming process can lead to a plastic instability condition known as diffuse necking. After the diffuse necking has fully developed the deformation localizes heavily and leads to the localized necking which in turn results in fracture. In that way, the diffuse and localized necking can be used to evaluate the forming limits of the material in question. After the uniform elongation the diffuse necking appears, that point in a stress-strain curve can be regarded as the onset point for a material failure path. Difficulties lie in the detection of the diffuse necking in a fabricated part. The diffuse necking as such does not mean a catastrophic failure in material. Instability occurs only after the diffuse necking advances to a localized necking. After the neck localization, all deformations outside the necking zone cease and the deformation speed, i.e. the strain rate, in that zone increases rapidly. The ultimate strain in the localized necking zone is determined by material properties (Jeschkea *et al.* 2011).

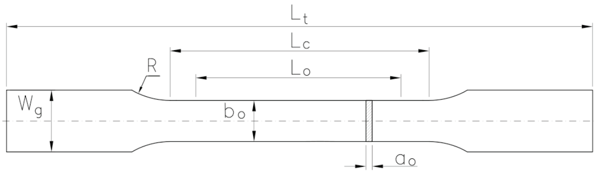
Considering the forming limits of materials the characteristics of the diffuse and localized necking give some estimates of the deformation potential of the material in question. A lot of research effort is therefore allocated in the experimental and theoretical field in metal forming to improve the quality and predictability of manufacturing. Research scale varies from standard sized tensile tests to micro-tensile tests (Jeschkea *et al.* 2011, Ghadbeigia *et al.* 2010, Hyoung *et al.* 2005).

In this study, the strain path formation of class 650 high strength steel was investigated. The tensile test was carried out according to the SFS EN 6982-1 on a Zwick Roell all-round test machine with 250kN maximum force. The digital image correlation (DIC) equipment is used to measure and evaluate surface strains over the whole visible area of the specimen. The image acquisition is continuous so that the surface strain can be followed throughout the tensile test. From the images taken it is easy to identify and follow the strain development during the test. Strains, where diffuse and localized necking appear, can be identified by following the thickness reduction value which is calculated based on the surface strains in major and minor axis. Additional information based on the DIC analysis can be included in the stress strain curve after necking has occurred.

MATERIAL AND SPECIMEN SPECIFICATIONS

High strength, single-phase, class 650 steels provide substantially higher yield strength values for design than regular constructional steel and therefore it was selected as a test material. Single-phase steel with an advanced manufacturing process provides ample formability with higher load bearing capacity than regular constructional steel. Class 650 steels could be considered as a replacement for lower grade steels with similar price range but with substantially higher yield strength for design. Material strength increase is achieved by a thermo-mechanical rolling process below the re-crystallization temperature (Vierelä 2012).

Specimen dimensions used in the standardized tensile test are depicted in Figure 1.



Specimen dimensioning

a_0—Thickness of specimen 3 mm
b_0—Width of gauge length 12.5 mm
L_t—Total length of specimen 285 mm
L_o—Gauge length 80 mm
L_c—Length of straight part 120 mm
R—Radius of Fillet 37 mm
W_g—Specimen width 30 mm

Figure 1. Dimensions of the specimen

The tensile test specimens were cut from 3 mm thick sheets and machined according to the specification required by the adopted standard. In addition to the depicted dimensions in Figure 1 an additional thinning of 0.05mm is machined in the middle of the gauge length. Extra thinning is done equally on both sides to ensure that the maximum deformation occurs in the middle of the specimen rather than randomly in the gauge length. Even with extra thinning in the middle tolerance requirements stated in the standard are fulfilled. Extra thinning improves the success rate of the tests and helps image taking during the test set-up phase.

In test preparation phase the surface of the specimen is cleaned with ethanol to remove all traces of grease and other contaminants. After cleaning, the surface is painted with white paint to coat the gauge length with an even layer of matt finishing. The stochastic, high-contrast pattern is created by spraying the surface with black paint. The stochastic pattern is needed for DIC measurements.

TENSILE TESTS AND STRAIN MEASUREMENTS

The tensile tests were done on a Zwick Roell testing machine at room temperature with two distinct strain rates during the tests. The first strain rate is for determining the Young's modulus E in the elastic range and the second for the remaining test. The strain rates are 0.00025 1/s and 0.005 1/s respectively as suggested in the used standard. The images from the specimen surface were taken at a constant rate during the test for DIC based strain measurements.

Digital image correlation is based on the non-contact imagery from the tested specimen. The measurements and the following analysis are usually focused on deformation, strain and their derivatives. During the last few decades, DIC measurements and analysis have been developed in various projects and research groups. The development of camera technology is also one of the key factors in advancement in DIC measurements. The growing popularity of DIC is due to its advantages over conventional testing; it is non-contacting, full field and simple to use. (Hyoung *et al.* 2005, Cordero *et al.* 2005, Yang *et al.* 2010)

In DIC measurements the surface deformations are observed by following the positions and shape changes of unique facets, generated by the analysis software, in sequential images. A stochastic pattern is applied on the specimen surface to be investigated. The sequence of the images is taken during the test by industrial grade cameras with charge coupled device (CCD) chips for later analysis. The displacements and deformations on the whole visible surface are measured by tracking the multiple software-generated facets.

Figure 2 shows the set-up used in the experiments. A Zwick Roell Tensile test machine in conjunction with an ARAMIS camera system is used. System supplier's selected and verified 23mm focal length lenses with polarizing filters were used in experiments. Lighting for test set-up is provided by polarized led light sources. Circular Polarizing filters are used to minimize effect of ambient and reflected light during the test.

The images were recorded with two distinct frequencies, namely 1 Hz and 3 Hz. The use of two different image capturing frequencies is based on the notion that the elastic range and test range up to force maximum on a force strain curve are less important in this kind of evaluation.

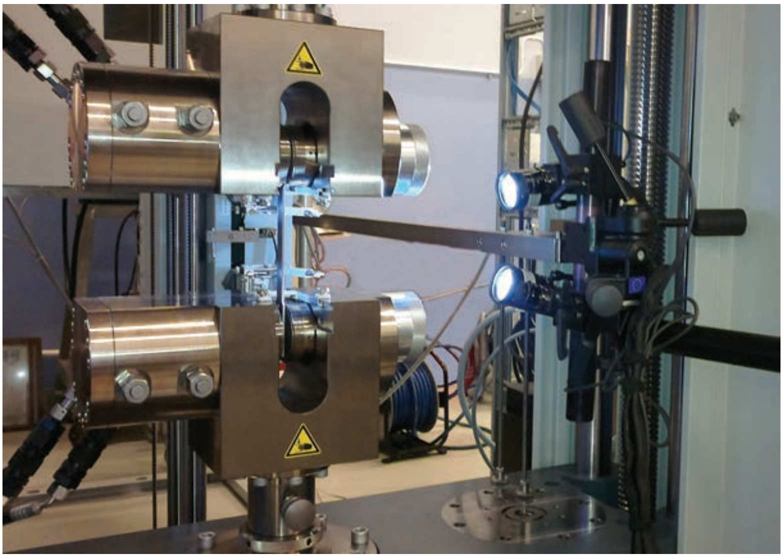


Figure 2. Layout of test setup

The main computer memory of the analysis hardware is also finite so by optimizing the number of the images the post-processing time is rationalized. The image resolution used in the analysis was 2448x 2048 pixels. The captured images were post-processed with a 3-D Analysis software ARAMIS from Gom gmbh (Germany). Based on the analysis, displacements and full-field strain information are acquired from the specimen surface. Depending on the selected measuring volume, i.e. the selected lenses and their aperture value, different accuracies are obtained (Gom gmbh 2008).

In practice, this means that the minimum length of the analytical extensometer based on the strain field generation depends on the size of the specimen. The facet size is also one factor when determining the needed accuracy. Smaller facets require longer computational times for an analysis. With a standard-sized tensile test specimen with a specific lens selection, local extensometers of 0.25 mm could be created and followed in sequence of images. In our studies the local extensometers of 1.0 mm in length could be used to follow local deformations. An example of analytical extensometers in use is shown in Figure 3.

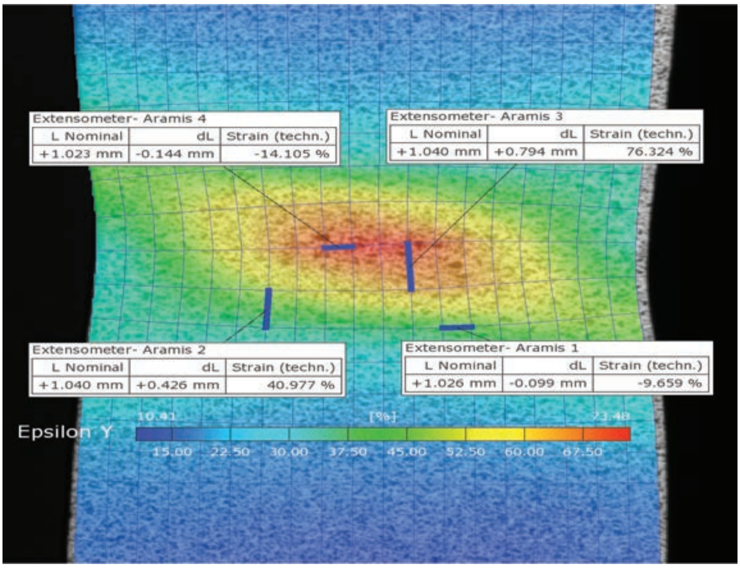


Figure 3. Demonstration of local extensometers

RESULTS

Curves from Tensile Tests

From the Zwick tensile test machine it is possible to get different curves from the test. Typically stress-strain, stress-time and force-elongation curves are exported for further analysis. Stress-time curve is good for the evaluation of test set-up stiffness which affects the control loop operation in the tensile test machine. Normally, changes in the test set-up stiffness, e.g. the large variation of specimen size, different grip set-ups, wear of test equipment etc., require a confirmation of control loop values. Typically the correct operation is confirmed by checking the elevation in stress versus time and by comparing that to the programmed value. The operation should be verified in the elastic region of the specimen material. The standard recommends the use of 30MPa/s as a start value for the stress increment speed for the first segment of the tensile test. This corresponds with the strain increment speed of 0.00025 1/s with the used gauge length.

In Figure 4 a typical force-elongation curve is shown. The force data are acquired from the load cell data of the tensile test machine and the elongation data are recorded with an accurate extensometer or from the crossbeam linear displacement sensor. The elongation data can be also acquired, as in this case, from the surface strain analysis with ARAMIS. Several options are available how the strain on the surface is evaluated for the ARAMIS-based stress-strain curve. The maximum force of 46.72 kN was recorded and A80 strain of 16.3% was measured with the tensile test machine. From the ARAMIS strain analysis it is possible to gain strain data within much smaller areas, so the local strains usually amount to much higher values than in larger scale extensometers.

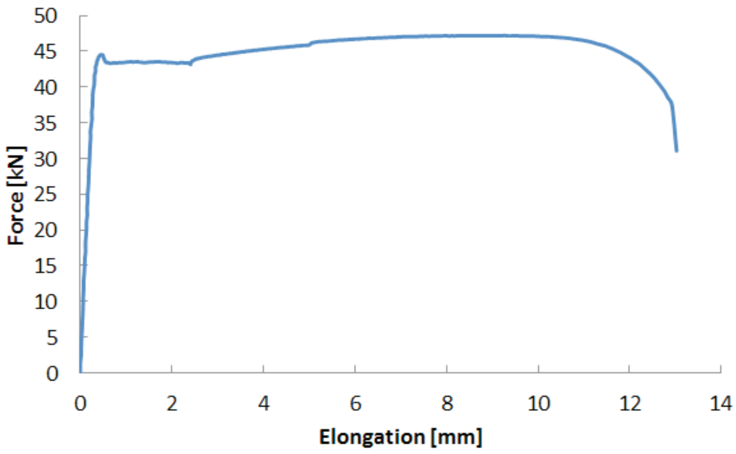


Figure 4 Force- elongation curve

Surface Strains and Strain Evolution in Tensile Tests

In order to follow surface strains and eventual strain evolution during the tensile test, an image sequence is recorded. From that sequence, consecutive images are analysed to construct a strain field. Every picture represents a momentary change in the strain. The recognized surface strains in the different stages of the tensile test are shown in Figure 5.

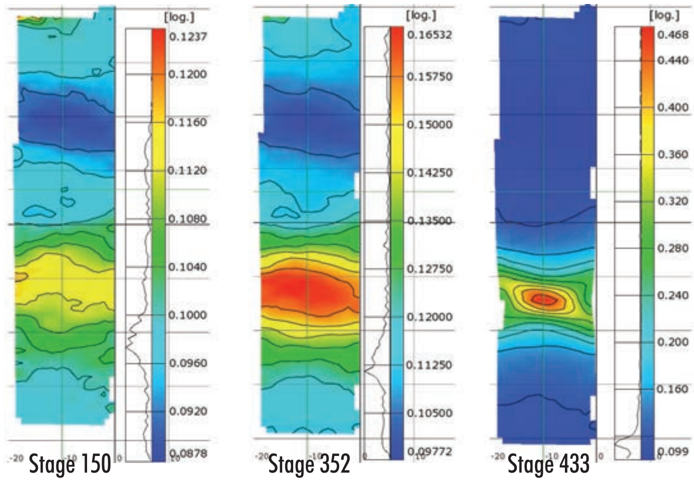


Figure 5. Major strain evolution during the test

Different graphs and curves can also be constructed, for instance, from the surface strain data with respect to time. Overlaid major strain evolution from the centre line of the test specimen can be seen in Figure 6a. Strain evolution is readily visible and can be pointed to a certain location on a surface. The uniform elongation is dominant at the early stages of the test and that can be seen as the uniform elevation of the curve. The onset of the diffuse necking shows a clear rise in the major strain values on a certain section length and outside that section the strain rises only moderately. After a certain point, the diffuse necking evolves to a localized necking which can be seen as a sharp peak as the strain increases rapidly between the taken images. The strain levels outside the necking zone remain the same which also indicates a strain concentration in a very small section length.

The strain evolution can also be followed at several points on the surface with respect to time. The point of the maximum strain is marked as 1. The maximum strain point is selected from the image before the breakage. Points 2 and 3 are positioned 10 mm and 20 mm from Point 1 to represent the points in the areas which exhibit the diffuse necking and uniform elongation. In Figure 6b, the strain evolution during the test is seen. The strains in all three points increase similarly up to the onset of the diffuse necking. The strain values at Points 1 and 2 start to deviate from that at Point 3 when a strain level of 0.10 is reached, which indicates diffuse necking.

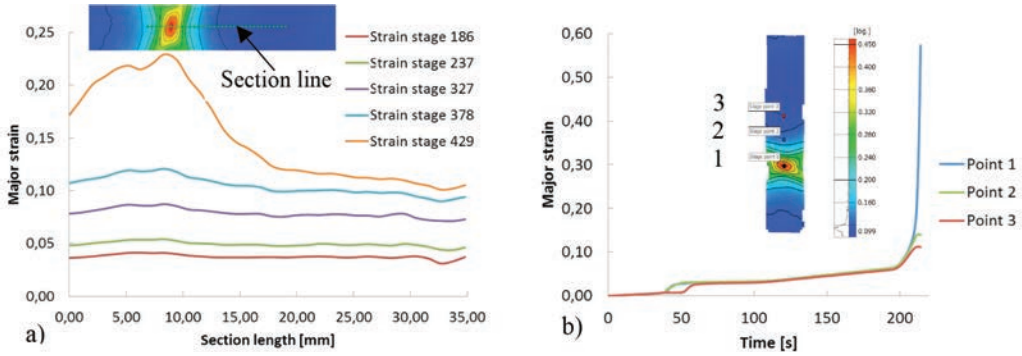


Figure 6. a) Major strain per section length per different stages, b) Major strain evolution at selected points

Further evolution from the diffuse necking to the localized necking can also be seen from the curves in Figure 6. The strain outside the necking zone, Points 2 and 3, cease at the reached level and inside it, Point 1, rapidly increases. Figure 7a shows the strain rates of Points 1 to 3 during the test. The increase in the strain rate can be seen before 0.10 strain. In that same strain level the strain rate of Point 2 levels while at Point 3 it starts to decline. Points 1 and 2 are within the diffuse necking area. After 0.18 strain the strain rate at Point 2 starts to decrease which indicates the start of the local necking in the area around Point 1. The bifurcation of the strain levels in the different points of the specimen can therefore be used to indicate the required strain for the beginning of the diffuse and localized necking. The higher order derivatives of the strain could also be used to evaluate the starting point. The thickness reduction, i.e. thinning, of the specimen cross section is depicted at the same points also in Figure 7b and that could be used to analyse the onset point of the diffuse and local necking.

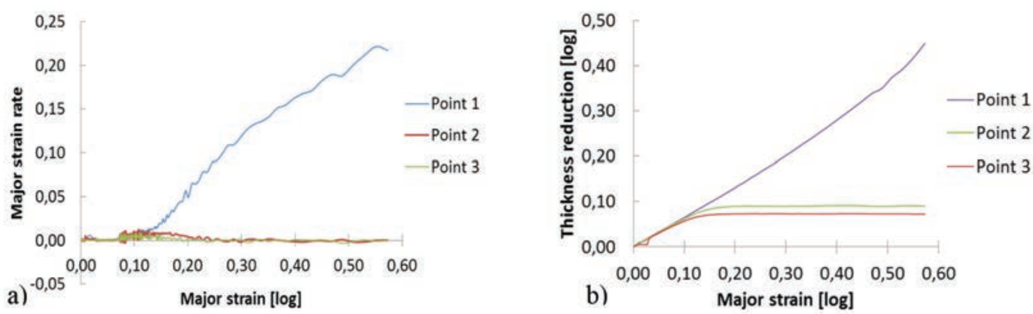


Figure 7. a) Major strain rate at selected points, b) Thickness reduction at selected points

Construction of Stress-Strain Curve

A validated numerical analysis is often used as part of the design process in several branches of industry. In metal forming applications some fundamental information about the characteristics and behaviour of the material is needed. Basic uniaxial stress-strain data is acquired from a normal tensile test. The data from tensile test is normally accurate enough up to the point of the uniform elongation. Stress and strain states in real forming situations are rarely so simple that the basic uniaxial stress-strain data could completely define the behaviour of the material. In order to improve the quality of the material data, true stress-strain data is constructed, which takes the deformation of cross section of the test specimen into account during the test and that way better describes the behaviour of the material [Coppieters *et al.* 2011].

Modern, complex forming processes require material data that describes, with reasonably accuracy, the stress-strain path in order to get more and more accurate forming simulations. This could be acquired from an optically assisted strain analysis of a normal tensile test. To further enhance the accuracy, for instance, the anisotropic behaviour of the material could be taken into account. Also different methods are available for testing various stress strain states, for instance a hydraulic bulge test, to gain more detailed data.

Figure 8a illustrates the changes in the geometry of the specimen during the tensile test. In Figure 8b deformations of the surface at different stages of the test are shown. It also shows the section line from where the data is acquired. Radius R1 in Figure 9 represents the curvature of the surface at the final stage of the test.

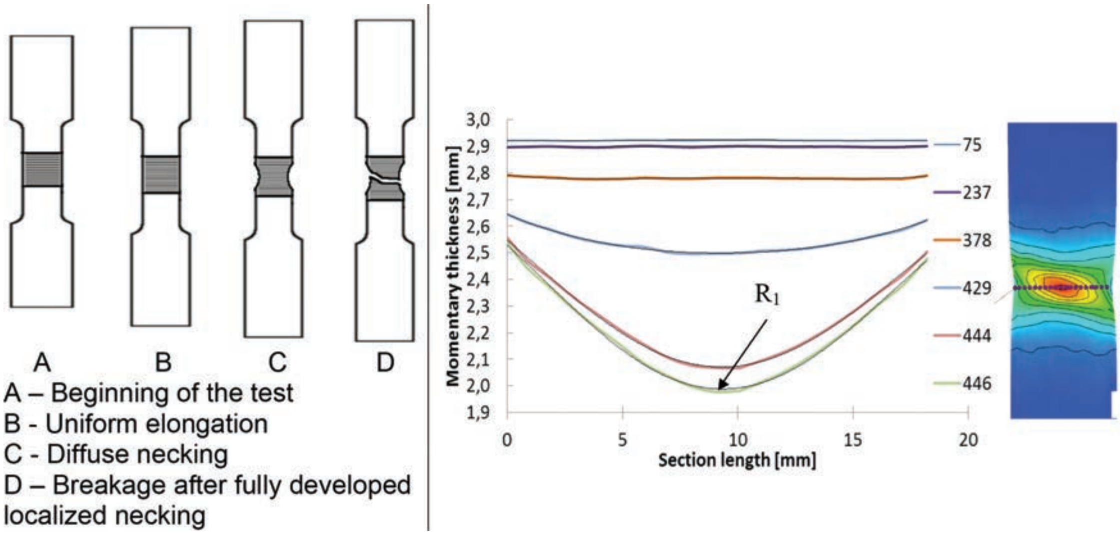


Figure 8. a) Changes in geometry of specimen during tensile test, b) Surface deformation in thickness direction

The biconcave shape represents the true area of the specimen during the test. The area is calculated from the thickness reduction and the minor strain data values at the point of the maximum major strain. The thickness reduction is plotted along the horizontal line and the 4th order polynomial fitting is made to acquire equation for it. The fitted equations are different for each stage of the test as can be realized from Figure 8b. The area below the fitted curve is presented in a general form in equation 1.

$$A_{conc} = \int_{0}^{width} Polyfit(x)dx \tag{1}$$

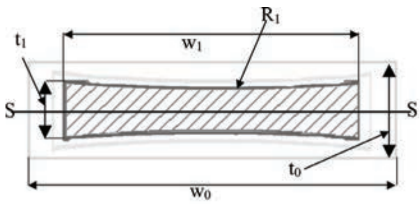
An ideal, momentary rectangular area is calculated with Equation 2 using a momentary width and maximum thickness values. The width values are based on the average minor strain of the section line.

$$A_{ideal} = W_1 * t_1 \tag{2}$$

The Momentary DIC acquired area of the symmetrically biconcave section is then calculated with Equation 3.

$$A_{DIC} = 2 * A_{conc} - A_{ideal} \tag{3}$$

Symmetry is assumed through the thickness so that both sides have same contour as seen in Figure 9.



Analysis parameters
t_0—Thickness of specimen at the beginning
W_0—Width of specimen at the beginning
t_1—Thickness of specimen at the end
W_1—Width of specimen at the end
R_1—Radius of concave section

Figure 9. Biconcave shape of cross section during the tensile test

The momentary stress is then calculated with corresponding force data from the load cell using Equation 4.

$$\sigma_{DIC} = \frac{F}{A_{DIC}} \tag{4}$$

From that data, the geometry corrected stress strain curve can be constructed. The DIC based stress strain data is seen in Figure 10, which also shows the stress strain curves based on Aramis software analysis, power law fit and tensile test data. The stress strain curve from Aramis software is based on the single point strain data, which is selected from the surface and load cell information from the tensile test machine. The power law fit is done with parameters based on the tensile test machine analysis. The data from the tensile test is transformed into a true stress strain from the engineering stress strain data before adding it to Figure 10. Therefore, the true stress strain curve based on the tensile test data in significantly shorter than the others.

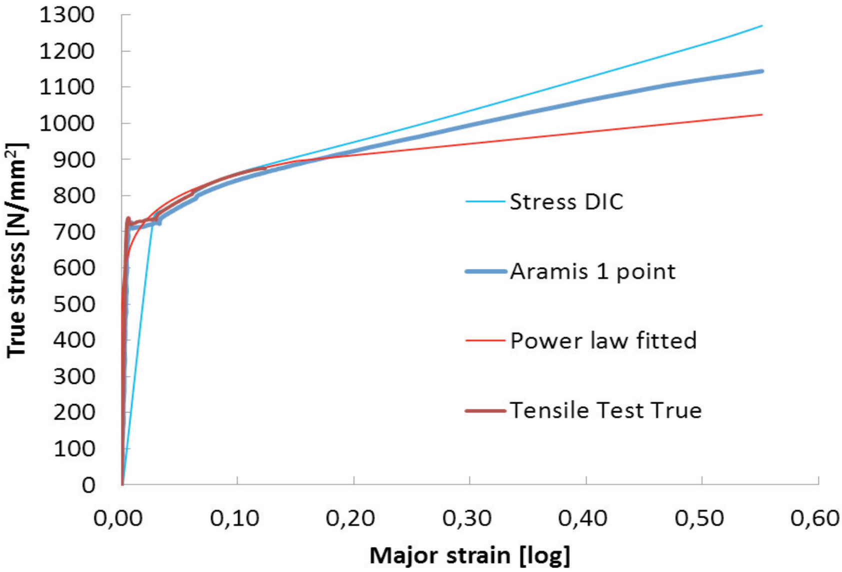


Figure 10. Comparison of various stress strain curves

DISCUSSION AND CONCLUSIONS

In this paper a method for estimating the momentary true area of the tensile test specimen and subsequently the true stress affecting the specimen is presented. The method is based on data acquired by the test equipment and using digital image correlation software. The cold formable steel of strength class of 650 N/mm^2 was tested and analysed with the presented method. The onset of two necking types, namely the diffuse and localized necking, were analysed and identified by the bifurcation of the strain data at selected points on the specimen surface near the maximum strain point. The stress strain curve obtained by the presented method is higher than the ones from the Aramis software and power law fitted from the tensile test data. The assumptions of the volumetric constancy and strain surface symmetry were made. The cross section of the breakage area is also assumed to be perpendicular to the length axis of the specimen. In reality, the breakage is not straight but normally tends to be at an angle with the horizontal line. The angle depends on the tested material. For the stress analysis the angled breakage surface means that the shear stresses are affecting that area and the total axial stress is something else than assumed. The validity of the cross section area analysis based on surface deformation data could be tested further by a discontinued tensile test series. A number of specimens would be strained to a predefined amount of elongation and by cutting the specimen at the maximum point of the major strain, the cross section area and shape could be analysed with optical measurements. For use in simulation the DIC assisted stress strain curve is yet to be tested and it is one of the topics for further studies with this technology.

ACKNOWLEDGEMENTS

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BOLTS FOR SLIP RESISTANT JOINTS IN TOWERS FOR WIND TURBINES

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ABSTRACT

This paper deals with a new solution to connect various segments in a tubular steel tower for wind power plants. Tests are performed to determine appropriate bolts for such connections. They focus on checking the development of pretension forces in the bolts during a period of one week. The behaviour of four different types of bolts is described and evaluated. For one type of bolt the force reduction is monitored for two different clamping lengths. Finally, recommendations for further tests are given.

INTRODUCTION

After the earthquake in Japan and the destruction of the nuclear power plant in Fokushima renewable energies turned into a topic more interesting than ever. Especially in Germany, where politicians decided to stop energy generation by nuclear power within the next decades, but also in other European countries, wind energy becomes more attractive and rewarding. In cooperation with various European universities and industrial partners Luleå University of Technology developed a new solution for in-situ assembling of tubular steel sections in a wind turbine tower by the use of slip resistant connections. This work has been carried out in the course of research of Heistermann 2011.

BACKGROUND

Steel tubular towers for wind turbines, as shown in figure 1, can be up to about 100 m high. This height is limited by transportation matters on the one hand, so that the diameter of the tower sections may just reach 4.3 m, as well as the fatigue endurance in the flange connections on the other hand. Until today, flange connections are the most common solution to assemble the tower segments, cp. figure 2.

Figure 1. 80 m high tubular steel tower from Martifer, Portugal (Veljkovic *et al.* 2012)

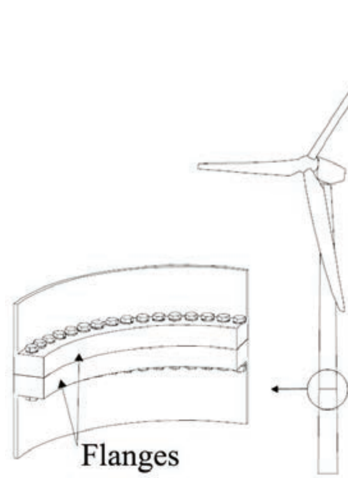

Figure 2. Bolted L-flange connection (Seidel 2001)

Depending on the fabrication process this connection detail has a fatigue class with ranges between 36 and 71, defined by Eurocode 3. Instead of using flange connections the use of slip resistant connections with open slotted holes is suggested, see figure 3. This will raise the fatigue detail category of the connection up to 112, see Veljkovic et. al. 2012.

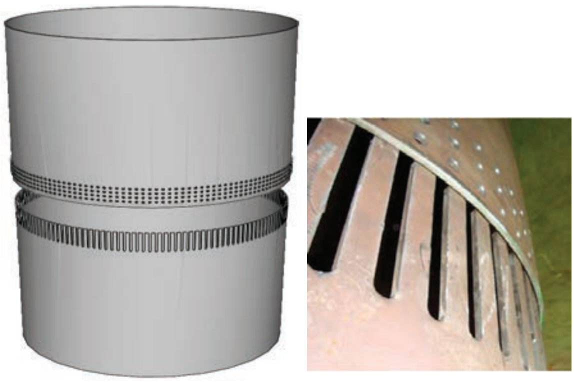

Figure 3. High Strength Friction Grip Connection (Veljkovic *et al.* 2012)

The design can then simply follow the rules for slip resistant connections according to Eurocode 3 instead of using more complex models for flange connections, developed by Petersen 1998 or Seidel 2001.

To ease the construction and maintenance of the tower, the behaviour of various bolts is analysed. Herein the crucial point is the loss of pretension in the bolts.

Loss Of Pretension

In a slip resistant connection the loss of pretension in the bolts equals the failure of the connection when it reaches a certain level: As soon as the bolts do not provide the pretension force any more, which the connection has been designed with, slip may occur and the connection fails. Therefore, it is important to understand the behaviour of the bolts and be able to calculate changes in bolt force.

This reduction of bolt force is a well-known phenomenon and usually divided into 3 phases: First, initial loss of pretension takes place. This happens within the first couple of seconds after tightening and is mainly depending on the tightening process. Then short term relaxation appears. This is said to occur during the first twelve hours following the joint assembly. After this longterm relaxation starts and continues asymptotically.

Bolts

Four different types of bolts have been used in this investigation: Tension Control Bolts (M30), Huck BobTail lockbolts (M20), standard bolts in combination with NordLock washers (M30) and Friedberg HV Rändel (M20). The last one differs from the others, as it is a press fitted bolt. Photographies of these bolts can be found in figure 4.

Figure 4. TCB, Huck BobTail lockbolt, standard bolt with NordLock washer and Friedberg HV Rändel (Heistermann 2011)

The Huck BobTail lockbolts have even been checked in two different lengths to see the influence of the shank length. The longer ones with a clamping length of 40 mm are called "long", whereas the ones with a clamping length of 18 mm are named "short". Until today it is common practice to use extension sleeves in order to raise the thickness of the clamping package, as longer bolts are assumed to loose comparatively less pretension.

METHODS

To follow the development of the bolt forces, the strains in the bolt shanks have been measured by the help of inserted BTM-6C strain gauges. For inserting the strain gauges a drill hole of 2 mm in diameter is necessary, cp. figure 5. After gluing the strain gauge into this hole, which starts from the bolt head, a calibration of the bolt is performed. By this it will be possible to directly translate the strains in the bolts measured during testing of the connection into bolt forces.

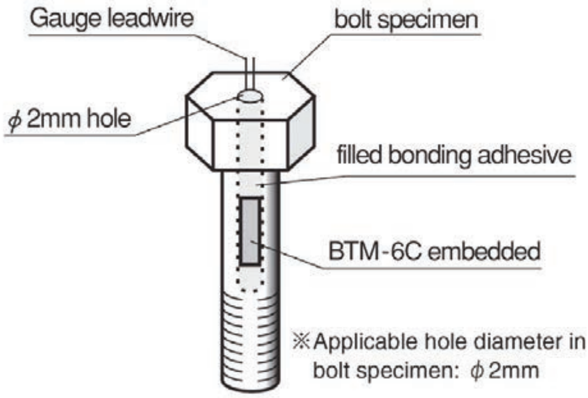

Figure 5. Position of strain gauges in the bolts (Tokyo Sokki Kenkyujo Co. Ltd.)

TESTS

As a monitoring of the development in bolt forces so-called relaxation tests were performed on all four types of bolts. For this, a main plate and a cover plate were joined by three bolts, cp. figure 6. The latter one functions as a substitute for single washers to facilitate the assembly in the actual tower during final application of the bolts. The plate thickness varied due to the length of the bolts. Then the bolts were tightened and the development of strains was constantly monitored for one week.

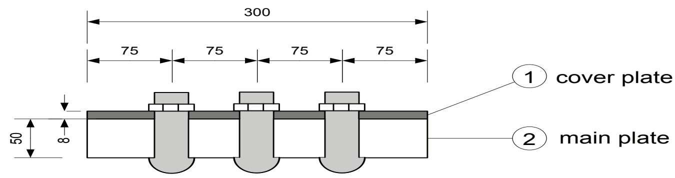

Figure 6. Specimen for relaxation tests (Heistermann 2011)

All plates have the same dimensions of 100 mm width and 300 mm length. The diameter of the holes varies between 33 mm for M30 bolts and 23 mm for M20 bolts. Just for the fitted Friedberg bolts the hole diameters were smaller; 20.1 mm. Since the loss of pretension depends on several factors, such as bolt and plate material and also thickness of the plate coating, a variation in coating thickness of the plates is also taken into account: For each type of bolt a specimen with two, one or zero painted surfaces is tested (R2, R1, R0). For this primer an ethyl silicate zinc rich paint with an average thickness of 80 μm is used.

RESULTS

Tables 1 to 4 show the losses of pretension in all tested bolts over a period of 1 week. The values are given in percentage of the maximum pretension force, which was achieved directly after tightening.

Table 1. Loss of pretension Tension Control Bolts in % of the maximum pretension force

	TCB		
	R2	R1	R0
after 10 seconds	2,77	3,03	2,82
after 60 seconds	3,61	3,73	3,24
after 10 minutes	4,50	4,35	3,41
after 1 hour	5,25	4,89	3,58
after 12 hours	6,40	5,76	3,90
after 12+2min	6,40	5,76	3,90
after 12+2 hours	6,47	5,82	3,92
after 24 hours	6,72	6,02	4,00
after 36 hours	6,91	6,16	4,05
after 1 week	7,52	6,53	4,02

Starting the test by tightening the bolts and the adjacent performance of the test by constantly monitoring the strains in the bolts could be executed without any problems. As expected in advance, the specimens with more coating show higher losses of pretension force, see table 1. Also the development of bolt force was foreseen: First the drop is relatively high, then it slows down and becomes asymptotical.

Table 2. Loss of pretension for standard structural bolts with NordLock washers in % of the maximum pretension force (Heistermann 2011)

	NordLock		
	R2	R1	R0
after 10 seconds	1,84	2,58	1,80
after 60 seconds	4,74	3,73	4,50
after 10 minutes	5,68	5,73	6,52
after 1 hour	6,79	7,48	7,40
after 12 hours	7,74	8,59	8,23
after 12+2min	7,74	8,59	8,23
after 12+2 hours	7,79	8,65	8,27
after 24 hours	7,91	8,76	8,35
after 36 hours	8,06	8,93	8,45
after 1 week	8,50	9,49	8,77

When tightening the standard bolts with NordLock washers problems arose due to the fact that the washer pairs overrun. This could be avoided by tightening the bolts very carefully.

The measured losses of pretension, cp. table 2, meet the expectations in so far that their course starts quickly and slows down by time. But it does not show a clear trend with regard to the coating thickness of the plates. The losses measured after one week end up in about the same range for all specimens.

Table 3.Loss of pretension for Friedberg HV Rändel in % of the maximum pretension force (Heistermann 2011)

	Friedberg		
	R2	R1	R0
after 10 seconds	31,31	3,26	1,08
after 60 seconds	77,71	19,10	33,97
after 10 minutes	78,90	21,69	35,20
after 1 hour	93,04	22,92	35,43
after 12 hours	93,24	22,93	35,70
after 12+2min	93,24	22,93	35,70
after 12+2 hours	93,25	22,92	35,71
after 24 hours	93,26	22,90	35,71
after 36 hours	93,30	22,91	35,74
after 1 week	93,36	22,94	35,80

Table 3 shows the results from the tests of Friedberg HV Rändel bolts. These are press-fitted bolts, which have to sit very tightly in the according hole. Installing the bolts proved to be complicated, since due to the inserted strain gauges a special carefulness was necessary. The bolts should be pulled into the hole by turning the nut from the other end. When doing so with the R2-specimen, a breaking noise appeared, but a failure of the specimen was not visible.

However, the measured values for R2, as shown in the first column of table 3, clearly indicate that the bolts must have been broken. A loss of pretension of more than 90% is not reasonable.
Also the measured data for specimens R1 and R2 seem not reliable: such an escalate within the first couple of minutes after tightening is inadequate.
In contrary to the data of Friedberg HV Rändel bolts, the measured values for Huck BobTail lockbolts show reasonable courses from the beginning until one week after tigthening, cp. table 4. But they as well do not meet the expectation that thicker surface coating leads to higher losses of pretension.

Table 4. Loss of pretension for long and short Huck BobTail lockbolts in % of the maximum pretension force (Heistermann 2011)

	Huck long			Huck short		
	R2	R1	R0	R2	R1	R0
after 10 seconds	18,32	23,65	23,00	37,85	42,17	33,38
after 60 seconds	17,80	23,66	23,34	38,24	42,17	38,07
after 10 minutes	18,15	23,69	23,39	38,53	42,21	38,58
after 1 hour	18,63	23,81	23,63	38,97	42,37	39,06
after 12 hours	19,21	24,24	24,09	39,68	42,90	39,83
after 12+2min	19,21	24,25	24,09	39,68	42,90	39,83
after 12+2 hours	19,24	24,28	24,12	39,73	42,95	39,87
after 24 hours	19,38	24,40	24,25	39,45	43,10	40,02
after 36 hours	19,46	24,51	24,34	39,64	43,26	40,19
after 1 week	20,01	25,01	24,82	40,41	43,93	40,85

Comparing the two types of lockbolts, the longer bolts loose much less of their actual pretension force than the shorter ones. The losses in percentage for short Huck BobTail lockbolts are about double as high as for the long bolts.

CONCLUSIONS

1. The tightening methods for standard bolts with NordLock washers and Friedberg HV Rändel are not practicable for larger numbers of bolts.

2. The measured losses for standard bolts with NordLock washers seem to be on a proper level. However, further tests should be performed to ensure that the surface coating definitely has no influence.

3. Friedberg HV Rändel should be tested further to check whether they are applicable as prestressed bolts.

4. The data of the TCB specimens is reasonable and can be extrapolated for longer periods. After e. g. 20 years, losses of 10,5% (R2), 8,8% (R1) and 4,8% (R0) will be attained.

5. Long Huck BobTail lockbolts show less loss of pretension than the short ones. However, further tests should be performed to ensure that the surface coating definitely has no influence.

6. The tests with Tension Control Bolts show the best results. However, due to the limited number of monitored bolts, further tests are strongly recommended.

ACKNOWLEDGEMENTS

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STEEL STRUCTURES

INTERACTION OF NON-METALLIC MEMBRANES WITH SUPPORTING STEEL STRUCTURE

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ABSTRACT

Fabric/foil membranes used for roofing of various canopies/shelters and their integration with supporting steel structure are described. The approaches for global analysis of the systems, their accuracy and influence of important parameters such as prestressing and rigidity of the membrane perimeter are discussed. Requirements concerning possibility of separate modelling of membranes and supporting steel structure are analysed for practical examples. Instead of specialized software, attempt to use common engineering software with shell and linear elements in geometrically nonlinear analysis (GNA) proved to result in acceptable solutions. Prestressing of the membranes and connecting linear elements are essential prerequisite of proper behaviour of these structures. Therefore, the paper also describes various measurement methods of internal forces in prestressed elements, both linear ones (rods, ropes) and membranes. Finally, some recommendations for the analysis and realization of non-metallic membrane members with prestressed peripheral elements and a steel supporting structure are given.

INTRODUCTION

The visual expression of structures is becoming more and more crucial not only for unique structures but also for common ones, based on availability of novel structural elements. Notable tensile surface structures are required by architects, designers and developers and corresponding new forms are being developed, as tensegrity and tensairity structures (see e.g. Lewis 2003, Pauletti and Brasil 2003, 2005, Wakefield 1999). Specialized companies (e.g. Base Structures Ltd., Tension structures.com, Mehler Ltd., TechArchitects sro., and others) have realized several unique tensile structures using fabric/foil membranes in the last few decades.

Fabric/plastic membranes were traditionally used for temporary structures and in “warm” countries. With increasing knowledge about new materials, as glass fabric coated by PTFE or silicon, polyester fabric + PVC + PVDF, polyester + titanium dioxide, expanded PTFE coated by fluoropolymer (TENARA®), or foils from ethylen-tetrafluoretylen (ETFE) or tetrafluoroethylene-hexafluoropropylene-vinylidene-fluoride terpolymer (THV) and respective technology (Seidel 2009), such structures are becoming frequently used in standard situations (e.g. Foster and Mollaert 2004). However, complex analysis of membrane structures in interaction with steel structure (carbon/stainless steel perimeter elements) is rather demanding.

Supporting steel framework forms usually an integral part of the membrane structure. Steel elements are used as anchor, perimeter, valley and ridge ropes and also as stiff load-bearing structure and anchorage. In design, with the exception of commercial software packages (MEMBRANE NDN, EASY technet GmbH) purpose-made software analyzing separated membranes alone is commonly used (e.g. STRAND), providing the resulting forces and deflections for analysis of a supporting steel framework. Such procedure, however, leads to considerably misrepresented results.

The present paper, therefore, deals with membranes integrated into steel structure and design possibilities when using commonly available software (SCIA Engineer, Comsol Multiphysics). The study assumes a routine prestressing of membranes and deals with realistic methods of assembly and prestressing, level of which is a fundamental prerequisite of the structural behavior. The paper also describes experience obtained from extensive in-situ and laboratory measurements concerning prestressing of rope or rod elements.

MODELLING OF STEEL FRAMEWORK WITH MEMBRANE

Expansion of fabric/foil membranes both in permanent and temporary large space or just aesthetic structures, possibly shelters or canopies, has forced designers to integrate a membrane into load-bearing steel structure. There is a question, whether results of a separate membrane analysis may be introduced into steel frame analysis and vice versa. The shortages of such procedure are analyzed and techniques to their minimization are proposed in the following paragraphs. Only mechanically prestressed membranes having anticlastically curved shapes are treated here.

PRESTRESSING OF MEMBRANES AND CONDITIONS FOR CORRECT DESIGN

The shape of a membrane structure must ensure tensile straining under all loading conditions. The prestressing of a membrane is an essential requirement providing geometrical rigidity and ability to take over a loading producing reverse straining.

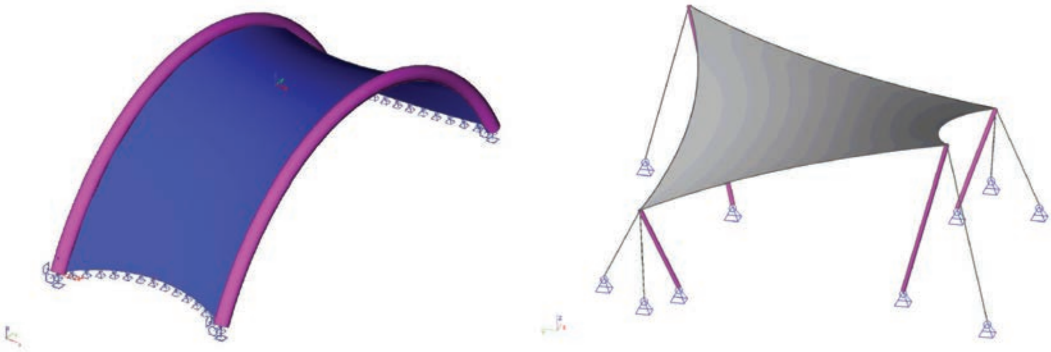

Figure 1: Examples of an arch supported membrane and a HYPAR.

Prestressing may be introduced into the membranes according to erection methods, see Figure 1:

- either by extension of a membrane against stiff perimeter/ point supporting structure (arches, high points, ridge and valley configuration),
- or by extension of a membrane in two reverse curved directions among four points with two of them in higher position (HYPAR, hyperbolic paraboloid shape).

In practice, separate modeling of a membrane and steel framework is common. The membrane is designed by a specialized office and boundary data given to steel designer. However, the interaction between membrane and steelwork is obvious. Separate modeling may only be successful provided the design of the membrane considers geometry and rigidity of the supporting framework (arches, frames, pylons, perimeter elements, etc.). In case the support is not fully rigid and behaves elastically, the introduction of real rigidities is necessary, otherwise resultant membrane stresses and deflections are distorted and data provided to the designer of steel structure may include inaccuracies.

ANALYSIS OF MEMBRANES BETWEEN ARCH BEAMS

Such situation is demonstrated through comparing various models of a typical arch membrane spanning between arch edges, Figure 2. The membrane characteristics were taken as E = 1000 MPa, ν =0.25, t = 1 mm (membrane thickness). Two models were investigated: first the steel tube arches built-in at supports both in and out of plane (the tubes of cross section 324x25 [mm]), second the fully rigid support along all arch shape.

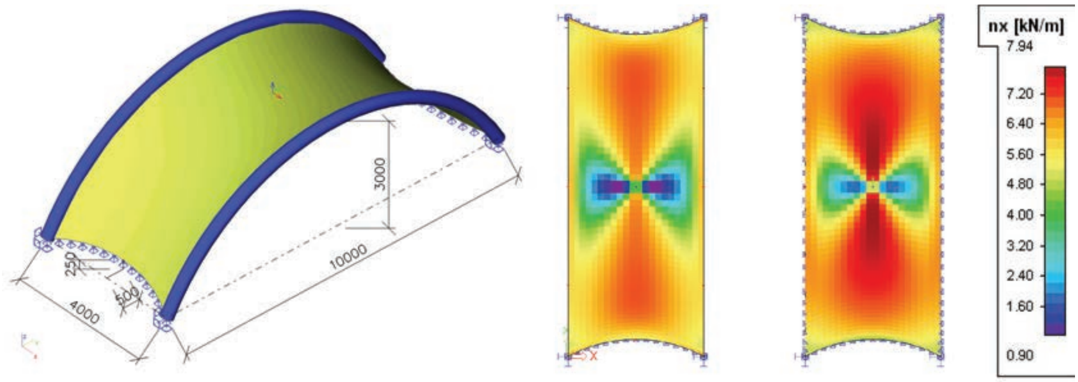

Figure 2: Example of an arch membrane between arch beams.

Comparison of unit transverse membrane forces n_x [N/mm] in case of given arch beams (left) and rigid support (right).

Under transverse horizontal prestressing $\varepsilon_x = 0.004$ (i.e. 4 N/mm) and vertical (snow) loading of 1 kN/m^2 the maximal transverse unit force for stiff support is 12 % higher in comparison with the tube arches case, and maximal membrane deflection attains 59 % only. Taking such values for a separate analysis would lead to incorrect, lower prestressing of the membrane. Nevertheless, it is not easy to simulate flexible arch tubes by simplified elastic linear supports, because the value of rigidity along the arches is changing (e.g. in arch supports approaching infinity). For example, taking uniform rigidity of the given tube arch in horizontal direction (709 N/mm) according to maximal deflection due to uniform horizontal loading acting on the arch ($u_x = 1.41$ mm), the resulting transverse horizontal deflections for prestressing are higher by approximately a factor of 2.2, see Figure 3.

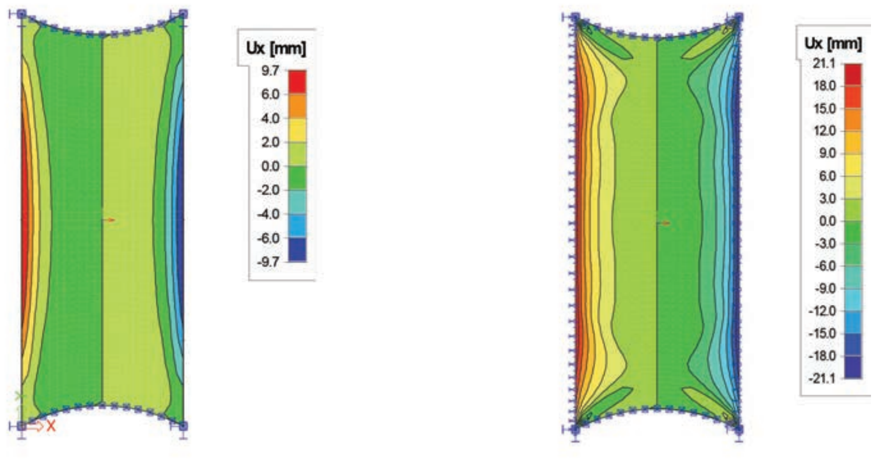

Figure 3: Transverse membrane deflections (u_x) in tube arch assembly (left) and in the membrane under simplified elastic linear support (right).

Parametric studies with various arch beams, their spans, cambers and dimensions clarified the support rigidity and will be published later.

ANALYSIS OF ERECTION METHODS OF HYPAR MEMBRANES

Erection methods and ways of introduction of prestressing have substantial influence on resulting forces, stresses and deflections. Designing the steel framework, not only membrane reactions but also membrane rigidity and deflections from prestressing have to be allowed for.

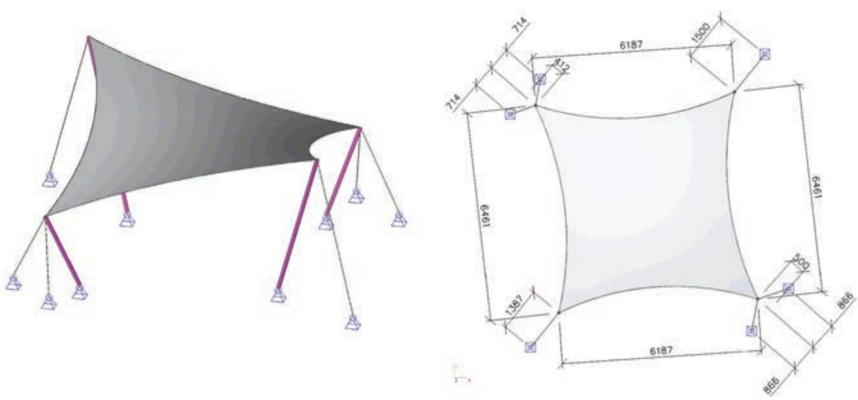

Figure 4: Example of a HYPAR membrane, geometry.

Example of prestressing a simple membrane (Figure 4) is illustrated. The required prestressing, similar to the arch membrane above (about 1 N/mm in all directions), is reached by two ways: first the perimeter ropes are shortened, implicating top of pylons moving into membrane center (max. approx. 33 mm); second the prestressing of outer guys moves the top of pylons in an outer direction (max. approx. 29 mm), Figure 5. Numerical modelling of complex membrane structures always requires consideration of the erection methods, resulting in final configuration of the structure.

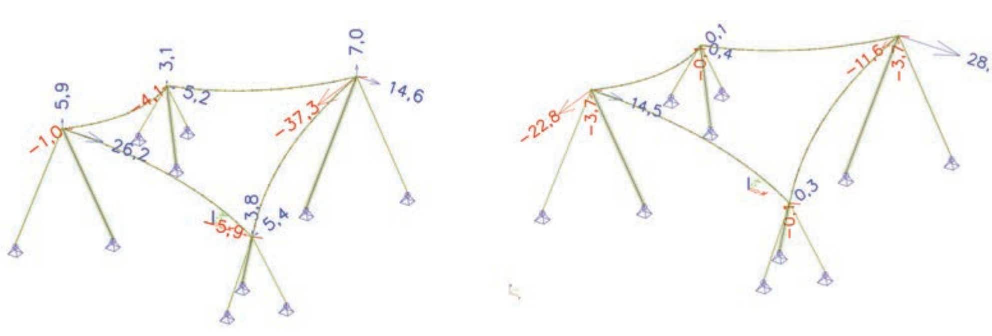

Figure 5: Resulting deflection of HYPAR perimeter [mm].
Deflections due prestressing of perimeter ropes (left), and due to inclination of pylons (right).

GLOBAL ANALYSIS OF THE MEMBRANE/STEELWORK INTERACTION

Nowadays routine software dealing with linear/planar elastic finite elements enables joint modeling of membranes with steel framework. It is always necessary to introduce prestressing as a fundamental load case (and permanent action), before any other variable loadings. The prestressing represents basic configuration of the structure, capable of bearing other loadings. Prestressing has to ensure only tensile stresses within the membrane, which may roughly be verified by linear analysis (LA). Calculation of action combinations requires geometrically nonlinear analysis (GNA) with proper initial geometry and appropriate numerical iterative procedures (e.g. N-R, arc–length method, etc.). It is advised to start with minimal number of freedom releases and strut nonlinearities (e.g. tension members only). As far as the model is stable and converges, these may be supplemented. More complicated assemblies always require coherent knowledge of behavior of all elements of the designed structure.

SCIA Engineer software is commonly used in Central Europe countries and, therefore, the GNA results were compared with results of software COMSOL Multiphysics - Structural Mechanics module (for both see References), which analyses physical tasks expressed through partial differential equations (PDE) by FEM.

First simple square membrane (3000x3000x1 [mm]) prestressed in all directions by shortening $\varepsilon = 0.001$, second arch shape membrane (plan dimensions 10x3 [m], camber 2 m) with $\varepsilon = 0.004$ were studied, both loaded vertically with 1 kN/m2.

The greatest resulting differences were found to be in the second case (ratio COMSOL/SCIA for internal forces 122 %, deflections 114 %, when the standard shell elements were used, and 133 % and 159 % respectively, when tension membrane elements were used), see Figure 6.

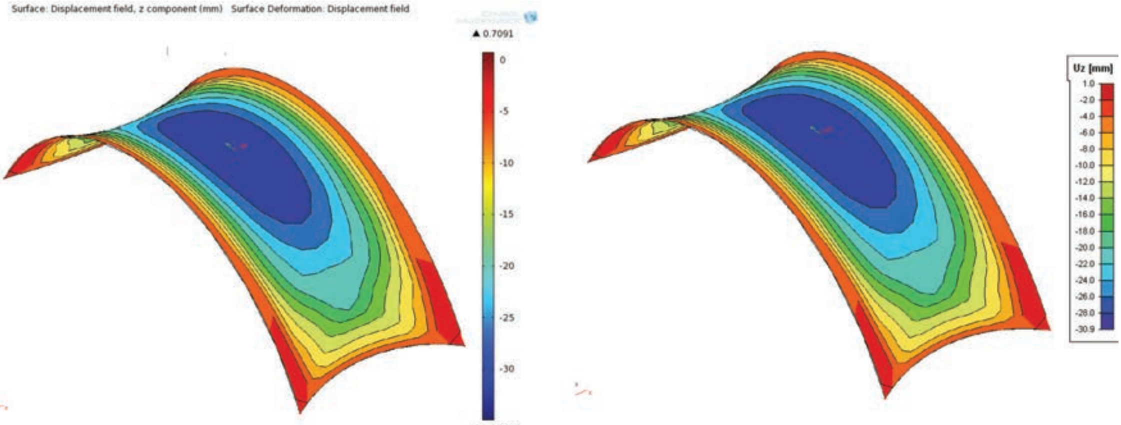

Figure 6: Vertical deflections of the arch shape membrane [mm].
Results of COMSOL software (left) vs. SCIA Engineer software (right).

It was concluded, that using shell SCIA elements the results provided good agreement in all values of internal forces and deflections and the software is capable of credible modelling.

MEASUREMENT OF PRESTRESSING DURING ACTIVATION AND UNDER OPERATION OF THE STRUCTURE

The correct level of prestressing in all structural parts (guys, ties, perimeter ropes, membranes) is fundamental. Therefore, the methods of prestressing measurements are briefly described:

Measurement Before Commencement and During Erection

Strain gauges are frequently used for this purpose, requiring full bridge (gauges on both surfaces of the element to exclude bending moment and compensating ones to eliminate temperature influence). The gauges have to be installed and read before prestressing to establish initial values (unloading already stressed element is usually not feasible). Some difficulties arise in placement of gauges at ropes. The only suitable position is at circular section just at or behind anchor or turnbuckle. However, stresses are not uniformly distributed due to change from sleeve to full cross section and their concentration on surface is evident. Either calibration of the measurement or insertion of load cells is necessary (provided aesthetics or detailing enable such a solution).

Transducers of various types (mechanical, inductive etc.) are option to the above. Inserted load cells (or calibrated manometers of hydraulic devices) are useful in case of final activation of elements as long as no other straining is induced by introduced prestressing.

Measurement on Already Prestressed Structure

For this purpose, **frequency measurement** may be used, consisting of determination of natural frequency of the element transverse vibration (Kolousek 1967). The axial force may be derived from theory of strings with respect to bending stiffness, axial force, sag, boundary conditions (e.g. boundary elasticity may complicate the results), placement of turnbuckles or other prestressing elements violating uniform mass along the element. In case of two hinged element the simple formula for axial force at j - frequency reads:

$$N_{(j)} \quad \mu \left(\frac{2Lf_{(j)}}{j} \right)^2 \quad \left(\frac{j\pi}{L} \right)^2 EI \qquad (\text{with } N \text{ in } \frac{\text{kgm}}{\text{s}^2}) \tag{1}$$

where E [kg/(ms^2)] is modulus of elasticity, L [m] length of element, μ [kg/m] mass, I [m^4] second moment of area and $f_{(j)}$ [s^{-1}] denotes j - natural frequency. In case of rigid end fixings the relation transforms to:

$$N_j \quad \left(\frac{\sqrt{|t^2 \quad 4yz|} \quad t}{4y^2} \right)^2 \tag{2}$$

with similarly

$$t \quad f_j \quad \frac{j}{L^2}\sqrt{\frac{EI}{A}}; \qquad y \quad \frac{j}{2L}\frac{1}{\sqrt{\quad A}}; \qquad z \quad \frac{j}{2L}\frac{1}{\sqrt{\quad A}} \; 4 \; \frac{j^2 \;\, ^2}{2} EIL^2 \tag{3}$$

and ρ density (for steel 7850 kg/m^3).

A special testing device may also be used to determine the axial force in the element from calibrated deflection-axial force relation. The instrument having saddles accommodating the measured element is shown in Figure7. The force induced by the jack to produce required deflection is measured through calibrated sensor. The PIAB branded device is designed and calibrated for certain range of rope diameters.

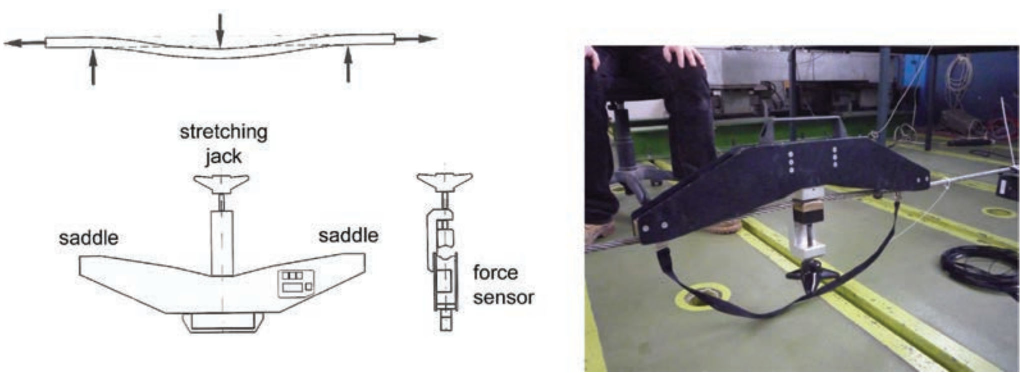

Figure 7: Scheme and photo of instrument PIAB RTM 20C for measurement of axial force in a loaded rope

Electromagnetic permeability measurements provide another method for the determination of axial force in an investigated element, applicable to rods. However, the material of the specimen needs laboratory investigation.

Membrane state of stresses may also be established by measurement. Producers of membranes have developed various instruments for this reason. The basic method represents biaxial measurement via transducers on a membrane sample, often used to determine respective modulus of elasticity. Second method works on the principle of frequency response of acoustic spectrum excitation in given boundary conditions. From the response, the natural frequencies are determined and in accordance with investigated material also the internal stresses. Another method is based on similar principle as the instrument for measurement of axial force in ropes described above. Measured is the deflection induced by a piston force of given value acting on a circular base.

Determination of internal forces in prominent structures often requires combination of the described methods. The choice is influenced by the element type, ratio of prestressing/resistance, anticipated value of prestressing, weather conditions etc.

EXPERIENCES WITH MEASUREMENTS OF INTERNAL FORCES

To evaluate the resulting internal forces in rod and rope elements, various methods of measurement were used and results compared.

Prestressing Force in a Steel Rod/Tie

The measured tie formed the bracing of a steel structure, Figure 8. The tie was prestressed to required force by a hydraulic system with pressure indication, enabling to determine the factual axial tie force.

Figure 8: Hydraulic system and strain gauges arrangement.

First used was the method described above, using strain gauges arranged in full bridge with thermal compensators. Secondly, frequency measurement was employed, using purpose-made instrumentation with triaxial accelerometer and digital data transmission (see Necas, 2010). After an impulse of loading, the response of natural frequencies was monitored, Figure 9. Resulting forces emerge from average values of three measurements.

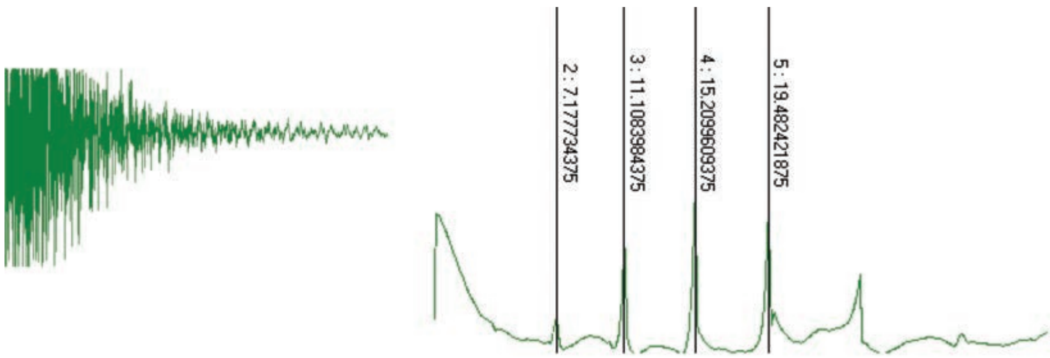

Figure 9: Record of acceleration perpendicularly to axis of tie (left).
Analysis of the vibration spectrum by the fast Fourier transform, FFT (right).

Evaluation of Equations (1) and (2) against experiments shows better agreement with the latter (i.e. rigid fixing in boundaries), where the tension force agreed in 97.3 % with measuring by strain gauges while it was 114.4 % with Equation (1). Frequency measurement also pointed out to need of using second and higher frequencies in the evaluation, as the first one is usually not legible. Within the measurement the nonlinear behavior and/or damping were not studied but were expected to be negligible.

Prestressing force in a rope

Special lab specimen with a 6-strand rope of 12.5 mm diameter with wires of 0.56 mm and strength 1570 MPa was prepared, with length of 4000 mm between swaged sleeves, Figure 10. One support was optionally equipped with a cantilever of given rigidity to create elastic propping. Tension from standard turnbuckle was measured by digital calibrated load cell. The measurements were performed by three methods, using: strain gauges, instrument PIAB, and accelerometer (details above).

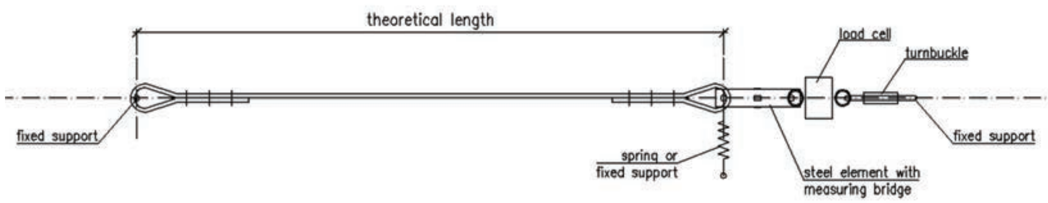

Figure 10. Scheme of specimen during measurement of tension force

From comparison of results the following points emerge:

- *Strain gauges measurement:* low forces (roughly up to 3 % of strength) give incorrect values; for higher forces the difference from correct value is constant and may be used as calibration coefficient.

- *PIAB RTM instrumentation:* for higher forces (roughly from 5 % of strength) the results are precise; average deviation (about 3.1 %) may be used as calibration coefficient.

- *Frequency method:* shows increasing deviation in the whole extent of measuring. Calculations using the first natural frequency give correct values for rigidly supported rope only, while in the case of one-sided elastic support the results are inapplicable. Force resulting as average from the first 5 frequencies for both conditions in supports is, however acceptable.

CONCLUSIONS

The results provide basic information concerning interaction of membranes with supporting steelwork and requirements for correct design from statics point of view.

The joint modelling of membranes with steel framework using common software (e.g. SCIA Engineer) is essential. Any membrane analysis has to consider geometric nonlinearity and sag of ropes from own weight. Use of a separate modelling requires taking into account mutual interaction of the membrane and steelwork, including method of activation of prestressing.

Methods for determination of internal forces described in the paper may be used during assembly and prestressing according to presented drawbacks. Results of majority of special rigs are in good agreement with reality.

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A SEMI-RIGOROUS APPROACH FOR INTERACTION BETWEEN LOCAL AND GLOBAL BUCKLING IN STEEL STRUCTURES

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ABSTRACT

With an increasing trend towards the use of higher strength materials, members in steel structures become more slender. The cross-sectional plate elements of such members also become slender, triggering possible interaction between local buckling of the flange and web elements and the overall buckling of the column. The paper proposes use of plate buckling response, in terms of in-plane load and axial deformation, as modified stress-strain curves for use in column analysis. These curves can be derived from numerical analysis of such plates or may be based on experiments, where available. When the rigorous ultimate strength analysis of such columns is carried out using numerical techniques such as the finite difference method, rapid solutions are obtained for an otherwise very complex problem. The paper includes a parametric study aimed at examining the behaviour of stiffened plate elements such as those used in box-girders.

INTRODUCTION

Stiffened panel construction has been widely used in steel box girder bridges for some time. It has been used for ship and aircraft plating for even longer. Following the collapse of the three steel box girder bridges in 1970, at Milford Haven, Melbourne and Koblenz respectively, much experimental and theoretical work was undertaken worldwide to study various stability problems associated with stiffened panels in compression. The strengthening effect of stiffeners on regular and arbitrarily stiffened plates has recently been studied by Liu and Wang [1], using finite elements. The problem of interaction between global and local buckling of stiffened plates, using a semi-analytical approach, was described by Brubak and Hellesland [2].

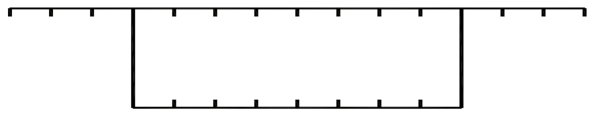

Figure 1 - Typical cross-section of a box-girder with stiffened plates

The study described in this paper relates primarily to stiffened plates that are subjected to uniform compression such as the bottom flange of the girder near a continuous support or the top flange towards the middle of the span (Figure 1). In either case, the flanges are subjected to compression, which could cause local buckling of the plate panels between the stiffeners or local and global buckling of the plate and stiffener combination.

It is recognised that the ultimate load of a thin plate and stiffener combination may be considerably above the load for local buckling of the plate. Determination of the ultimate load of plates is very distinct from that of finding the elastic buckling load. The problem is complicated by the presence of residual stresses due to welding and of initial geometrical imperfections. The distribution of residual stresses in welded plates, and their effect on the ultimate strength has been discussed by Dwight and Moxham [3], among others.

Stiffened compression panels are essentially anisotropic plate elements. An exact solution of the problem is likely to be very tedious. However, in many practical cases, the rigidity of a stiffened panel in the direction of longitudinal stiffeners is far greater than that in the transverse direction. The post-buckling behaviour of such a panel in such a case approaches that of a strut consisting of an individual stiffener together with an associated width of the plate, that is, there is no appreciable overall post-buckling reserve. This approach has been adopted here to study the effect of residual stresses due to welding and the effect of initial lack of straightness on the strength of stiffened compression panels covering the full range of slenderness ratios.

COMPUTATIONAL APPROACH

The computer program used here was developed originally for the study of composite columns in biaxial bending, and was general enough to analyse a wide variety of cross-sections including reinforced concrete columns, concrete encased steel stanchions, concrete-filled steel tubes, and bare metal sections, all of arbitrary shape. Non-linear stress-strain curves for constituting materials as well as any residual stresses can be included. The method, which is readily applied to stiffened plates in compression, has been fully described elsewhere [4]. A key feature of the method is that it is extremely fast when compared with finite element computations and yet provides similar level of accuracy.

INERACTION BETWEEN OVERALL AND LOCAL BUCKLING

For panels having a large number of closely spaced stiffeners, the inelastic behaviour of the stiffened plate can be approximated by that of a strut consisting of an individual stiffener and an associated width of the plate. A semi-empirical approach to take this into account was suggested by Vojta and Ostapenko [5] using an average stress-strain curve, defining the local behaviour of the plate, instead of using the material stress-strain relationship. Adopting the same approach, the average stress-strain curves selected in this paper are based on Ractliffe's experiments [6].

A compression panel of practical dimensions was arbitrarily chosen (Figure 2). It is assumed that the steel plating has evenly spaced longitudinal stiffeners spanning between cross-frames. The breadth to thickness (b/t) ratio of the plate panels between stiffeners was taken as 60. Although with this b/t ratio, the section chosen is more slender than would normally be used, it was selected so that the interaction of local and overall buckling could be included in this study. The dimensions of the stiffener were chosen so as to preclude lateral buckling of the stiffener. With this approach, the full strength of the stiffener is realised. The loading on the stiffened plates, resulting from the transverse loading on the box-section, is assumed to be uniaxial in the direction of the stiffened span with equal end eccentricities.

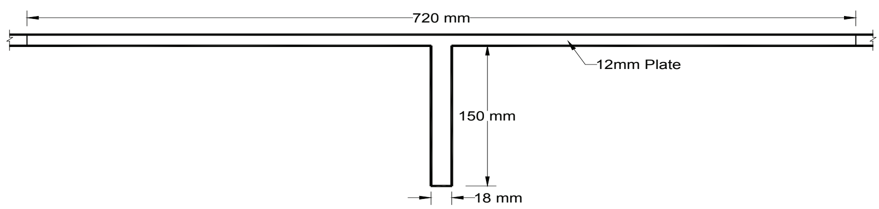

Figure 2 - Dimensions of the stiffened plate cross section

INITIAL LACK OF STRAIGHTNESS

In stiffened plates, two types of geometric imperfections may be commonly encountered. The first corresponds to the lack of straightness of the stiffener along its line of intersection with the plate. This overall out-of-plane deformation of the stiffener is denoted by Δ_0 (Figure 3). The second type of geometric imperfection, which may be called the ripple component of geometric imperfection, relates to the additional initial deformations δ_o in the plate elements measured with respect to a surface parallel to the surface defined by the stiffener out-of-plane deformations. In the present study only the stiffener out-of-plane deformations are considered. The ripple component of imperfection is usually small in magnitude compared with the stiffener out-of-plane deformations and mainly affects the local plate buckling strength. Thus the initial lack of straightness of the stiffener-plate combination is taken to be the same as the stiffener-out-of-plane deformation.

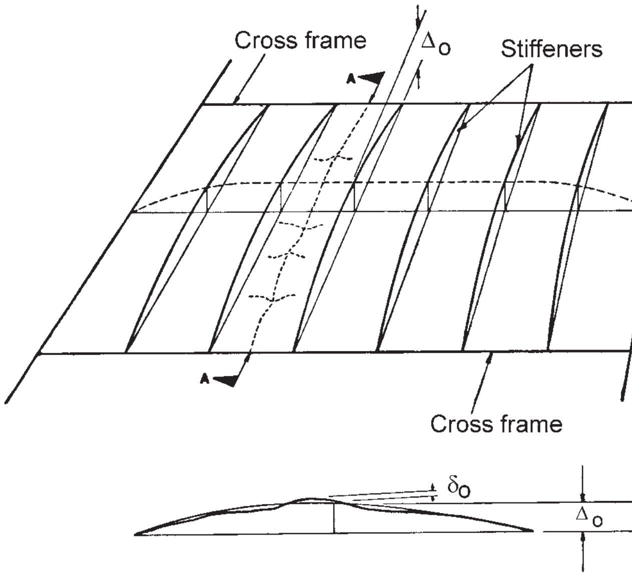

Figure 3 - Stiffener and plate ripple component of out-of-plane imperfections

In general, it is difficult to estimate the magnitude of the lack of initial straightness for practical cases. In the present study, for bending with the plate in greater compression than the stiffener (labelled Mode A bending) a value of $L/400$ has been used, where L is taken as the length of the stiffened plate between the cross-frames. For the other mode of bending, in which the stiffener has a greater compression than the plate (labelled Mode B bending), an initial lack of straightness of magnitude $L/600$ has been used. These are pessimistic compared with current international standards.

The calculated failure loads for different slenderness ratios are expressed as a fraction of the squash load. For the material of the plates chosen in this study, the modulus of elasticity is taken as 205000 N/mm^2 and the yield stress of the material is σ_y = 335 N/mm^2, resulting in slenderness ration for which Euler stress equals the yield stress as 77.7.

Figure 4 shows the variation of failure loads with slenderness when the stiffener-plate combination section is given an initial lack of straightness. The failure loads for Mode B bending, that is, failure by compression in the stiffener outstand are less than those for Mode A bending with failure by compression in the plate up to a slenderness ratio of about 150. For slenderness ratios greater than 150, Mode B results are fractionally greater than those for Mode A, in spite of a smaller amount of initial lack of straightness. The maximum loss of strength for Mode A bending is around 27% and for Mode B bending, approximately 47% compared with ideal elastic-plastic behaviour.

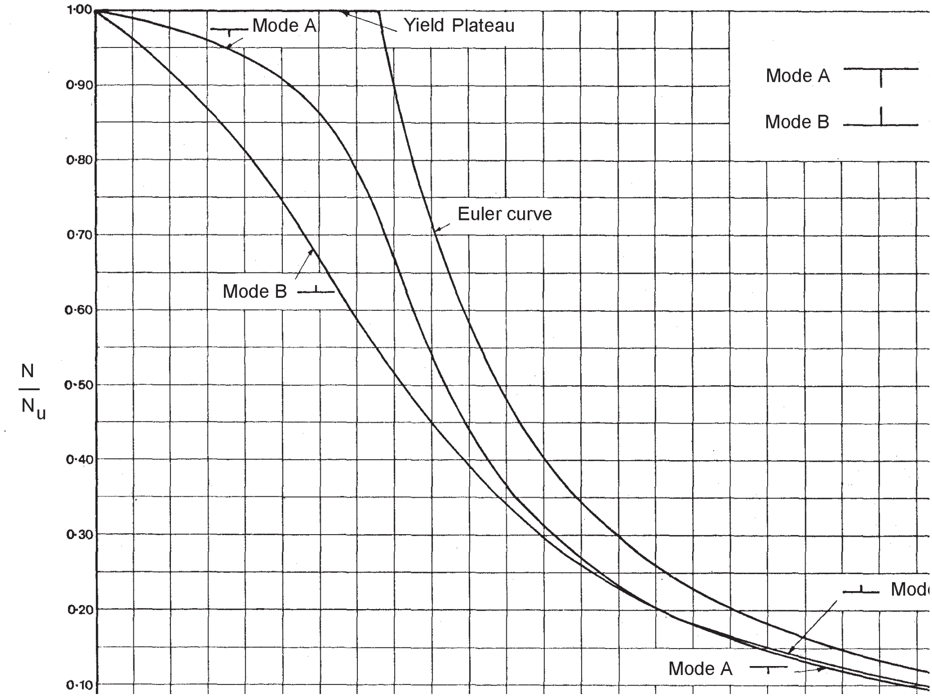

Figure 4 – Failure loads including effect of lack of straightness

RESIDUAL STRESSES

Dwight and Moxham [3] recommended an idealised residual stress pattern for use in calculations. This pattern is defined by a certain width of the plate over which the residual stress in tension equals the yield stress. This width of the tension block is thought to be largely independent of the total width of the plate. In addition, when two or more plates meet at a weld, the width ηt of the tension block for each of these is assumed to be the same, and may be calculated by the following equation.

$$\eta t = \frac{CA}{\sigma_y \Sigma t} \qquad (1)$$

Where, Σt is the sum of plate thicknesses meeting at the weld, C is a constant whose value recommended by Dwight and Moxham is 400 tonf/in^2 (6000 N/mm^2), A is the cross-section of the added weld metal and σ_y is the yield strength of the plate. For the cross-section shown in Figure 2, the value of A is arbitrarily taken as 100mm^2.

Knowing the lengths of the tension blocks, the average stress arising in the compression zone is calculated by satisfying the equilibrium of normal forces in the section. The resulting uniform stress-distribution will have an unbalanced moment about the horizontal centroidal axis due to the unsymmetrical shape of the cross-section. To ensure complete static equilibrium, correcting stresses in the compression zone are calculated for a moment that is equal and opposite to the unbalanced moment. Figure 5 shows the resulting residual stress distribution, used in subsequent computations. The maximum compressive residual stress σ_{rm} occurs at the lower tip of the stiffener and is approximately equal to 0.18 σ_r .

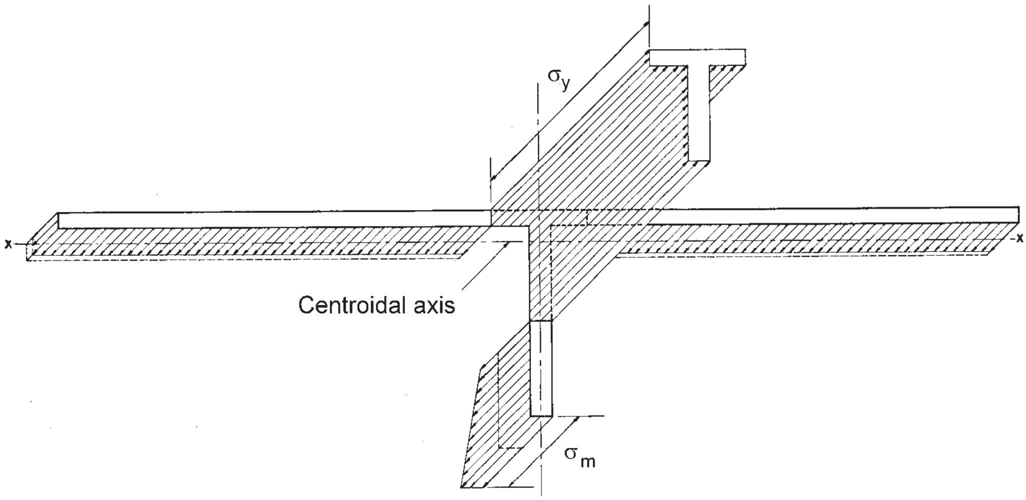

Figure 5 - Calculated residual stress pattern in the stiffened plate

The combined effect of the initial lack of straightness and residual stresses was obtained with the column cross-section having a residual stress pattern shown in Figure 5 for the two modes of bending with the same imperfections as adopted for obtaining Figure 4. The results are plotted in Figure 6. Mode B results are found to be much lower than mode A results for a range even larger than in the case without residual stresses. An interesting feature of these curves is the cusp obtained for both the modes of bending. The cusp occurs due to the rectangular nature of the residual stress-pattern. For a residual stress-pattern with gradual transition from the compressive to the tensile zones, the cusp in both the curves would vanish resulting in a smooth curve. However, sharp transition between tension and compression in residual stress patterns is characteristic of welded sections and cannot be avoided. The stress at which the cusp occurs is about the same for the two modes of bending, but there seems to be no direct relation with the magnitude of maximum compressive residual stress.

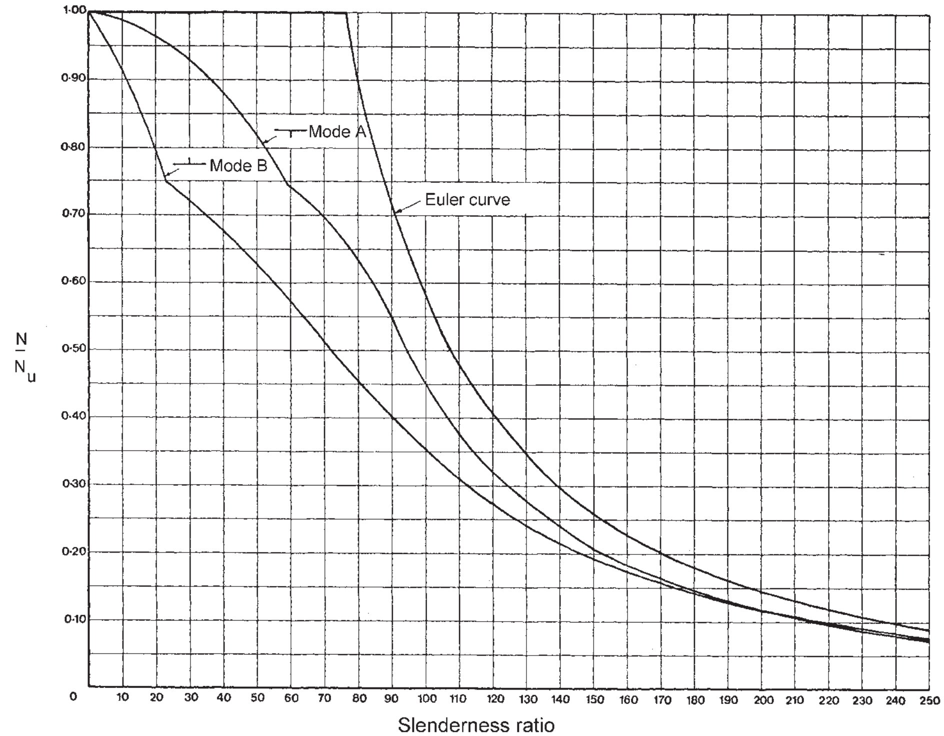

Figure 6 – Failure loads including effects of lack of straightness and residual stresses

LOCAL FLANGE BUCKLING

When the spacing between the stiffeners is large, the strength of the stiffened plate in compression is adversely influenced by the local buckling of the plate situated in between the stiffeners. It becomes of interest then to consider the interaction between the local buckling of the plate and the overall buckling of the stiffener. The concept is that as the plate panel buckles, with geometric imperfections as shown in Figure 3, the load-deflection response of the plate panel can be interpreted as the average stress-strain response of the panel. This follows since stress = force / area and strain = deformation / length.

This approach requires availability of experimental or computational load-deflection characteristics of a range of plate geometries. Admittedly, available experimental data is very limited to a small range of parameters. With the finite element programs widely available, generating such data is not so onerous, especially when presenting results in non-dimensional form. Indeed a compilation of extensive parametric results was published in book form by Williams and Aalami [7]. While the reference gives a large volume of results in terms of stresses, very useful for design, no load-deflection data, that could have been used in the present study, was included.

In order to demonstrate the effectiveness of the approach adopted in this paper, use has been made of experimental results described in Ractliffe [6]. The reference gives experimental load-deflection graphs for plates with width to thickness ratios of 54 and 66, among others. Ractliffe gives curves both for welded as well as for stress-free plates. For the purpose of this study, the curves for welded plates were adapted. For the stiffened plate cross section shown in Figure 2 with a width to thickness ratio of 60, the load deflection graph and hence the average stress-strain characteristic are obtained by interpolation within the range of available experimental results and extrapolation beyond that range (Figure 7). Since the effect of residual stresses is already implicit in the load-end shortening curve, no further residual stresses are considered in the present analysis.

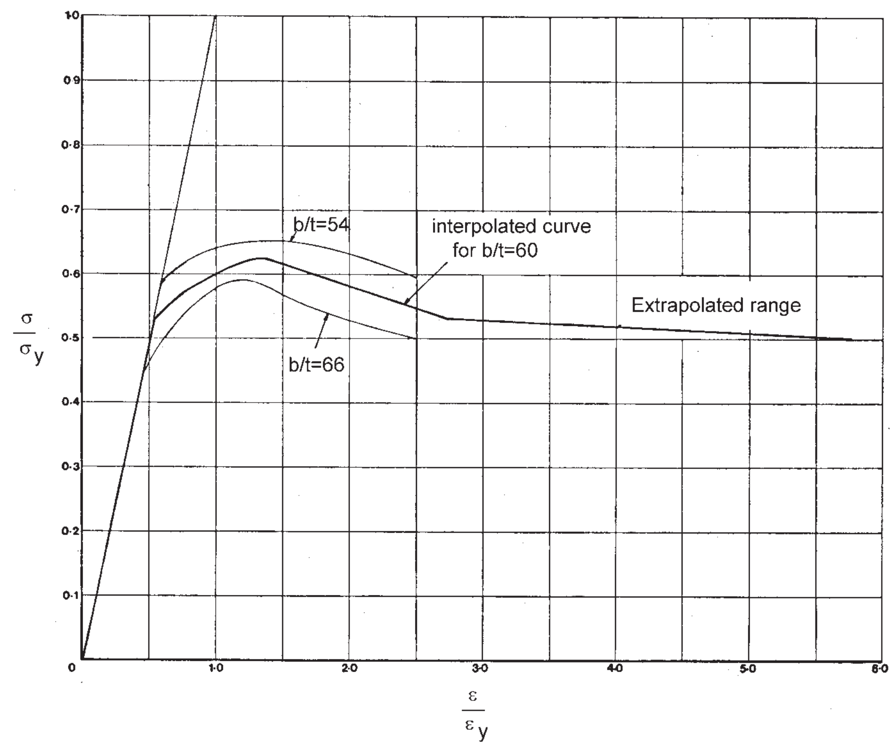

Figure 7 – Average stress-strain curve for a buckled plate of b/t=60

EFFECT OF LOCAL FLANGE BUCKLING ON FAILURE LOADS

Figure 8 shows the variation of failure loads with slenderness when local flange buckling effects are included alongside the overall buckling of the stiffened plate It should be noted that only Mode A bending is relevant, because in Mode B bending, the plate will be in tension and hence will not exhibit local buckling. In calculating the failure loads an initial lack of straightness of $L/400$ was considered. Also plotted are the failure loads obtained without considering local plate buckling (compare Figure 4).

The figure shows the significant effect of local buckling of the plate panel on the failure strength of the stiffened plate. It is notable that at a slenderness ratio above about a value of 110, the effect of local buckling appears to vanish. This is easily explained because at this slenderness or higher, the failure stress of the stiffened plate approaches its Euler stress, and so the local buckling of the plate becomes irrelevant. This critical value would, of course, be different for different plate panel slenderness ratio compared with the slenderness of the stiffened plate.

EFFECTIVE WIDTH CONCEPT TO ACCOUNT FOR LOCAL PLATE BUCKLING

Use of effective width to account for local buckling of plate has been well established in design standards for over half a century. Figure 8 shows the strength of the stiffened plate cross-section when combined with the relevant design curve as given in the British standard for steel bridges, BS 5400: Part 3 [8]. It is estimated that use of Eurocode 3 [9] would give similar results. It may be observed that the correlation between the theoretical results and the results based on the standard used is good. The differences are attributed to different levels of geometric imperfections used.

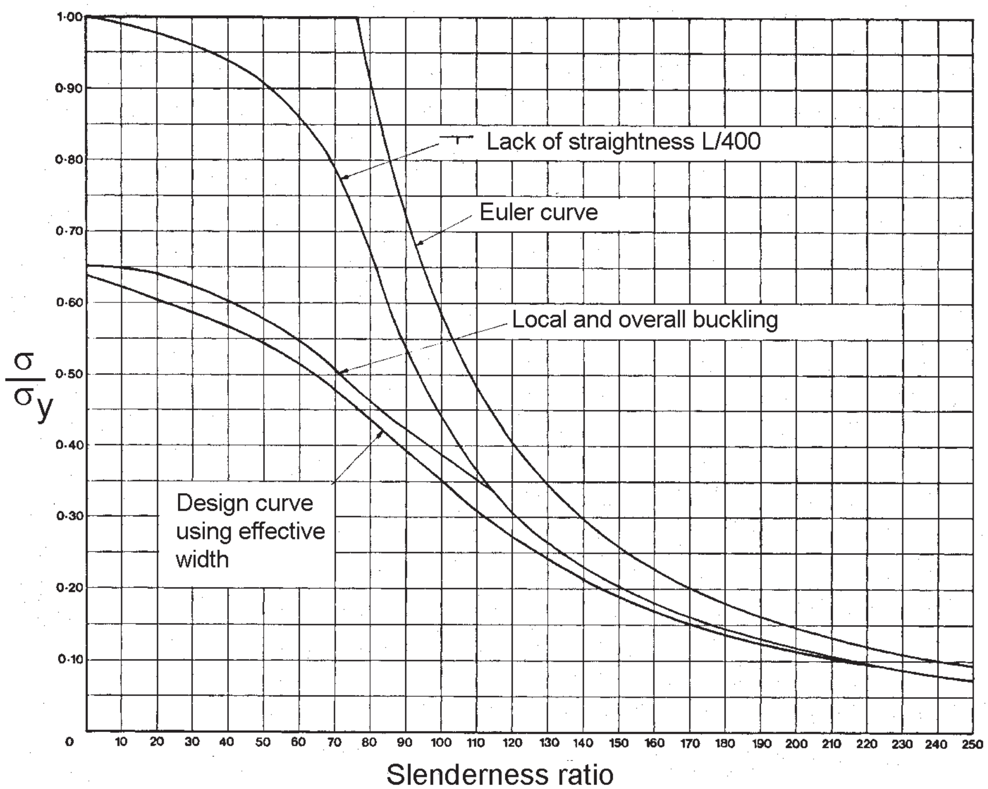

Figure 8 – Failure loads including local and overall buckling

It should be pointed out, that the effective width that may be used to compute the collapse load are not necessarily the same as those required for rigidity, that is, for the calculation of deflections. It is also recognised that the correlation between the loads obtained from effective width calculations and the collapse loads obtained by a consideration of average stress-strain curves to include local plate buckling is itself subject to verification by further research to demonstrate that either approach would agree well with true results for a full range of b/t ratios. This approach does require that stiffener outstands need to be proportioned so as to prevent their own local buckling.

CONCLUSION

It has been shown that the presence of residual stresses and initial lack of straightness may reduce the strength of stiffened plates in compression significantly. In most cases residual stresses as predicted by currently available theory have been shown to have a detrimental effect on the strength of stiffened plates in compression.

An initial lack of straightness, of course, has a detrimental effect on the stiffened plate strength of the same order of magnitude as the residual stresses. Indeed this forms the basis of buckling curves in most standards today. Initial imperfections of the same magnitude have a greater effect with the stiffener in a greater compression than the plate (mode B), when compared with the case of the plate in a greater compression than the stiffener (mode A).

The paper has described a semi-empirical method for considering local plate buckling together with overall buckling of stiffened plates. The method is based on the use of load-end-shortening curves for plate panels in place of material stress-strain curves. Local buckling effects depend, of course, on the width to thickness ratio of the plate. With more adequate load-end-shortening curves, it should be possible to obtain a more accurate estimate of the effect of local plate buckling on the stiffened plate strength. It was also shown that the use of effective widths such as those specified in existing standards leads to a satisfactory correlation with strength curves obtained from the inelastic column failure criterion combined with average load-end shortening curves.

FUTURE WORK

The usefulness of the method can be exploited when deriving buckling curves for structural elements using newer grades of high strength steel with the ultimate strength up to 960 N/mm^2. With such a high strength, structural elements inevitably become more slender. The local buckling of flanges of I or H section members, or in the walls of rectangular or square box sections, poses identical complications as for plate elements in stiffened plates. Using the approach described here, the design curves for structural elements made from the new grades of steel can be developed with speed. Of course, any new buckling curve will need to be validated with at least a limited number of full-scale tests.

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BAR ELEMENT BUCKLING OF COLD FORMED THIN-WALLED STEEL SECTIONS IN CONNECTION WITH RUSSIAN FEDERATION STANDARD

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INTRODUCTION

Recently wide expansion of production of thin-walled structures on world market can be seen. Thanks to rational use of material, transport simplicity and possibility to erect without crane equipment, especially for low-rise buildings, application of this kind of structures makes it possible to save the material and manpower resources greatly. However, in Russian Federation the essential barrier for an extension of this market segment is a complete absence of any standards for manufacture and construction of these structures. Meanwhile, foreign standards like EN 1993-1-3:2006[1] in Europe and SEI/ASCE 8-02[2] in the USA cannot be used because of great difference in other relevant standards, established methods for designing and accuracy of production and assembly on the building site. In this way, there is an important issue to develop own method for the design of cold-formed steel structures considering proved methods and experience in design abroad.

METHODS

During calculation of thin-walled cold-formed elements a number of specific features should be taken into account. These are:

1. local and distortional buckling of the sections;

2. necessity of calculation by global tridimensional design model in view of existence in most cases of biaxial load (Fiig. 1) and factors described above

3. presence of production imperfections and probable damages that have great influence on local buckling of the section;

4. presence of residual stresses and strengthening of the material in bending places of the section during its production;

5. Special features in the calculation of joints and there influence in distributing of loads to the cold formed steel bar element.

The solution of this problem in accordance with Eurocode-3 is a time-consuming iterative process that is connected with increasing non-effective zones, displacement of main section axis and redistribution of the stresses in the section and in the whole element (Fig. 2). Numerical evaluation also is a complex process of modeling and calculation. Meanwhile the results remain limited (only for selected model) and do not permit analytic evaluation.

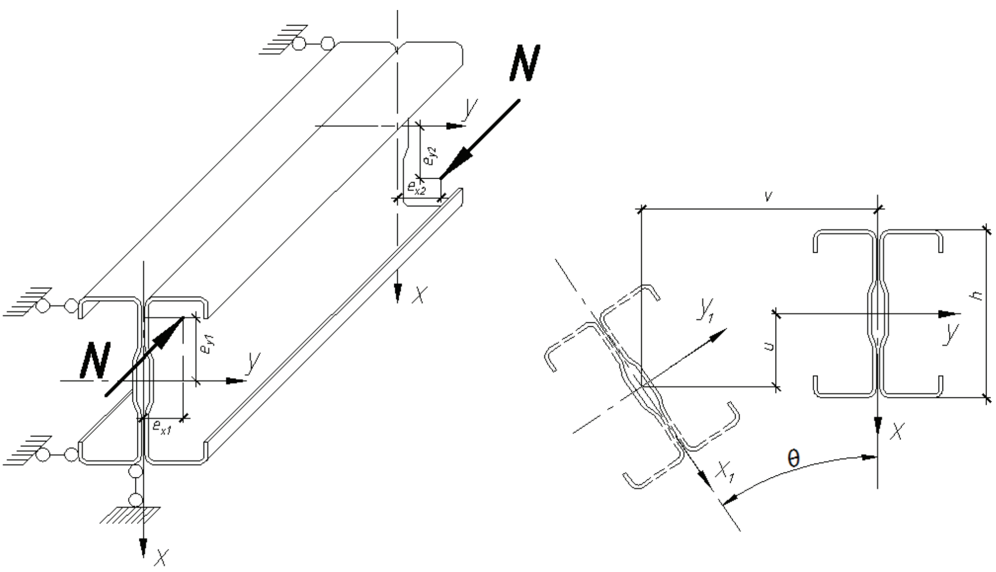

Figure 1. Global tridimensional bar element model

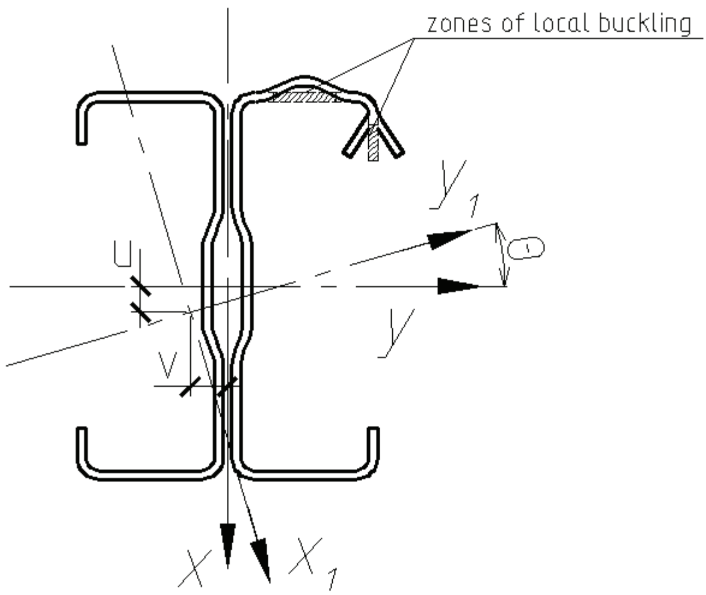

Figure 2. The displacement of main section axis because of local buckling

In this connection the solution of bar global buckling with local and distortional buckling influences has been suggested to perform with deformitive theory of elastic bar by Vlasov, generalized by Broude and Beilin. The system of the differential equations after its preliminary integration of first and second equation along with taking into account symmetry of the section, takes on form:

$$\begin{cases} \mathrm{EJ}_x v'' - N^0 v + M_y^0 \theta - M_z^0 u' = -M_x^0, \\ \mathrm{EJ}_y \mathrm{u}'' - N^0 u - M_x^0 \theta + M_z^0 v' = -M_y^0, \\ EJ_w \theta^{IV} - GJ_k \theta'' + M_y^0 v'' - M_x^0 u'' + i^2 N^0 \theta'' = 0. \end{cases} \tag{1}$$

For the calculation of the system of equations (1) the analytical-numerical method of Belyy is used, where the solution is expressed as the linear combination of the particular solutions:

$$\begin{cases} u = u_n + u_b + u_{pl} + u_0, \\ v = v_n + v_b + v_{pl} + u_0, \\ \theta = \theta_n + \theta_b + \theta_{pl} + \theta_0 \end{cases} \tag{2}$$

where u_n, v_n, θ_n – displacements from the non-deformitive calculation;

$u_b = U_b \psi_b(z),\ v_b = V_b \psi_b(z),\ \theta_b = \mathrm{T}_b \psi_b(z)$ – functions from the bifurcation solution, which are found with an accuracy of the constant U_b, V_b, T_b, and which have displacement dimensions.

$$\psi_b(z) = \sin(\pi \cdot \frac{z}{L})$$

u_0, v_0, θ_0 – initial displacements of the bar;

$u_{pl}, v_{pl}, \theta_{pl}$ – displacements and torsion angle, which take in account local, distortional buckling and plastic deformations.

For its definition a special algorithm "Section" is used, which by a non-linear process allows the determination of the deflected mode, curvatures and torsion angles. The solution (2) is substituted in the system of equations (1) and then the energy method of Bubnov-Galerkin is used. This results in a system of three algebraic equations relating to the unknown variables U_b, V_b, T_b. Thus, in the solution (2) all parts become known and it can be used to determine "non-linear efforts" and estimate global buckling.

It should be noted that the influence of inelastic stage, taking into account local material strength and residual stresses has been developed by Belii[3]. In [4] this method has been extended to cold-formed steel structures with local buckling for single sections (such as Csection) by Astachov. Developing this method for complex compound sections algorithms have been made up and a program that takes into account the influence of local and distortional buckling on global buckling of the bar is now available.

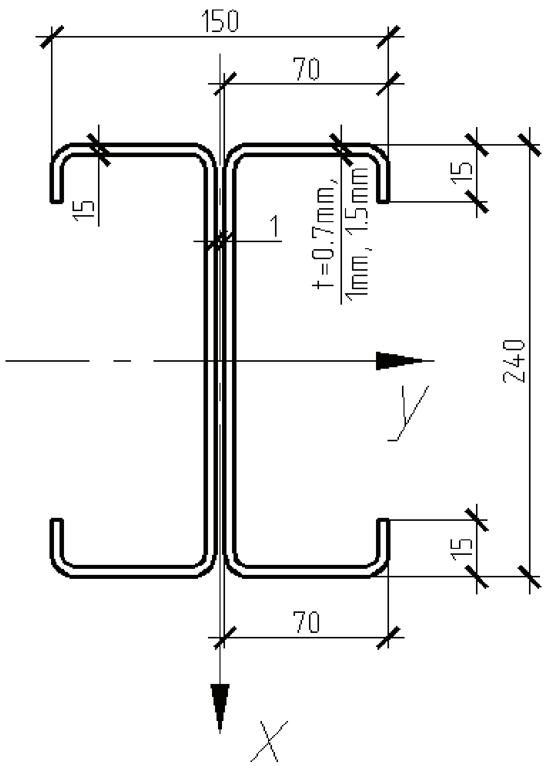

Figure 3. Considered bar section

As an example of the solution with this method has been considered a bar of 3m length (slenderness about $\lambda_x = 100$, $\lambda_y = 30$) with a section described in Fig. 3. This section has been loaded with an equal biaxial load on both sides as it can be seen in Fig. 1. Three variants of thickness were considered namely, 0.7mm, 1mm, 1.5mm.

In the graphs of Figures 4, 5, and 6 are presented tridimensional displacements u, v, θ (Fig. 1) of middle cross section of the bar. The bar with local buckling influence was described with a continuous line and the bar without local buckling influence with a dotted line.

For sufficiently thin elements (Fig. 4) with 0.7mm thickness and normalized part section slenderness for flanges: $\lambda_f = 1.44$, for webs: $\lambda_w = 5.8$, that are determined by (3), exhaustion of load bearing capacity happens mainly by local buckling, that begins at 37% of the ultimate load. Meanwhile, comparison of graphs indicates that influence of local buckling reduces global buckling by 19%.

$$\bar{\lambda} = \frac{\bar{b}/t}{28.4 \cdot \sqrt{\frac{235}{f}} \cdot \sqrt{K_\sigma}} \tag{3}$$

If we consider thicker bar element of 1mm thickness (Fig. 5), we find that the local buckling reduction begins at 52% of the ultimate load capacity and reduces it by 19%..

In comparison, thick elements with t=1.5mm (Fig. 6) local buckling reduces the ultimate load by 5%.

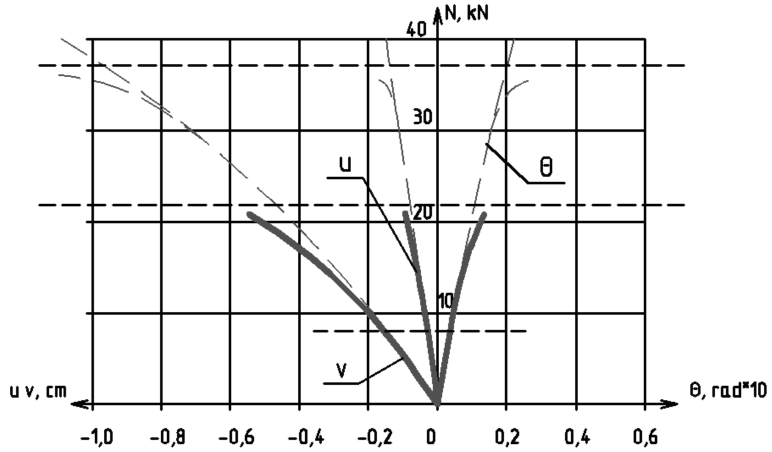

Figure 4. Graphs of tridimensional displacements in middle section of the bar with 0.7mm thickness.

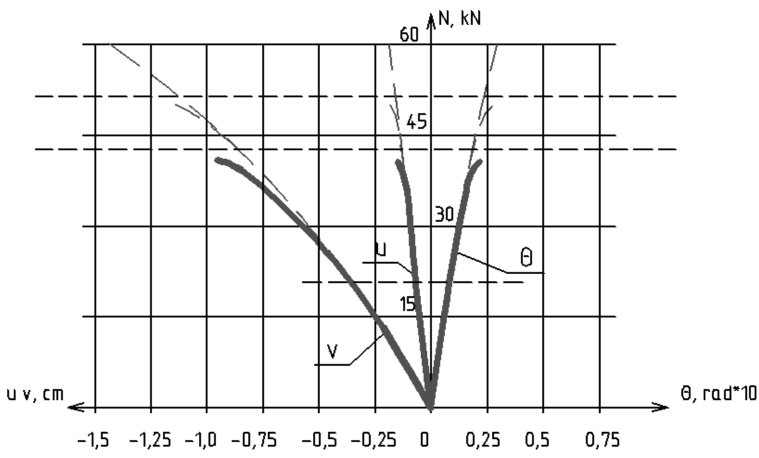

Figure 5. Graphs of tridimensional displacements in middle section of the bar with 1mm thickness.

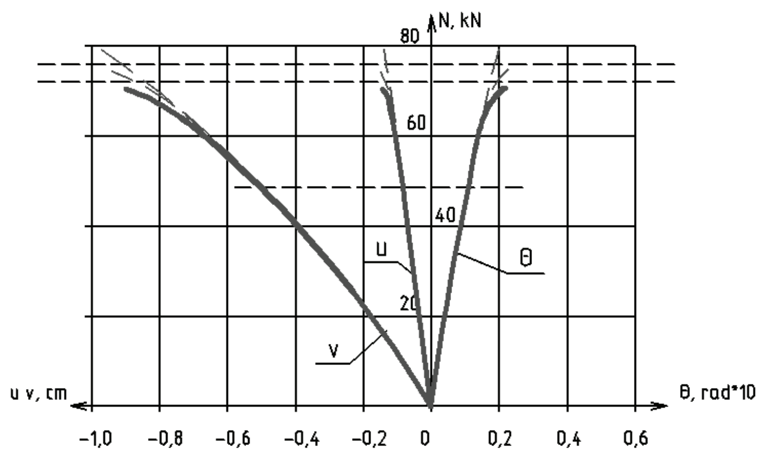

Figure 6. Graphs of tridimensional displacements in middle section of the bar with 1.5mm thickness.

CONCLUSION

1. Application of analytic-numerical method, described in [3] and extended with [1] on cold formed steel structures, makes it possible to get the solution some degrees quicker.

2. Suggested method allows generalizing particular solutions of global, distortional and local buckling in the unified algorithm that takes into account their cross-effect.

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MULTI-CRITERIA OPTIMIZATION OF BUILDINGS

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ABSTRACT

Decision making during pre-design of buildings should be based on transparent data especially when dealing with costs, environmental issues and customer preferences. This transparency requirement will be full-filled using mathematical optimization techniques, because then we should formulate criteria functions and corresponding constraints in the design space with exact mathematical forms, which can be evaluated and fine tuned by the decision maker. In this paper two cases and their multi-criteria optimization problem formulations and solutions are described. The first case deals with a 120 m² single family house and the second 10000 m² supermarket to be built in the Helsinki region, Finland. The criteria functions are such as capital costs, energy consumption, environmental impact and customer preferences. The constraints are coming from the codes of practice. The optimization problems are solved using genetic algorithms and results are shown applying multi-criteria decision making (MCDM) methods to study the Pareto-optimal solutions obtained.

INTRODUCTION

Construction management decisions typically involve several conflicting aspects that need to be considered. These decision-making situations can be formulated as multi-criteria optimization problems, where the different aspects of a building project constitute the conflicting criteria that are optimized simultaneously. It is widely recognized that most of the total cost and the performance of the building is determined by the decisions made in the conceptual design phase. Therefore, applying multi-criteria optimization in this early phase can lead to considerable savings in the building project [Miles, 2005].

Optimization by itself has been applied to many kinds of problems structural optimization being an important and broadly studied area [Cohn, 1994]. Also, ideas of multi-criteria optimization have been utilized. The classic paper of Koski [Koski, 1994] shows the conflict between weight and displacement of a given plate structure. Grierson and Khajehpour [Grierson and Khajehpour, 1999, Grierson and Khajehpour, 2002] presented multi-criteria optimization of a high-rise office building showing a three dimensional Pareto space of capital cost, revenue income and life cycle cost. Wang [Wang et al., 2005] introduces lifecycle environmental impact as criterion against life cycle cost.

In this paper two example building design problems of multi-criteria optimization are presented. First, a fairly simple problem of single family house with five criteria, then a problem of 10000 m² hardware store with four criteria.

PROBLEM FORMULATION

The solution of an optimization problem begins with problem formulation which includes choosing the design variables, constraints and criteria functions. After this, a suitable algorithm should be chosen to solve the problem. Once the solution is achieved, the quality of the solution should be somehow evaluated.

The problem formulation is shown for each problem in its own section below.

SOLUTION USING GENETIC ALGORITHM

The method chosen is *multi-objective genetic algorithm* (GA) of *Global Optimization* toolbox of Matlab [Mathworks, 2010]. The optimization problem in both examples becomes discrete and therefore methods for continuous problems cannot be used. There are other approaches - like the constraint method – for solving multi-criteria problems but population based methods like GA give a set of *non-dominated* solutions already with one optimization run whereas with other approaches one run results in only **one non-dominated solution**.

Genetic algorithm mimics the evolution in nature. The vector of design variables is set up as a string of chromosomes. Usually binary coding is used. First, the initial population is randomly created, then each individual is evaluated with respect of fitness function (usually the criterion function). The fittest are chosen to produce offspring to the next generation. This procedure is repeated until the number of generations specified by the user is completed or other stopping criterion becomes applicable. Even though there is no rigorous mathematics behind this or guarantee of method leading to optimal solutions, the results tend to be very good. Different versions of genetic algorithms have many variations of mating, mutation, cross over et cetera. For reader interested in details references [Holland, 1975] and [Deb, 2002] might be worth reading.

Due to stochastic nature of the method used, several runs have to be completed even with multi-objective GA. This will give a set of solutions. Typically, hundreds or thousands of solutions are required. First, the sets given by different runs are combined. Then the dominated solutions are removed. Then a set of non-dominated - or Pareto optimal - solutions is achieved. For the domination of points in two criteria space, see Figure 1. In a minimization problem, in the combined set of two sets marked with crosses and circles, the points forming the front closer to origin rounded with dashed line are not dominated by any other point and thus are called non-dominated.

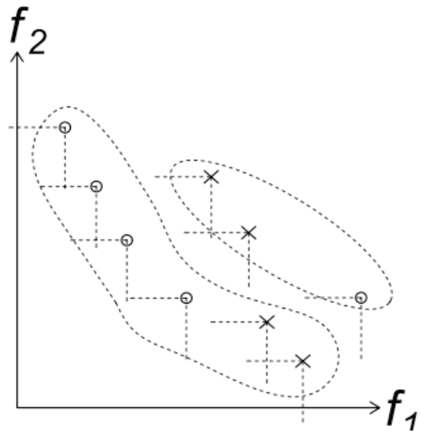

Figure 1: Domination of the points

MCDM methods

From among the set of non-dominated solutions usually only a few can be investigated further and if a building project is considered, only one of the proposed mathematically equal solutions will be built. Searching the multidimensional criteria space can be done with graphical representation, if the number of the criteria is two or three. If there are more criteria, this becomes impossible. To overcome this problem, several multi-criteria decision making methods to support the designer have been introduced. The authors have investigated six of them in [Mela et al., 2012].

SINGLE FAMILY HOUSE

A person building a single family house is interested in many qualities when planning a project: construction cost, maintenance cost, aesthetics, environmental impact, structural safety, fire safety, to name only a few. It is clear that family house is a useful object for testing multi-objective optimization. In this paper, the following five criteria were considered:

- Construction work cost
- Construction material cost
- Energy consumption
- Customer preference
- Environmental impact

The building considered (seen in Figure 2) is a fairly simple design with a rectangular plan. The design variables include the height and length of the house, roof and wall insulation thickness, the number and size of windows and the wall type. The first two are continuous and other discrete variables.

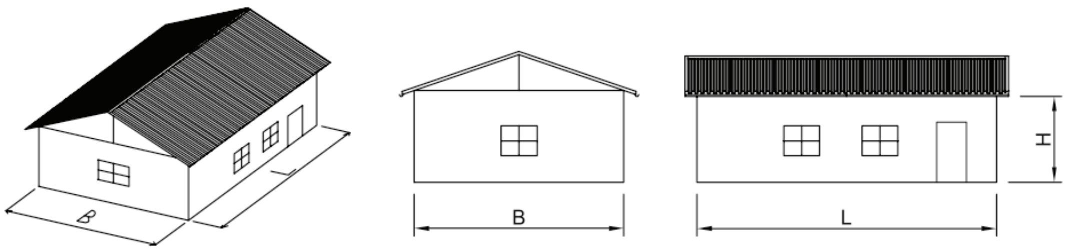

Figure 2: The single family house and some design variables

Both the construction work and material cost were calculated using the cost data from [Mittaviiva Oy, 2010]. Also, the structure types were taken from the same reference. The material and construction costs were taken as separate criteria because in many cases the client is willing to do at least some of the work by themselves and then they might be only interested in the cost of materials.

Energy consumption can be calculated in many ways. The approach adopted now is based on [EN ISO 13790, 2008] and Finnish regulations [D3, 2012, D5, 2012] but only taking some components of the energy consumption into consideration. Now, the annual energy consumption of a building is expressed as:

$$f_3(\mathbf{x}) = E(\mathbf{x}) = \sum_{i=1}^{12} [(Q_{H,tr} + Q_{H,al}] + Q_w \quad (1)$$

where

- $Q_{H,tr}$ is the monthly heat transfer by transmission of the envelope of the building;
- $Q_{H,al}$ is the monthly heat transfer by air leakage through the envelope of the building;
- Q_w is the monthly energy needed for warm water heating;
- i means that calculation is based on monthly data;

In Equation 1 it can be seen that some major components like ventilation and (in warm atmosphere) cooling are now omitted. So the number thus achieved as criterion value does not represent the actual energy consumption but it can be used to compare different solutions.

Whereas typical engineering quantities are well-defined, the customer preference is a matter of taste. It depends on the aesthetic taste of each customer, and such a function has to be constructed separately for each instance. Also, sometimes it is impossible to express the customer preference as a mathematical function. In this case it was decided that the criterion favours large living area, high ceiling and large windows. This leads to the following expression when maximization problem is changed to minimization problem:

$$f_4(\mathbf{x}) = -\left(w_A A_l + w_H H + w_w A_w\right) \qquad (2)$$

Where w_i are weights for different measures and A_l is the living area of the building. Weights chosen now are $w_A = 1\ \mathrm{m}^{-2}$, $w_H = 5\ \mathrm{m}^{-1}$, and $w_w = 5\ \mathrm{m}^{-2}$. Thus, the function f_4 is dimensionless. The comfort function is in parentheses of Equation 2 and it should be maximized, so its negative value should be minimized, as all other criteria in the final problem.

The environmental performance of buildings is measured by different indicators representing different environmental aspects. These indicators can be categorized to those describing environmental impacts, resource use, waste categories and output flows. In this work, the global warming potential (GWP) is chosen as a measure for environmental impact of the building.

A general equation for computing GWP of a building is

$$f_5(\mathbf{x}) = GWP(\mathbf{x}) = \sum_{j=1}^{N} a_{j,i} GWP_j \qquad (3)$$

Where $a_{j,I}$ is the gross amount of product or service j used in the building and GWP_j is the global warming potential of product or service j ([EN 15978, 2012, pp. 44]).

The usual constraint in a building includes some strength checks for load bearing structures. In this case it was observed that the spans are fairly small and the strength of the wall quite significant even with the lowest insulation thickness, so the resistance checks were left out. Still constraint for maximal allowed heat transfer through the envelope was constrained by equation

$$\sum U_i A_i \leq \sum U_{ref} A_{ref} \qquad (4)$$

Where U_{ref} and A_{ref} are reference values defined by [D3, 2012]. Secondly, Finnish building regulations [G1, 2005] say that the area of windows A_w is required to be over 10 % of the living area A_l

$$A_w \geq 0.1 A_l \qquad (5)$$

More details and results of single family house optimization can be seen in [Heinisuo *et al.*, 2012]. Also one version of this problem was considered as an example problem in [Mela et al., 2012].

HARDWARE STORE

The second case considered is a one-story building without windows or other openings. This kind of building could be used as a hardware store or maybe some other commercial or industrial applications. The design of a one-story building may seem a simple task but as the spans are long the amount of different material and structural combinations becomes very large. In this problem formulation four criteria were considered:

- Capital cost
- Energy consumption
- Customer preference
- Environmental impact

Capital cost for different structures were approximated using general Finnish cost guides [Mittaviiva Oy, 2010] and [Haahtela and Kiiras, 2010] based on quantities.

Energy consumption could be calculated as in the family house problem but here a more simple approach was adopted. In the case of hardware store, the linear thermal bridges in corners are almost constant and so is the temperature difference of outside and inside air. So the only thing that was considered is the sum of thermal transmittance of the parts:

$$f_2(\mathbf{x}) = \sum_i A_i U_i \tag{6}$$

which is dependent of the design variables (wall and roof structures with different insulation thicknesses).

The expression for customer preference in the case of a hardware store was not also found in the literature and therefore it was created based on interviews with real estate owners and investors. The main idea is that the more free area without columns the more freedom there is for different space divisions and the space can be used for many purposes. This is thought to make the space easier to rent or to sell with a higher price. The expression ended up as follows:

$$f_3(\mathbf{x}) = C_{pr} = \frac{12000\sqrt{n_{ic}}}{A_{room} l_{fr,avg}} \tag{7}$$

where

- n_{ic} is the number of interior columns
- A_{room} is the floor area which is free of columns (see also Figure 3)
- $L_{fr,avg}$ is the average free distance from arbitrary location from wall (see also Figure 4)

Environmental impact was calculated in a manner similar to the single house problem.

The optimization of the hardware store was divided into two optimization problems of two different structural systems. The difference between systems is the direction of beams which can be seen in Figure 5. The design variables in both systems include spans, the number of bays, dimensions of member, materials of members, wall types, roof types etc. The constraints are formed of applicable Eurocode standards for different materials and structures [EN 1993-1-1, 2005, EN 1993-1-8, 2005, EN 1992-1-1, 2005, EN 1995-1-1, 2004] applying loads defined by standards [EN 1991-1-1, 2002, EN 1991-1-3, 2004].

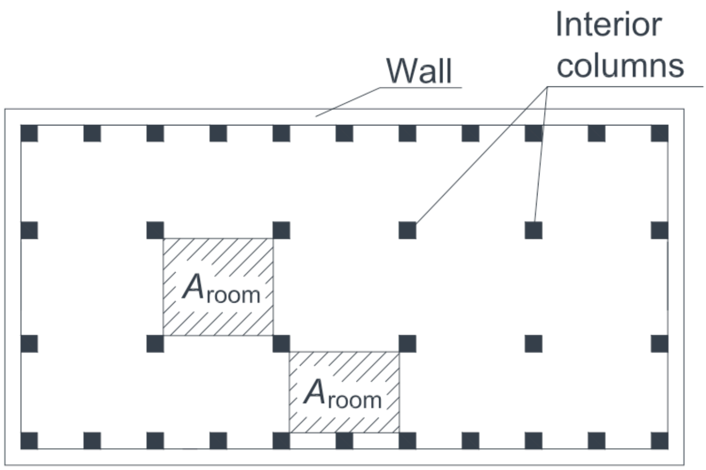

Figure 3: Definition of area A_{room}

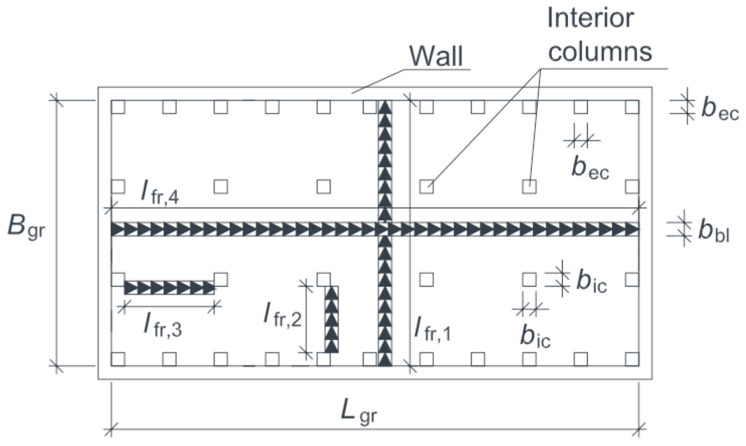

Figure 4: An illustration of the measure $l_{fr;avg}$

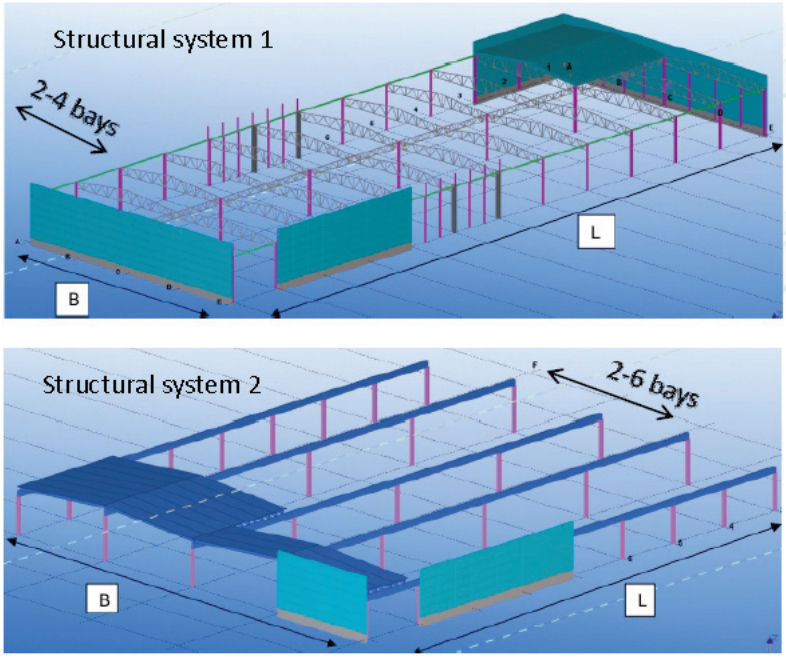

Figure 5: Structural systems 1 and 2.

The original intention was to use Tekla Structures [Tekla, 2012] and its plugins for material lists and cost evaluation. For that purpose a parametrical link from Matlab [Mathworks, 2010] was created but for optimization purposes it turned out to be too slow. Still, a “store-macro” was also obtained as an outcome of this case study.

The solution procedure yields up to one thousand non-dominated solutions. At a four dimensional criteria space handling the results need expertise. Moreover, for every individual solution, a vast amount of additional result data is available. Some of the results can be seen in Tables 1 and 2. First, in Table 1 the effect of the area on optimal solutions was studied. The best solutions for each objective function in structural system 2 can be seen in Table 2. More detailed results and analysis will be published in the near future.

CONCLUSION

The main conclusion is that multi-criteria optimization can be used as a tool at early design phase of a building. The results of multi-criteria optimization may seem hard to interpret, but they contain a whole lot of transparent information about the design problem. Also, good solutions beyond intuition and experience of a designer can be found with the optimization tools.

Table 1: Found optimum solutions among both structural systems and their ratios to building area at different areas.

Area	Cost [M€]	Cost per area [€/m^2]	ΣUA [W/K]	ΣUA [W/(Km^2)]	GWP [tn CO_2 eqv.]	GWP per area [kgCO_2eqv/m^2]
6000	0.7386	123	660	0.11	165	27.50
8000	0.929	116	841	0.11	200	25.00
10000	1.0007	100	984	0.10	216	21.60
12000	1.056	88	1016	0.08	236	19.67

Table 2: Best solutions for each objective function in structural system 2.

Store	Cost [€]	ΣUA	C_{pr}	GWP [tn]
min(€)	1.0068	2867	24.14	467
min(ΣUA)	2.203	984	16.40	1796
min(C_{pr})	2.1131	1923	0.86	1221
min(GWP)	1.301	3251	35.35	216

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SUSTAINABLE REFURBISHMENT OF A MULTI-STOREY RESIDENTIAL HOUSE - STEEL SOLUTIONS

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1. Introduction

The objective is to study the impacts of alternative renovation scenarios on building stock in terms of energy use and greenhouse gases. The focus of the study is on residential buildings. In addition to the assessment of the renovation concepts on building stock also the energy sources, the significance of building materials, especially the steel products in renovation projects, different renovations concepts and the economic impacts of building renovation are discussed.

New methods, steel products and concepts for sustainable renovation of buildings are presented. New technologies are often resisted because those require process changes and unknown risks and not-foreseen costs are suspected. The premise of the presentation is that the Sustainable Renovation is not hindered because of the lack of information, technologies and assessment methods, but because it is difficult to adopt new processes and working methods in order to apply new technologies, especially advanced steel solutions. The objectives are to understand barriers and impacts, develop new working processes, develop new business models and develop effective steering mechanisms for the Sustainable Building.

A case study is presented. The target of the case study is to assess the renovation of a group of a multi-storey residential house from 1970s in Finland.

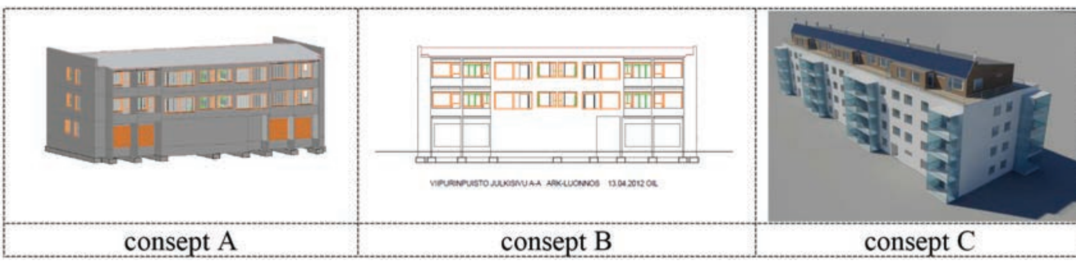


Figure 1 Consepts A, B, C

Three alternative consepts are assessed:

- consept A: no particular sustainable measures
- consept B: large scale sustainable improvements
- consept C: large scale improvements and extra measures with steel structures

The assessment takes place with help of sustainability indicators. At the same time the target is also to evaluate usability and usefulness of sustainability indicators in target setting and monitoring of the project and the steel products. The premise is that a logical outline of sustainable building aspects and indicators are needed in order to continuously improve and promote sustainable building. An outline and measurable indicators are needed for setting targets and for the follow-up of the results.

Background

The general principles on life cycle assessment (LCA) of products and services have been agreed upon and introduced with help of standardisation. In addition, there are international standards available on the formats, contents and processes of environmental assessment and declarations of products. The main deliverables include European Reference Life Cycle Database (ELCD) with European scope inventory data sets /6/ and Internationally coordinated and harmonized ILCD Handbook of technical guidance documents for LCA. ISO and CEN are currently developing building and construction related sustainability standards, workprogramme CEN/TC350 which cover all levels and all sustainability aspects . Nationally recognised methods have been developed for the environmental declaration of building products in addition to international and European standards e.g. RT Ympäristöselosteet - RT Environmental Declarations.
Sustainable development of buildings and other construction works, the draft for ISO 21929 (2010) defines that sustainability impacts can be categorised as follows:

- Environmental: climate change, deterioration of eco-system, use/depletion of resources
- Economical: economic value, productivity
- Social: health, satisfaction, equity, cultural value.

The sustainability of the developed concepts and technological solutions will be assessed considering building performance, service life, environmental impacts and life-cycle costs. Alternative refurbishment concepts will be environmentally assessed according to the procedures.

The basic environmental data will be collected and presented in such a way that the focus will be on the following environmental aspects:

- use of renewable energy
- use of non-renewable energy
- use of renewable natural raw materials
- use of non-renewable natural raw materials
- green house gas emissions
- wastes (problem wastes, other wastes)

Embodied energy will be dealt with as a separate parameter. Carbon footprint assessed on life cycle bases in terms of green house gases will be the main environmental assessment criteria in accordance with the project objectives. In the countries where most energy produced in co-generation processes, for example in Finland (2008) 21 % from power and 75 % from heat produced in co-generation process (CHP), the method used for allocation is extremely significant for the result.

Environmental impacts of refurbishment concepts

As an example the environmental impacts of a refurbishment concepts of an externall wall are assesed. The calculation is shown for a concrete element wall that was typical in multi -storey residential buildings 1960s and 1970s.

Consepts: original exterior wall + new insulation U= 0.17 ... 0.00 + façade board/ Ruukki photovoltaic solar panel, energy prize rise 4% / 10%

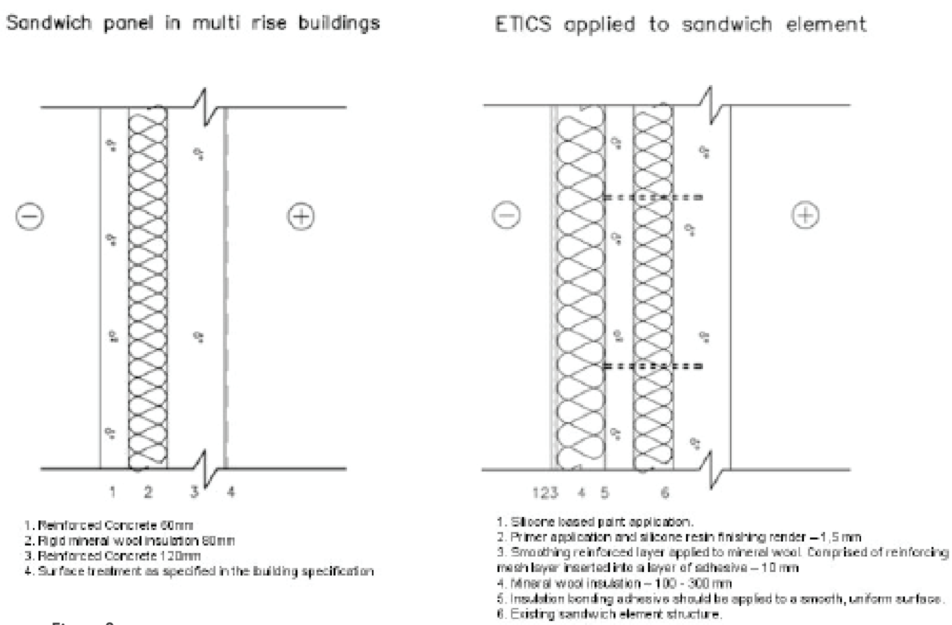


Figure 2

A three layer concrete wall structure has been very common in all European countries since 1960. The refurbishment method is well developed and the technology is widespread. The thick mineral wool layers examined in this concept are relevant in Northern European countries. In order to improve the heat insulation of the wall and the energy performance of the building, the refurbishment is done in such a way that a new thicker insulation and a new façade panel with thermo-steel purlins are added. Here it is assumed that the intended new U-value is 0.17 W/m^2 K which requires an insulation thickness of 100 mm (mineral wool, - value 0.035 W/m K). The exterior concrete wall is similar to the original wall.
The environmental impacts because of this refurbishment concepts come from

- manufacture (from cradle to gate) and transportation of new replacing products (exterior concrete wall + heat insulation)
- construction of the new structure considering the material losses.

The example is calculated with Susref Tool using the Finnish values, time period 50 years. The outcome for the refurbishment is as follows in tables:

Table 9

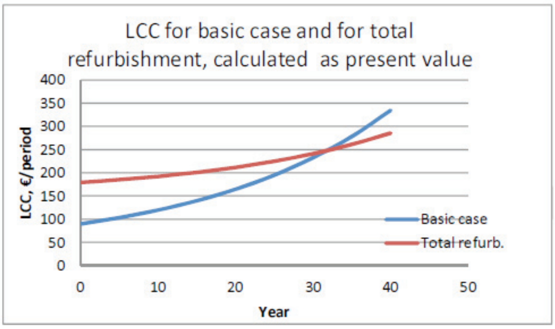
LCA results

	Embodied fossil enery	Embodied ren. Energy	Raw material	CF material	CF heating	CF heating	CF total
	$MJ/m^2/period$	$MJ/m^2/period$	$kg/m^2/period$	kg/m^2	$kg/m^2/a$	$kg/m^2/period$	$kg/m^2/period$
Basic	5 401	1 913	254	minor effect	11,0	552	552
Total	2 814	922	270	25	4,8	239	265
Savings to basic	2 587	991	-16	no savings	6,2	312	287

Table 10

LCC results calculation for 50 year period

Renovation type	U-value	Energy losses	Construct. costs	Heating cost	Heating cost present value	LCC
	W/m^2K	kWh/a	€/m²	€/m²/a	€/m²/period	€/m²/period
Basic	0,40	45	90	2,5	398	488
Total	0,17	20	179	1,1	173	352
Savings to basic	0,23	25	-89	1,4	225	136



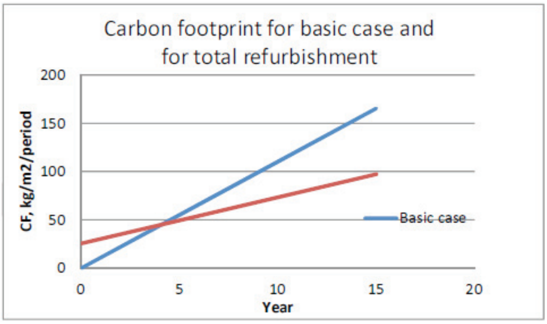


Figure 3 LCC and LCA values

Table 11

LCC, (€)

year	0	5	10	15	20	25	30	35	40	50
Basic case	90	103	120	140	165	195	233	279	335	488
Total refurb.	179	185	192	201	212	225	241	261	285	352

LCA, Carbon footprint (kg CO2e)

year	0	5	10	15	20	25	30	35	40	50
Basic case	0	55	110	166	221	276	331	386	441	552
Total refurb.	25	49	73	97	121	145	169	193	217	265

The following figures describe the results with different parameters.

Heat energy type affects the LCC savings as follows:

Table 12

Heating energy type	Savings to basic solution (e/m^2/period)
District heating (FI)	136
Electricity (FI)	280
Oil	280
Gas	116
Coal	198
Electricity(27)	327

This heat energy savings table tells too about relative prize of different energy sources used in the calculation.

The results show that it takes even more than 25 years to make the wall refurbishment profitable but as for the CF- values the time period is about the same.The results are once again very much dependent on the assumptions in the calculations and it is worth testing with alternative scenarios.

Table 13 Photovoltaic Solar Panel

	per 1 m2	per 100 m2	per 400 m2
Power	0,12 kWp	12 kWp	48 kWp
Modules	1,3 pcs	132 pcs	528 pcs
Grid feedin/a	76 kWh	8 070 kWh	32 280 kWh
CO2 emissions avoided/a	67 kg	7 130 kg	28 510 kg

If the rise of the energy price would be 10% the payback-time would be only about 5 years.

In a scenario where energy prize rise up to 10% it might be profitable to decrease the U-value and take in use new products like Rautaruukki's Photovoltaic Solar Panel. The solar panel can produce energy and change the original LCA and LCC assessments totally.

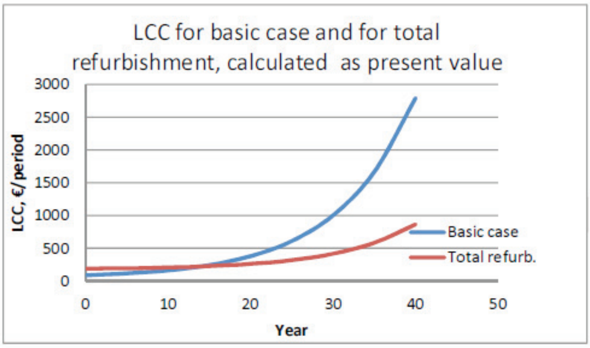
Simulating this case the results are the following:

Table 14

LCC results	calculation for	50	year period			
Renovation type	U-value	Energy losses	Construct. costs	Heating cost	Heating cost present value	LCC
	W/m²K	kWh/a	€/m²	€/m²/a	€/m²/period	€/m²/period
Basic	0,40	45	90	4,5	7819	7 909
Total	0,10	11	190	1,1	1959	2 149
Savings to basic	0,30	34	-100	3,4	5861	5 761

Table 15

LCA results							
	Embodied fossil enery	Embodied ren. Energy	Raw material	CF material	CF heating	CF heating	CF total
	MJ/m²/period	MJ/m²/period	kg/m²/period	kg/m²	kg/m²/a	kg/m²/period	kg/m²/period
Basic	13 737	1 925	584	minor effect	26,5	1325	1 325
Total	4 095	594	331	40	6,6	332	372
Savings to basic	9 643	1 331	253	no savings	19,9	993	954



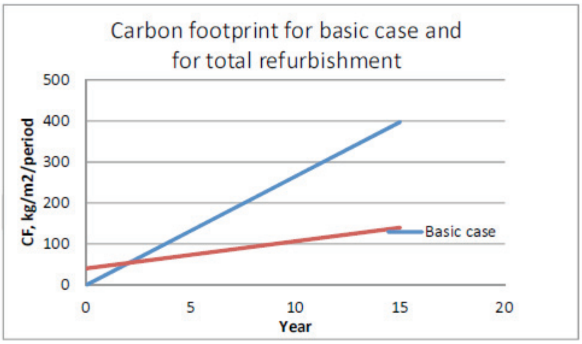


Figure 4 LCC and LCA figures with U-value = 0.10 W/m²K

In this scenario the payback- time is about 10 year on condition that the estimated invest cost is right. Energy losses are 11 kWh/a and CF material + heating/a = 46,6 kg/m² much smaller than the solar panels capacity 76 kWh/a and CF emissions avoided/a = 67 kg/m².

The external wall refurbishment structure can change the building from a consuming unit to a productive one with renewable energy!

Viipurinpuisto Apartment House Case

Renovation of Viipurinpuisto, two stories + basement, floor area 13 m x 26 m = 338 m2, total net floor area 900 m2, external wall area 785 m2, window area 64 m2, volume 3000 m3, includs necessary actions as follows

consept A	consept B	consept C

Figure 5

Consept A
- pipeline works

Consept B
- pipeline works
- renewable heating energy: solar, heat pumps, wood, hydro-power
- renewing of windows (U –value 1,8 to 1,0 W/m2K))
- external additional insulation
- ventilation, heat exchange efficiency= 0.8, air change rate= 0.5

Consept C
- pipeline works
- renewable heating energy: solar, heat pumps, wood, hydro-power
- renewing of windows (U –value 1,8 to 1,0 W/m2K))
- external additional insulation
- ventilation, heat exchange efficiency= 0.8, air change rate= 0.5
- lifts
- additional storey with steel structures

The consepts are evaluated as well as from the LCA and the LCC point of view. The LCC estimation is made according the Apartment house example /22/ presented before and LCA assesment is made with the help of Enslic simlif tool /23/. The parameters have been chosen to meet Hämeenlinna, Finland circumtances.

Viipurinpuisto LCA- assessmnet

The Consept C evaluation is shown as follows:
Table 23 Consept C LCA of a multi-storey house
- pipeline works
- renewable heating energy: solar, heat pumps, hydro-power
- renewing of windows (U –value 1,8 to 1,0 W/m2K))
- external additional insulation
- heat exchange efficiency= 0.8, air change rate= 0.5
- lifts+ additional storey with steel structures, heated floor area 900+300 m^2

Summary of yearly impact impact

Anticipated building life time	50

	kWh/tot are:	%	kg equiv CO2/m2	%
User Electricity	36 000	28 %	0,8	4 %
Building Electricity	36 000	28 %	5,3	22 %
Space cooling	0	0 %	0,0	0 %
Total electricity	**72 000**	**55** %	**6,1**	**26** %
Space heating	27 277	21 %	8,0	33 %
Hot water	30587	24 %	4,5	19 %
Total heating	**57 864**	**45** %	**12,5**	52 %
Total energy use	**129 864**	**100** %	**18,7**	**78** %

	kg/m2	%	kg equiv CO2/m2	%
Exterior walls incl windows and doors	5,7	26 %	1,1	5 %
Attic	4,0	18 %	2,2	9 %
Basement	1,2	5 %	0,2	1 %
Slabs	9,2	42 %	1,2	5 %
Internal walls	2,0	9 %	0,6	2 %
Total material use ("per year")	**22,0**	**100** %	**5,3**	**22** %
Total yearly impact			**24,0**	**100** %

Estimated Energy Need

City (closest to scrolled cities)	Kiruna
Heated Floor Area (HFA), m2	1 200
Building volume, m3	3 358
Air Change Rate, ACR	0,5
Heat exchange efficiency, η	0,8

kWh/year	Estimation	Real/simulated	Energy source	Fraction	kg equivCO2/m2
Space heating	27 277		Peat	100 %	8,0
			Solar heat	0 %	0,0
Hot water	30587		Peat	50 %	4,5
			Solar heat	50 %	0,0
Ventilation	10 874		Electricity Swedish mix	100 %	0,3
Cooling	0		Nothing	0 %	0,0
Property heating	18000		Peat	100 %	5,3
Household electricity	36000		Electricity Swedish mix	100 %	0,8
Total	122 738	0		Sum	18,9

	Area	U-value	U*A
Ground Floor	300	0,17	50
Attic	390	0,09	36
External wall	785,6	0,17	136
Windows	64	1,00	64
External Doors	24	2,50	60
Total	1 264		295

The LCA results of the consepts A,B and are compared in the table. Consept C has the best values as for the yearly impact/ m^2 and the energy need / m^2. If we compare the results with other similar studies the results are fairly close each other.

Table 24 LCA results

	floor area (m2)	yearly impact/m2 (CO2e)	total energy need (kWh)	energy need /m2 (kWh)
consept A	900	31,6	125703	140
consept B	900	18,8	92436	103
consept C	1200	18,7	122738	102

Table 25 Operational energy for existing multi-storey building (29 apartments), for renovation and for new construction. /22/

	Existing building	Renovation, Low energy building envelope	Renovation, Passive structure building envelope	New, Passive energy building
	MWh/a (kWh/m^2)	MWh/a (kWh/m^2)	MWh/a (kWh/m^2)	MWh (kWh/m^2)
Heating energy (district heating)	241 (130)	93 (50)	70 (38)	18 (10)
Service water heating	94 (51)	94 (51)	94 (51)	94 (51)
Electricity	82 (44)	55 (30)	55 (30)	55 (30)
Total	417	242	219	166

Use of natural resources in building refurbishment and operation phase has an impact to the environment. The impact magnitude depends not only on the insulation but also on the refurbishment case, energy efficiency target, façade materials and roof materials but also on the use of energy raw-materials needed for building operation.

The Consept C what includes an enlargement in form of an extra steel – frame storey proves to be very competitive even from the LCA assesmet point of view.

Viipurinpuisto LCC assesment

Table 26 LCA Economical analysis of energy renovation of a multi-storey house

			CONSEPT A		CONSEPT B		CONSEPT C	
Total room area (m2)			900		900		1200	
		unit						
1. ACQUISITION COST								
Salable gross floor area 350 m2/site	1000	e/m2		0		0	350	-35000
New pipelines 900/1200 m2	500	e/m2	900	450000	900	450000	900	450000
Renewable heating	100 000	e		0		100000		100000
Renewing the windows (m2)	1000	e/m2	0	0	48	48000	48	48000
Refurbishment of the facades (m2)	200	e/m2	0	0	589	117800	589	117800
Ventilation HEE= 0.8 ACR=0.5	500	e/m2	0	0	900	450000	900	450000
Refurbishment of the roof	500	e/m2	0	0	350	175000	0	0
Lift			0	0	0	0		200000
Additional storey with steel	3000	e/m2	0	0	0	0	350	1050000
Energy solutions	25	e/m2	0	0	0	0	1200	30000
TOTAL				450000		1340800		2410800
TOTAL e/ room-m2				500		1490		2009
2. LIFE CYCLE COST IN 50 YEARS								
Acquisition cost				450000		1340800		2410800
Resale value				-100000		-900000		-1950000
Financial cost				50000		50000		50000
Heating savings	-200	e/m2		0	900	-180000	900	-180000
Energy solutions	-50	e/m2		0		0	900	-45000
TOTAL				400000		310800		285800
PAYBACK TIME(50000e/a+ 4%)				10		7		6

The possibilities to remarkably improve energy efficiency in economical ways are directly connected to needs for extensive renovation of an outdated building. However, also separately done changes of windows, refurbishment of facades etc. should lead to the reasonable improvement of energy performance as the case is when we compare the consept A and B.

Development and utilization of renovation concepts means progressive ways of the management of the renovation. The economic impacts of concepts can be summarized as follows.

- significant reduction of energy consumptions and carbon foot print
- reasonable increase of investment cost
- reasonable savings in life cycle costs
- increase of resale value

The most remarkable risks concern management of changes in energy production e.g. Consept A is very vulnerable to the future energy prise rises.

The most durable increase of economic market value by means of extensive renovation can be achieved when the building or the block of buildings is located in a relatively valuable neighbourhood and when the whole neighbourhood is renovated at the same time. In these cases the costs of renovation can be compensated with help of the increase of market value and with saleable extra stories as Consept C shows. The increased use of sustainable building classification methods may also increase the valuation of renovated areas. Effects on economic values of houses and buildings may be significant because of improved performance and because of aesthetical improvement.

There future measures are to develop and implement

- Steel LCA/LCC – Tools
- LCA/ EcoDesign – Modelling
- Building Components: Steel Frame Floor Structures / vibration
- Attic / Foundation Frame systems
- CE - Product Certification

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STEEL SECTOR ISSUES

OPTIMIZING OFFERS IN STEEL CONSTRUCTION PROJECTS

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ABSTRACT

When planning a commercial offer for a steel construction project, the construction project manager needs to optimize between the volume of the preceding planning work and the amount for needed steel in the current construction project.

In ex post analyses it has been shown that in practice it would have been possible to decrease the amount for needed steel in a steel construction project with additional structural planning work, from the point of view of the cost minimum (e.g. Tenhunen 2011). The mathematical optimisation with production and cost functions (assuming certainty) gives solutions which may crucially differ from the actual ex ante choices made by the construction project manager. In this article we examine these differences.

There seems to be at least three important explanations for the differences:
- **The expectation / probability of winning the offer competition.**
- **The collaboration principle with an external planning office.**
- **The used principle of calculating the offered value may vary.**

The main finding in this paper is that there are rational reasons for the construction manager to optimize the preceding planning work on a lower level than the mathematical costs minimum or profit maximum would suggest.

The paper also formulates a mathematical method for optimizing offers in steel construction projects in practice.

INTRODUCTION

It would be a great help to the construction manager to know the costs minimum ex ante. However, the choice of the planning/steel combination pointed out by the costs minimum would not necessarily be the profit maximising choice.

In the following we specify one rational method for the construction manager to optimize the preceding planning work with the volume of steel in the project differently from the mathematical costs minimum or profit maximum. As result, construction managers will have a solution which they may use when formulating their offer for the project.

The formulation in this paper is based simply on the production technology described by the production technology PAR. The example used in the calculations is earlier analysed by Tenhunen (Tenhunen 2011). The analyses will concentrate on the optimization and substitution between the needed amount of steel in the construction project and the preceding planning work for the project offer. The PAR technology was first presented by Tenhunen (Tenhunen 1990).

In this short-term examination, the main unchanged factor is the input level of the company capital equipment, such as machines and facilities.

THE OPTIMISING PROBLEM

There are mainly two kinds of cost groups in a steel construction project. One class of costs is defined by the preceding planning work. These costs will actualize based on the volume of planning hours and the hourly price of planning. The other group of costs is defined by the volume of steel in the project. These costs are defined by the volume of steel and the average price per ton of steel in the project. However, these costs will actualize only in the case when the whole project will actualize, in case the offer will be endorsed. Also the income from the project is not certain depending on whether the offer is endorsed or not.

Formulating the above conditions into the Lagrangean equation $L(K,W,\mu)$ ex ante, we have

[1] $$L(K,W,\mu) = E[V - s*K - w*W)] + \mu*(Y(K,W) - A)$$

where

V = offer value	
K = amount of steel	
W = amount of planning hours	
s = unit price of steel	
w = unit price of planning	
C = total costs	
A = a constant showing the isoquant level	
$E(\,)$	is the statistical expectation operator
$Y(K,W)$	is the production function
$C - s*K - w*W = 0$	is the cost constraint
$Y(K,W) - A = 0$	is the isoquant constraint
μ	is the Lagrange coefficient

The first assumption made here is that the offer value will finally be defined by setting a pricing coefficient $h \geq 1$ to the budgeted costs C of the project.

[2] $$V = h*C \qquad (h>1)$$

Including [2] into [1] we get

[3] $L(K,W,\mu) = E[(h*C - s*K - w*W)] + \mu*(Y(K,W) - A)$

Looking for the maximum of equation [3] we set the partial derivatives to zero.

$$\frac{\partial L}{\partial K} = E[h*s - s] + \mu*(\frac{\partial Y}{\partial K}) = 0$$

$$\frac{\partial L}{\partial W} = E[h*w] - w + \mu*(\frac{\partial Y}{\partial W}) = 0$$

[4] $$\frac{\partial L}{\partial \mu} = Y(K,W) - A = 0$$

From [4] we can solve

$$p(h*s - s) + \mu*(\frac{\partial Y}{\partial K}) = 0$$

$$h*w - w + \mu*(\frac{\partial Y}{\partial W}) = 0$$

[5] $$Y(K,W) - A = 0$$

In Equation [5], p is the probability for winning the offer competition. From [5] one can easily observe that the second partial derivatives of K and W are negative, thus [5] represents a maximum in the mathematical optimisation.

We can solve from the two first equations in [6] as follows

[6] $$p*s*(h\text{-}1) = -\mu*(\frac{\partial Y}{\partial K})$$

$$w*(h\text{-}1) = -\mu*(\frac{\partial Y}{\partial W})$$

Dividing both sides of the both sides of the equation pair in [6] leads to

[7] $$\frac{s*K}{w*W} = \left(\frac{1}{p}\right)*\frac{\theta}{\phi} \qquad (0<p\leq 1)$$

where

[8] $$\theta = \left(\frac{\partial Y}{\partial K}\right)*(\frac{K}{Y})$$

$$\phi = \left(\frac{\partial Y}{\partial W}\right)*(\frac{W}{Y})$$

are the input elasticises of K and W, in the optimum.

It has been shown earlier (e.g. Border 2009; Raval 2011; Varian 2006) that in case of certainty (p=1), the optimum implicates

[9] $$\frac{s*K}{w*W} = \frac{\theta}{\phi}$$

Where the ratio of the input shares of the production factors K and W equals the ratio of the corresponding input elasticises of the production function.

Assuming the production function has constant returns to scale, we have

[10] $\alpha + \beta = 1$

From equations [7] and [9] we can conclude that

- The smaller the probability p of winning the offering competition, the less planning work W is worth inputting to the project ex ante.
- The smaller the probability p of winning the offering competition, the bigger amount of steel K in the project is acceptable ex ante.

COLLABORATION WITH A PLANNING OFFICE

Some steel construction companies collaborate with external entrepreneurial planning offices with the following business principle:

The construction company pays to the planning office a pre-agreed share k ($0<k\leq 1$) of the planning work in advance and the remaining share 1-k only if the offer is the winning one. This makes the probability of winning higher, because the optimal input ratio of the construction company is favorable to a higher amount of planning.

In this case the construction manager optimizes between the inputs based on the smaller planning price k*w.

The maximized Lagrange equation L(K,W,μ) in this case is the following

[11] $L(K,W,\mu) = E[h*C(Y,s,k*w) - s*K - w*k*W)] + \mu*(Y(K,W) - A)$

Looking for the maximum of equation [4] we set the partial derivatives to zero.

$$\frac{\partial L}{\partial K} = E[h * s - s)] + \mu * (\frac{\partial Y}{\partial K}) = 0$$

$$\frac{\partial L}{\partial W} = E[h * k * w - w * k] + \mu * (\frac{\partial Y}{\partial W}) = 0$$

[12] $$\frac{\partial L}{\partial \mu} = Y(K, W) - A = 0$$

From [12] we can solve

$$p * s * (h - 1) + \mu * (\frac{\partial Y}{\partial K}) = 0$$

$$k * w * (h - 1) + \mu * (\frac{\partial Y}{\partial W}) = 0$$

[13] $Y(K,W) - A = 0$

In Equation [13], all terms are equal to Equation [1]. Additionally, k is the share of planning work paid in advance.

The first two equations in (13) can be solved as follows:

[14] $p*s*(h\text{-}1) = -\mu * (\frac{\partial Y}{\partial K})$

$k*w*(h\text{-}1) = -\mu * (\frac{\partial Y}{\partial W})$

Dividing both sides of the equation pair in [14] leads to

[15] $\frac{s*K}{w*W} = \left(\frac{k}{p}\right) * \frac{\theta}{\phi}$ (0<p≤1)

where

[16]

$\theta = \left(\frac{\partial Y}{\partial K}\right) * (\frac{K}{Y})$

$\phi = \left(\frac{\partial Y}{\partial W}\right) * (\frac{W}{Y})$

are the input elasticises of K and W in the optimum.

From the equations above, we can conclude additionally that:

- The smaller share of planning work (k) is paid before an offer is made, the more planning work W is worth inputting to the project ex ante.
- The bigger the share amount of paid planning work (k) before the offer is made, the less planning work W is worth inputting to the project ex ante.

INTERPRETATION OF THE EXAMPLE

This example project has been adopted from Tenhunen (Tenhunen 2011).

In the Lakalaiva steel bridge project in Finland, the most competitive offer for steel beams was done after a remarkable amount of structural planning work. The total weight of the steel container beams of the cross road bridge S6 is (quite exactly) 335 tons. The total value for the ready erected steel construction modules of the construction project (of bridge S6) was about 4.02 million Finnish Marks (670.000 Euros) in the year 1992. This corresponds to some 1 million euros in the year 2011. At the cost level of 1992, the cost for one kg of ready erected steel (in the form of containers steel beams) in the project was 2 Euros/kg.

The planning hours used in the structural steel planning of bridge S6 have been around 800 hours. Thus the planning costs in the realized version have been some FIM 216.000, corresponding to EUR 36.000 in the nominal money level of 1992. The realized total cost for the steel structures (container beams in this case) in the Lakalaiva Cross Road Bridge S6 has been 706.000 Euros in the nominal money level of 1992. Exact numbers are gathered originally from the document 12.8.1992 of the planning company A-Insinöörit Oy (A-Engineers, 1992).

Estimated values in the example (Tenhunen 2011):

The realized values of the variables in the winning offer were:
W = 0.800 thousand hours of planning work
K = 0.335 million kg of steel
w = 45.000 Euros per thousand hours of planning work
s = 2.000.000 Euros per million kg of steel

Utilising the risk aversion principle, the point of costs minimum, within the set of **Par production functions** (including the CD technology), implied the following values to be used (extreme PAR solution)
W_{PAR} = 1.500 thousand hours of planning work at least (extreme Par solution)
K_{PAR} = 0.3033 million kg of steel at most (extreme Par solution)

The estimation results for the PAR production function values for the input elasticises in the costs minimum were
Input elasticity Share of steel = 0.9
Share of planning = 0.1

Substituting the extreme Par solution into the Equation [9] gives

[17] $$\frac{s*K}{w*W} = \frac{\theta}{\phi}$$

[17*] $$\frac{s*K}{w*W} = \frac{2000000*0.3033}{45000*1.5} = \frac{\theta}{\phi} = \frac{0.9}{0.1} = 9$$

The realised solution of the construction manager was, according to [15], was

[18] $$\frac{s*K}{w*W} = \left(\frac{k}{p}\right) * \frac{\theta}{\phi}$$

[18*] $$\frac{s*K}{w*W} = \frac{2000000*0.335}{45000*0.8} = 18{,}6 = \left(\frac{k}{p}\right) * \frac{\theta}{\phi}$$

By substituting the Par production function input elasticity values, we get the Par-estimate (ex post) for the probability to win the offer:

[19] $p_{PA}/k = 48.4\%$

The result above is quite reasonable.

We do not know whether there existed any collaboration in 1992 between the construction company and an external planning office. However, in case yes and k would have been k=0.5, then the Par-estimate for the probability to win the offer:

[20] $p_{PA} = 96.8\%$.

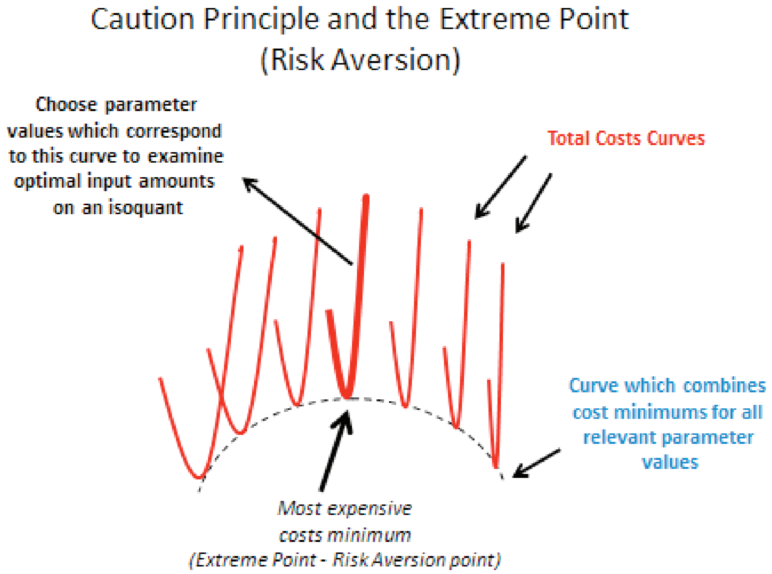
A METHOD FOR OPTIMISING OFFERS

From the normative point of view, the following method for optimising offers would be suggested:

1. Calculate a set of possible cost minimums with iterating parameters within a group of mathematically broad production technologies (e.g. PAR), assuming certainty ex ante. The results will suggest alternative numerical solutions to equation [9], giving each solution a set of exact parameter values. For the calculation you need to have two initial points on one isoquant - real or estimated.

2. Utilise the risk aversion principle in the analysis, by choosing the risk aversion point solution (in case it mathematically exists) from the above sets, which gives the local minimum for one input and a local maximum for the other input, among all the sets of parameters. Fix the parameters.

3. Include your estimates for p and k into equation [15] and solve for K/W. Fix then the corrected ratio (K/W)c – then find a combination of K and W, which gives you the needed production on the chosen isoquant. This needs to be done with the set of parameters given by stage (II). Finally, costs can be calculated with the formula C = w*W + s*K. The Offer Value will be V = h*C from Equation (2), where (c≥1), depending on the target margin.



The example:

1. The set of possible cost minimums within PAR technology in case of the Lakalaiva Bridge was calculated by Tenhunen (Tenhunen 2011).

Nature of inputs	Substitution parameter a	Distribution limit parameter c	Scale parameter A	Cost share limits for structural steel min – max (depending on the input ratio)	The cost share of structural steel in the optimum	Optimal level of structural planning hours	Optimal level of structural steel million kgs	Cost minimum Euros
Supplements	-0,5	-0,4868	235,9	0,4868 - 1	0,644	3900	0,159	494.000
Supplements	-0,1	-0,4593	235,5	0,4593 - 1	0,704	3500	0,187	531.500
Neutral	0	-0,452	235,2	0,726	0,726	3300	0,197	543.200
Complements	+0,2	-0,4374	234,9	0,4374 - 1	0,758	3100	0,2095	558.500
Complements	+1	-0,375	233,7	0,375 - 1	0,844	2300	0,2538	611.100
Complements	+2	-0,2898	231,8	0,2898 - 1	0,875	1800	0,2827	646.420
Complements	+3	-0,205	230,2	0,205 - 1	0,891	1600	0,2958	663.580
Complements	+5	-0,0574	227,4	0,0574 - 1	0,9	1500	0,3033	674.100
Complements	+8	0,0872	225,0	0 – 0,9118	0,878	1800	0,2912	663.450
Complements	+11	0,1643	224,2	0 – 0,8357	0,795	2400	0,2637	635.450
Complements	+15	0,2154	224,3	0 – 0,7846	0,705	2900	0,2340	598.500
Complements	+20	0,2443	225,1	0 – 0.7557	0,756	3100	0,2160	571.500
The realized input combination (not optimum)					0,949	800	0,335	706.000

Table 1. The set of possible cost minimums ex ante within PAR technology (Tenhunen 2011).

2. The risk aversion solution from the above sets is

   Level of planning W = 1.5 thousand hours
   Level of steel K = 0.3033 million kg`s
   (K/W) = 0.2

   The parameters from the risk aversion solution, to be fixed, are c = -0.0574 and a = +5 and A = 227.4.

3. First we assume that p = 0.5 (Probability of winning the offer is 50 %) and the external planning office accepted a pre-agreed share k = 0.7 (70 %). Then

   The corrected ratio $(K/W)_c = (k/p)*(K/W) = 0.28$
   The optimal level of planning is $W \approx 1.14$ thousand hours
   The optimal level of steel is $K \approx 0.314$ million kg`s

A set of other combinations for the optimal offer (with the fixed parameters from point 3) are given in the following table:

	k = 50 % The planning office does the planning with a half-price	k = 70 %	k = 90 %	k = 100 % No agreement with the planning office
p = 1 Winning of the offer is sure	$W \approx 2.87$ $K \approx 0.287$	$W \approx 2.1$ $K \approx 0.294$	$W \approx 1.666$ $K \approx 0.3$	$W \approx 1.5$ $K \approx 0.3033$
p = 0.7 Probability of winning is 70 %	$W \approx 2.1$ $K \approx 0.294$	$W \approx 1.5$ $K \approx 0.3033$	$W \approx 1.225$ $K \approx 0.252$	$W \approx 1.114$ $K \approx 0.315$
p = 0.5 Probability of winning is 50 %	$W \approx 1.5$ $K \approx 0.3033$	$W \approx 1.14$ $K \approx 0.314$	$W \approx 0.91$ $K \approx 0.328$	$W \approx 0.843$ $K \approx 0.331$
p = 0.3 Probability of winning is 30 %	$W \approx 0.966$ $K \approx 0.322$	$W \approx 0.708$ $K \approx 0.347$	$W \approx 0.615$ $K \approx 0.362$	$W \approx 0.562$ $K \approx 0.376$

DISCUSSION

Above we have presented a new method for optimising offers. It is an optimum from a mathematical formulation of the Lagrangean maximising the expected outcome from the steel construction project.

The parameters for the construction manager to decide are

- to negotiate a deal with a planning office (to decide a reasonable value for k) and
- to estimate the probability for winning the offer (to decide a value for p).

The method itself gives a possibility to iterate many values for these parameters.

The results given above also assume that the pricing for the whole offer will be done by setting a pricing coefficient h≥1 to the budgeted costs C of the project. This (h) is handled as constant in the calculations.

The derivation of the results has also utilised the principle of risk aversion. The parameters within the group of PAR production functions have been chosen from the technology representing the technology with highest costs minimum (this is what we have called the risk aversion solution and its parameters).

Some development is needed, before this method can be used in practice. At the moment we lack a proper program for the purpose. However, the optimisation is possible manually, before a proper application is programmed.

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TRANSFORMING SAFETY AND SECURITY FIELD - FUTURE SKILLS

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ABSTRACT

The purpose of this paper is to describe safety and security field and its current transition, future possibilities as well as competence needed to build the future. Many companies and public sector are outsourcing their security services as well. This will further raise the level of competence needs and build up middle management functions accordingly. The paper describes the field, its change, raising business opportunities and competence needs based on the interviews of the company representatives. The most significant change in the field is the technological approach where systems are interoperable and data is transmitted and restored in electronic form. This also means that information safety is of crucial and of increasing importance within and between the companies. As the service solutions are widening, the need for broad-based competence is also required. On the other hand branded specialized services need focused competence where IT, customer service and entrepreneurial skills as well as ability to understand business are essential. Security field`s transition can be seen in two ways. First, risk management's increasing role forms a broad basis for security related issues. Secondly, security is an essential part of management systems and is integrated in other systems such as quality, logistics, environment as well as work safety and welfare. The development seems to be towards holistic services and multi-functional technologies as well as widening the service platform.

Keywords:future competence, safety and security business, business opportunities, cluster formation

INTRODUCTION

Safety and security together with environmental awareness are at the top of the megatrend list with growing interest and importance in the global operating environment and in open societies. Environmental catastrophes, terrorism, pandemic phenomena as well as security in information society are among the most discussed issues lately. Security is of growing importance in company business strategies as well. Companies are users of safety and security services but they are also forming a new type of business field or even a cluster around security issues. Both supply and demand is growing while the structures in the field are transforming. Legislative changes will allow new players from private sector enter traditionally public functions and new alliances and networks will be formed.

Security field is expected to have a yearly growth of around 8 percent. The growth potential lies in vertical networks where actors come from different business fields. The field could be seen as a combination of companies whose business or products are related to or marketed in order to increase safety and security. Business potential should be built based on customer needs rather than on technology orientation (Kupi et al. 2010).

Safety and security field covers a very broad variety of issues: work, environmental and fire safety and protection or personal, company, political, human, information or event security. There is at the moment an ongoing revolution in the security affairs (Harris 2012). Security can be defined broadly or based to the field where it is exercised. Security as a cluster should according to Lanne and Kupi (2007) be seen broadly combined with safety and data protection sectors. Security can be defined to be (Kovacich and Halobozek 2003): freedom from risk or danger, freedom from doubt, anxiety or fear; confidence or anything that gives or assures safety. Harris (2012) combines security domains to state or human areas. Corporate security means overall control of the different sectors of security (Miettinen 2002). Private security is defined to be crime prevention, order maintenance, loss prevention and protection (George and Button 2000).

Risk management is tightly bound to security affairs (Vesterinen 2011). Risks can be reduced by environmental design, technology, office security measures, computer security, transport and distribution security, contingency planning and preparedness for emergencies (Hearnden and Moore 1999). Different risk categories are incidental or intended, passive or active, internal or external and caused by human action or caused by nature (Miettinen 2002). There are three ways of handling risks: elimination, minimizing and transferring (Miettinen 2002). Risk identification (Hearnden and Moore 1999) can be done by identifying loss producing events, analyzing possible operative perils and hazards. A company´s growth brings new requirement for security. At some stage there rises a need for a security program (Berger 1999). Security policy includes data classification, verification and authorization procedures, management policies and other policies regarding, for example, information technology, HR, physical security or incident reporting (Mitnick and Simon 2002).

BACKGROUND

Security threats can be brought up by real world or electrical world, internally or externally or by different groups such as competitors, employees, subcontractors, partners, clients and outsiders (Heljasteet al. 2008). Tiilikainen (2006-2007) defines the segments and specialization fields of security to be banking and insurance, public services, retail, industry services, logistics and transportation, fire services and national defense forces.

Security literature covers mainly the fields of private security, information security, logistics and hotels, restaurants and events (George and Button 2000). They name five areas of private security industry to be manned security services, detention services, security storage or shredding, professional security services and security products.

The fields of information security can be seen to be administrative, technical and operational or related to risk management. Needs of information security are confidentiality, integrity and availability (Kangas 2006-2007). Miettinen (2002) lists the fields as Intellectual Property Rights (IPR), computer programs and equipment, information contents and materials, administration, privacy, information and communication technology and processes.

Securing logistics means delivering goods at right time, right place and in right order. This can create a competitive edge as security affects quality and costs, international logistic chains and job security. It is a holistic process from packaging, choosing the transport form, delivery time to individualizing products throughout the chain (Vesterinen 2011).

Security can be seen at different levels (Tikkanen et al. 2008): global and international security, building EU security, national security, internal security, public organization and company security, private security business and security organizations and cooperation. Security can also be seen based on the specific sector. Legislative and societal based functions are fire security, job safety, environmental protection, information security and contingency planning. Agreements and voluntary insurance define facility safety and crime prevention. Business based areas are facilities, information, foreign operations, production and function safety, crime prevention, security management and contingency planning.

The current characteristics of the security field are expansion, integration, outsourcing and drive for better quality (Palomäki 2011). Anyway the changes seem to be slow compared to other fields both in cost reduction and scaling the solutions for different customer segments. As safety and security field is also strongly bound to its customer fields, the challenges in innovation and networking lie in disconnectedness of the field, the artificial division into different sectors as well as strong technology orientation.

Lanne and Kupi (2007) tried to define security cluster based on cluster model but concludeed that it does not yet exist. The field can be described by different characteristics such as strengths, markets, companies, threats, product characteristics and business fields. Later Kupi et al. (2010) concluded that the traditional cluster model is not an optimum solution for the security sector but innovative networking models should be applied. They also give several recommendations for developing the business field: strengthening the social networks, new integrator for the field, vertical networking and developing business competence, research and education activities, Finland as a security test field, supporting internationalization, auditing the security services and enhancing innovation.

Internationalization and networking study (Kupi et al. 2012) was carried out in Finland in four business areas: senior care, supply chains, situation awareness and built urban environment. Strengths of the Finnish security field lie in technological, GIS and ICT know-how, reputation, neutrality, shipping expertise and reliability. Though, the networks had difficulties in identifying paying customers.

The most potential new business entities recognised (Palomäki 2011) are logistical security, domestic automation and security, solutions for the protection of environment and infrastructure, data security and protection as well as needs connected to health care. It would be fruitful to study business opportunities from B2B perspective as research has not covered safety and security field earlier. The needs of individuals have neither been touched. There is a need for new forums, research and education cooperation as well as supporting activities and structures in the field. More generally, a need for more comprehensive security solutions was discovered. The findings suggest that in order to answer to the customer needs of the security business sector, new security networks and customer orientation are needed.

FUTURE ORIENTATION: TRENDS

High technology and its use have, according to Kovacich and Halobozek (2003), effects on security. They saw five future challenges. Will nation states last? Societies and internet create massive one-to-one communication as a driving force. Dependency on high technology causes higher and more complicated protection needs. Professional and more sophisticated technological solution will emerge and global competition and outsourcing will increase. However security should be seen as a service and support profession. Trends in society, technology, business, global competition, criminal justice system, crime as well as other rapid changes need to be followed in order to understand the risks and meet the needs of the customers. Understanding better the competition could also give competitive edge.

Several trends will affect the business climate of the future. One of the most challenging phenomena is the companies growing bigger and more consolidated (Kupi et al. 2010). The dual use of military and civil technologies will most certainly affect the future security solutions as well. The five megatrends that will have huge impact on security business are urbanization, climate change, growth of developing markets, luxury products and aging together with the rising level of education and earnings.

FUTURE ORIENTATION: SCENARIOS

Three different scenarios for the Finnish security business field were built (Kupi et al. 2010) based on a futures table. Each scenario has different effects on the business climate and related competence needs. First scenario requires small flexible companies functioning mostly locally, the price being the most important competitive edge. The second scenario requires medium sized or big companies or networks of companies working in national or international business climate based on knowledge intensive products and value added for the customers. The third scenario is for sophisticated knowledge intensive network based and customer tailored international expert analyzing services and business solutions, which require high level of expertise and risk taking decision making.

Even though there seems to be different ideas of the future business potential, the common ground can be found in new innovative ways to bring together new competence and network structures (Kupi et al. 2010). The companies can use security products in three different ways; as their main product, integrating security solutions as a part of their customer tailored solution or increasing competitive advantage of their products by adding security features to them. Most customers want to have complete business services or concepts. They are not talking about technologies or systems anymore. Therefore there seems to be business potential for vertical integrators who function in several fields at the same time.

Globally we also need new solutions for important human security aspects such as clean water, waste treatment and sanitation, retainable energy and organic fertilizers for food production.   Effective security (Berger 1999) requires properly trained skilled personnel, effective modern equipment, responsible and understanding interpersonal relationship and the ability to apply these elements properly. Security personnel are mainly described by functions rather than competence: Security officer´s functions are housekeeping, customer care, preventing crime, enforcing rules, administering sanctions, responding to emergencies and gathering and sharing information. Security manager`s functions are personnel protection, access control, asset protection, investigations, risk management as well as other security functions (Button 2008).

Gill et al. (2007) define a model of security managers as modern entrepreneurs. Their role is to make security part of the business processes and integral to all activities. They stress the importance of influencing people as well as policies and objectives, strategic measurements, return on investment and impact on bottom line. Emphasis is on change management. These requirements make business skills more important than security expertise.

METHODS: THEMATIC INTERVIEWS

We studied the change in the security business field in Finland by thematic interviews in spring and summer 2012. Apart from security company representatives, we also interviewed specialists from related services such as logistics, mail and financial services as well as trade and free time activities. The thematic interviews targeted both pure security field companies and companies, in which security is only part of the overall business. Altogether 18 interviews were conducted. The questions covered strategy, change and competence from different angles to foster the futures thinking and dispel situational bounds.

We tried to target persons with insight in security business in their company or specialization field such as private security, logistics, retail, hotels, restaurants and event, public sector services, research as well as information security. Eight thematic questions were partly overlapping. The interview material was written up and the interviewees had a possibility to check the material produced to be in accordance with their views. The produced text was analyzed based on the literature. New approaches, signals and key words were emphasized during the process.

RESULTS

Security field`s transition can be seen in two ways: First risk management´s increasing role forms a broad basis for security related issues. Secondly security is an essential part of management systems and integrated in other systems such as quality, logistics, environment as well as work safety and welfare. The broadening definition of security from traditional guarding and surveillance to integrated services is still under way. But companies have realized new tailor-made customer requirements and need for networking beyond the existing partners. The development seems to be towards holistic services and multi-functional technologies as well as widening the service platform.

External changes affecting corporate strategy

In most companies security is not written on the strategy. Only one company stated, that there mission, vision and values are all about security. Security is mainly dealt at policy level and shown in information security, quality and contingency plans as well as liability. Risk management covers all these fields including security. Security is integrated in quality, environment, logistics, production and work safety and as part of the management system. Security is also seen as a part of the company brand.

When estimating the change, all companies state that the importance of security has increased and this has also affected the production of services. When estimating the future impacts most companies see specialization and tailor-made customer solutions to be the main answer. Procedures will also change: cooperation and expanding partnerships will be needed to meet the customer demands. Technology and tools will develop and new systems and ways of reporting will be developed.

Customer cost savings is the main driver of change. The other often mentioned trend is the increased feeling of insecurity especially of an individual working alone. Other important factors are legislation, internationalization, technological development and changes in the entire business field. Mergers and acquisitions have increased the sizes of companies. The customer interface is changing together with the field of competition. There are also newcomers in the business field such as environment and energy efficiency. The positioning of the companies in the service sector may also be changing. Most companies follow at least some indicators such as crime statistics, incidents, alarms or building volumes. Logistics field, which is heavily regulated, needs own personnel to follow the change in legislation and standards.

Technology enables new integrated solution where all information is digital. New analysis systems may further automate parts of human work. In many fields the prevention of crime and dark business seem to be growing in importance. The development is two-fold. Bigger service entities are demanded alongside specialized products. Information security has become crucial and its role will further increase. New possibilities for virtual security will emerge.

Changes: personnel structure

The speed of change requires anticipation and risk management skills. When dealing with individual customers the ability to identify risks and take them into account is crucial. Almost all interviewees also stress the importance of customer service and business skills. The need for technological skills and competence is at the same time increasing. Information security will require strong ability to manage own IP-space and network architecture especially in financial functions and e-commerce. Subcontracting will widen and contractual skills and understanding of risks and safety rules will become important.

Even though the traditional security companies resist the widening of the competence, there seems to be a clear understanding that it is anyway going to happen. Recruiting profiles based on mostly practical abilities are transforming into more generic and abstract skills of information technology, automation as well as planning and sales. This change will also include new need for middle management. Team leaders need ability for holistic thinking and prioritizing as well as leadership skills. Top management needs tools for measuring and evaluating support functions.

The effects of security on the number of employees seem to be a taboo. Many companies believe in stable situation. Increased outsourcing by the customers has had little effect on the amount of personnel as such, but some customer field companies have experienced slight decrease in personnel. The security companies have experienced slight growth in personnel and are expecting it to continue. The number of security managers and specialists has increased but at the same time number of administrative personnel is decreasing.

Internationalization and new customer segment bring with them new language skills, especially Russian and English. But in security field it is a huge advantage to be able to speak the language of any customer. Communication skills, business skills and language skills as well as multi-skilled workers are needed. The majority think that security skills and competence will be more and more integrated into other functions and job profiles. This makes ability for risk management an integral competence, requiring own tests, audits, and checks. Many also think that no changes are to be expected. In general the field needs to rethink the technology based approach and analyze the integration possibilities. Product based competence and scalability seem to be the key words for competitiveness.

Knowing your customer´s business is rising in importance. This will certainly mean new business concepts, new models of business as well as new combinations and fields of competence. The need for new kind of competence and combinations has been recognized. Digital technology is essential for all. Also development of new customer services and strategies requires new approaches. The work as well as competence profiles will definitely change.

Changes in recruitment and training policies

The personnel are recruited for permanent positions and changes are rare, when talking about experts and technical personnel. The seasonal and regional differences are huge and depend on the branch in question. The traditional security field is an incoming branch for young people and there the change of personnel has always been big. Many students and persons are looking for basic education work here. When recruiting expert level personnel, the competition gets harder. This is especially true when talking about basic security functions and seasonal changes. There exist some recruitment problems in technical as well as specialist fields but companies with good reputation and brand seem to have no problems in finding experts. The requirement for customer orientation is narrowing the recruitment base. Many see the combination of the specialty field expertise with increased security knowledge as the solution.

Recruitment is mainly conducted via the internet. Some companies use hiring services. Cooperation with educational organization is common even though some of the companies do not do it at all. As the entry level education is low, most companies arrange possibilities for degree studies using apprenticeship while working. One company even pays salary during the theory part of the studies. Exchange of knowledge and skills happens mainly if the company is international. Cooperation is most common between Nordic as well as Baltic countries.

Changes in competence needs of personnel

The security field is experiencing a major change as more and more information is in electronic form and domestic as well as company services are looking for new flexible solutions. New service entities will most probably be produced by networks of companies or big multi-field companies

Companies stress the need for new competence both technical and customer service ones. The technical skills concentrate around mobile surroundings, databases and systems. The customer service skills mean developing service culture, customer orientation and service skills. Customers and their business should be known as the security field is part of the brand. The amount of responsibility is increasing in all positions as service business and holistic solutions are developing. One employee could also serve several customers at the same time. Understanding business and ability to recognize individual customer needs is everybody`s business.

The third field of competence needs is related to the new way of doing things. Team and unit leaders need more broad based skills together with management and leadership ones. The work will be more entrepreneurial and remind the one of managing director of a small company. Project and project management skills will be essential.

Companies need better knowledge of the skills of their personnel and systems for competence development. The educational profiles and on-the job-learning models should be revised in order to better support this change. Education level requirements and competence needs are becoming higher also in security field. The legislation is not at the moment supporting this development but leaves room for unhealthy competition. Higher education should give abilities for service production in the situation where there is no demand for security planning or risk analysis in Finland.

CONCLUSIONS

The safety and security field is difficult to define because of its internationality and divergence and as it has not been interesting enough for researchers (Paasonen and Huumonen 2011). The development is heading towards multiservice and holistic service approaches with multiple uses of technologies. Statistics do give clear information about the field. The situation was almost the same with the ICT industry in late 1990's and early 2000's (Manninen and Meristö 2004).

A wide range of different trends and drivers are shaping the safety and security field. Not only political or economic issues are affecting to the field but social, technological and ecological issues as well. Based on literature and our interviews, we can summarize the changing factors of the safety and security field by PESTE analysis, where PESTE stands for political, economic, social, technological and ecological factors.

Political	1. Legislation not complete
	2. Privacy issues
	3. Individual citizen's right and responsibilities
Economic	4. Networked business responsibilities
	5. Holistic service needs
	6. Growing market potential
Social	7. Multiculturalism
	8. New forms of criminality
	9. Aging population
Technological	10. Combination of different technologies
	11. Automation
	12. E- and m-business
Ecological	13. Environmental catastrophes
	14. Environmental awareness
	15. EHS (Environment, Health, Safety)

The description of the safety and security cluster follows the basic idea of Porter's cluster analysis (1990) and it includes a core business field as well as supporting and related businesses. Based on our analysis of interviews made in spring 2012 the Figure 2 illustrates the preliminary draft of the safety and security cluster. The core of the cluster consists of security technology and security services companies. In addition, the core includes multisectoral companies with security services. Outside the core there are supporting and related businesses which include e.g. construction, logistics, insurance and financing as well as companies offering home and free time services. The cluster includes also enablers such as legislation, administration, education and ICT industry.

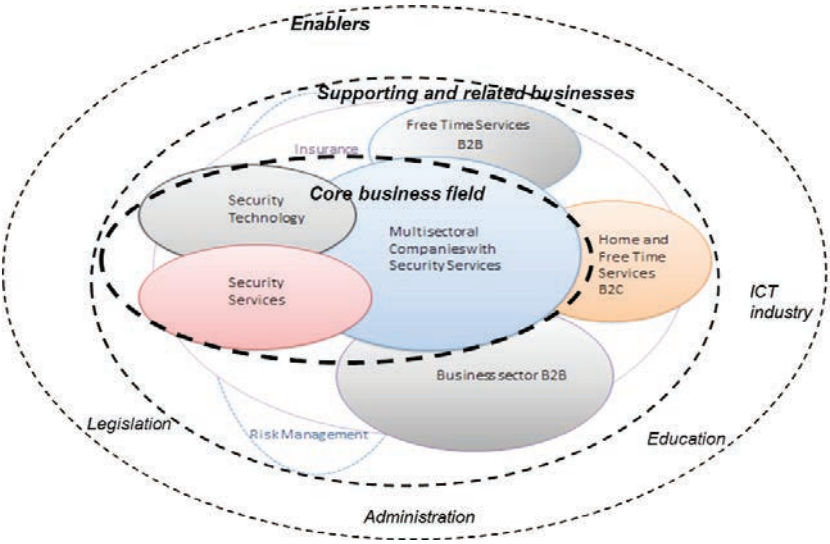

Figure2. Security field: a preliminary draft of the cluster.

As a result we presented a preliminary framework for security cluster structure together with PESTE-factors influencing on its future. Changing requirements for competences and skills to run business and serve customers successfully in the future were analyzed based on the company interviews.

When comparing the ICT and Security clusters there seem to be several similarities but also some clear differences between the two:

Outsourcing, the importance of technological development, service and customer orientation and need for efficiency and cost reductions are similar features. The importance of research and development functions as well as raising level of educational needs were not so clearly identified in security as in ICT sector. Even though there is a general approval of increased competence needs, the legislation and competition seem to inhibit the development. There might even be a polarization of companies in this respect in the future. Also convergence of the field seems to be a fact as it was in ICT. Here also the difference seems to lie in the understanding and concentration in core competence. None of the security field companies had defined their core competence. When looking at the skills and competence needs the broad based understanding of the business and customer needs as well as software skills and project management seem to be equal for both fields. That is also true when discussing the language skills. Missing competence and skills in service production, lack of higher university level education and thus the lack of research in the field seem to make a difference when comparing to ICT cluster. Security is in many ways similarly integrated in other fields and businesses. The major new opportunities could be found in the cross-boarders with user sectors such as healthcare, free time services and individual personalized services as has been the case with ICT as well.

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EVOLUTIONARY APPROACH TO PRODUCT DEVELOPMENT PROJECTS

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ABSTRACT

In this paper the product development activities are approached from the perspective of innovation steering. The whole innovation process is seen as an evolutionary system with irreversible changes in uncertainty and time. A non-linear and interactive nature of most innovative processes is discussed and consequences outlined. On a company level, it is a question about strategy and resources adaptation as well as nurturing such mental models that enable innovative outcomes, and thus the ability to sustain and strengthen competitive advantage of a company. In product development activities, a human system and mental model are at least as important elements as any physical facilities. Ignoring these human systems might cause a failure in an innovative process. Thus a lot of attention must be paid to the structure and functioning of an organization culture. The leadership philosophy as its best includes a combination of bottom-up and top-down practices. With an evolutionary approach a company can move further reactive adaptation by recognizing company´s core competences from a wider perspective and by understanding the company´s purpose instead of just focusing on one objective and a define application or solution.

Keywords: Proactive innovation; Evolutionary product development; Mental facilities

INTRODUCTION

Complexity in product development projects has been on the rise during the last decade or so. A constant pressure on ever increasing agility and lean processes has made it necessary for business management to be ready for non-stop adjustment of the elements involved in the research and development (R&D) activities. Surprisingly, few companies are willing to make constant, significant development and strategy changes voluntarily, but most are forced to. Hence, the pressure for any change is more often external or environmental than internal. Especially successful companies are seeking predictability and stability for their operation, avoiding all instability and chaos. They become easily preoccupied with defending what they have already achieved, and any change is tempered by the concern that there might be more to lose than to gain.

Much concern has been expressed about physical facilities related to R&D&I activities (Research and Development and Innovation), as if there was a strict cause-effect relationship between physical resources allocated to the R&D&I projects and the positive outcomes as a result. Nevertheless, more and more attention is paid now to the innovation drivers as a comprehensive ensemble consisting of all elements that might function as a stimulant for R&D&I system. In this system the mental facilities – or a right kind of state of mind - are taken into account as at least as essential elements as physical ones, and focus has been made on the interaction between the different elements. Challenges are faced especially in the border-crossing projects where the product development is implemented as collaboration between various partners, with competence and know-how scattered along the network. Besides, customer preferences as well as competition situation must be closely monitored to guarantee valid results for every single R&D project. The main dilemma here consists of numerous more or less evolutionary elements that must be consolidated in a way that makes it possible to optimize the resources available for a determined project and to guarantee the satisfactory results for all the parties involved.

Innovation management and product development are inextricably tied together, consisting of the whole range of R&D&I activities. With poor innovation management culture some good results out of product development projects are more like coincidences than purposeful business execution. The same kind of alliance can be found between R&D&I results and competitive advantage of a company. Without any goal-oriented R&D&I strategy the competitive advantage of a company stays on a mortally unsustainable basis. In order to remain on a more sustainable basis, a firm must become a moving target, creating new sources of advantages at least as fast as competitors can replicate the existing ones. In the long run, sustaining advantage also demands that its sources be expanded and upgraded fast enough. Impulses for development, so called innovation drivers, are typically either external factors consisting of forces outside the company, e.g. competition, new technologies, customer requirements, or internal factors arising from the company culture and / or strategy.

The purpose of this paper is to piece together a comprehensive platform where innovative results can be best created and the focus is on the management and leadership activities needed in an effort to generate an agile and committed collaboration among the actors involved in a product development project. We argue that with assertive coordination where the goals of organizations as well as individuals can be reconciled and constantly adapted to the changing operation environment, a commitment to cooperation can materialize and as a result, innovative and target-oriented solutions can be reached.

R&D&I ACTIVITIESAS: AN INTERACTIVE, ADAPTIVE SYSTEM

Evolutionary Theory Applied

Innovation processes are commonly non-linear by nature and require accordingly flexible and adaptive management tools. It is characteristic of any non-linear process that interaction within the system makes it impossible to determine the performance of the whole structure from a study of its isolated parts (Dershin, 2011). There are numerous significant feedback loops in a non-linear process causing a need for immediate changes in any driving factor or initial condition and thus, making outcomes hard or even impossible to foresee beforehand. Pantzar talks about the autocatalytic loop feedback cycle where one item in the system catalyzes another item with positive consequences to the whole system (Pantzar, 1992). However, alterations in initial states of facts (input) do not necessarily mean alterations in certain subsequent states (output). Thus, increase or improvement in some element within the range of innovation facility elements does not automatically improve the results of innovation activity related to a certain goal.

The role of coincidence at various stages must be taken into account as well as the effects of time element during the whole process. There is a constant pressure for change in most of the systems. Every single element within the system is predisposed to change no matter its original purpose, form or goal. The process is proceeding ahead and will be affected by various alternative efforts and aspirations during its function or state. Evolution, as we understand it, is change in uncertainty and time. According to Spencer, evolution is change from an indefinite, incoherent homogeneity to a definite, coherent heterogeneity through continual differentiations (Spencer in Hodgson 1993). The basis of evolutionary approach lies on the classic theoretical analysis of evolutionary economics. Thus, evolution includes a source of self-transformation, novelty and dynamic, irreversible processes as well as human risks (Dosi and Nelson 1994).

In the social context, evolutionary approaches stress the effect of the past in future development, and also emphasize the never-ending change or development. Development is then a continuum of the same progressive procedural and historic nature without a final, pre-defined or ideal goal. In Newtonian physics time is absolute and independent of other physical phenomena, but in general relativity theory, time is stated to be a fourth dimension of space and changes which take place in that dimension are irreversible by nature (Sorli and Sorli, 2004). So it can be stated that the physical time exists only as a stream of change. Quite stable system should be taken as an exception and something to avoid because of its nature of stagnation. Instead of stable ones, chaotic systems represent reality and movement though the direction of movement could be beyond foreseeing.

A chaotic system is most typically defined as an unpredictable system with chance and coincidence strongly shaping the result of the system (Lazlo, 1987). As Laszlo states, chaos is not the opposite of order but its refinement – the subtle, complex, and ultrasensitive form of order. Within each separate process, there are many internal and external variables involved that might coincidentally have different states or values, and thus radically shape the final outcome. As an open and complex system with different levels and sub-systems, the chaotic system gets continuously energy, impulses and information from its environment, which creates a constant need for adjustment and thus, tension within the system (Reunanen *et al.*, 2012). Tension might be slight or stronger, and it is this tension that makes the system bifurcate and move into a new state. Development of a system comes to reality with these small and bigger bifurcations.

R&D&I Platform Creation and Management

Change or evolution is a discontinuous and pulsing process which is influenced by both external, exogenous and internal, endogenous factors. From the evolutionary point of view the internal factors are often more interesting in the situational change than the external ones. On the company level, the ability to modify strategy is often blocked by the fact that a company's past strategy becomes embodied in skills, organizational arrangements, specialized facilities, and a reputation that may be inconsistent with a new, desired one (Porter, 1998). A company may have to destroy its old habits and advantages, too, to create some fresh view and new higher-order advantages to be able to sustain its position on the market. That is obviously one reason why smaller firms or those new to the industry, not bound by history and past investments, often become the innovators and the new leaders on the market.

According to evolutionary product development, changes can be made easily when ideas about new products and services can be found in the area of core competencies and core businesses of a company. But as a prerequisite, the core competencies must be recognized and widely enough analyzed without certain object orientation, so that those competencies could also be adjusted in time towards new products and services required (Prahalad and Hamel, 1990). Companies' capability for change is highlighted by Prahalad and Hamel by a statement that core competence of a company is "... management´s ability to consolidate corporate wide technologies and production skills into competencies that empower individual businesses to adapt quickly to changing opportunities".

On the strategic level, core competencies are bound with prevailing business strategy and thus, in many fields of business, the core competences as well as other key resources cannot be radically or rapidly adjusted (Day, 1990). For instance, a company processing paper from wood or wood cellulose needs some time for production changes, but nevertheless, more agility can be achieved by recognizing the core competencies within a company from wider perspective. Approaching the core competencies from wider perspective as a source of an advantage creator in this paper production case, the purpose of a company could be defined as “refining high quality products and services related to raw materials exploited from photosynthesis” instead of just “paper production”. Hence, widening the scope of core competencies beyond the existing products and production might function as one of the most elementary innovation stimulants clearing the path for the future success.

Pressure for a business strategy adjustment is twofold. On the one hand, there is a pressure coming from the markets and operation environment in general. According to the market-based strategy a company should adapt its resource to the changing competition situation and customer requirements. On the other hand, the business strategy adaptation could be done according to the internal competences and strategic resources under company’s control. In fact, these two approaches must be taken account simultaneously and adaptation of the strategy and resources must proceed hand in hand. Totally market-driven product development strategy might be a norm for certain businesses but mostly it is impossible to implement, e.g. in the paper production case above. Totally resources-driven strategy would no doubt lead to a mismatch on the market and severe problems in the long run.

In Figure 1 below the integrated product development strategy is described. The adjustment of market demands and strategic resources are well synchronized here by management and leadership activities leading to a parallel development of business strategy and strategic resources, which enables a company to execute integrated product development answering the requirements of changing operation environment.

In case of bad synchronization the resources might become outdated or sales plummet with mismatch on the market. Figure 2 below illustrates the situation where neither the business strategy nor the strategic resources are continuously updated based on the evolution taking place. The farther the market demands, business strategy and strategic resources are horizontally from each other the bigger the possibility of mismatch and failure in product development activities is. Practically, it is getting harder and harder to keep the competitive advantage of a company on a sustainable level in this mismatch situation.

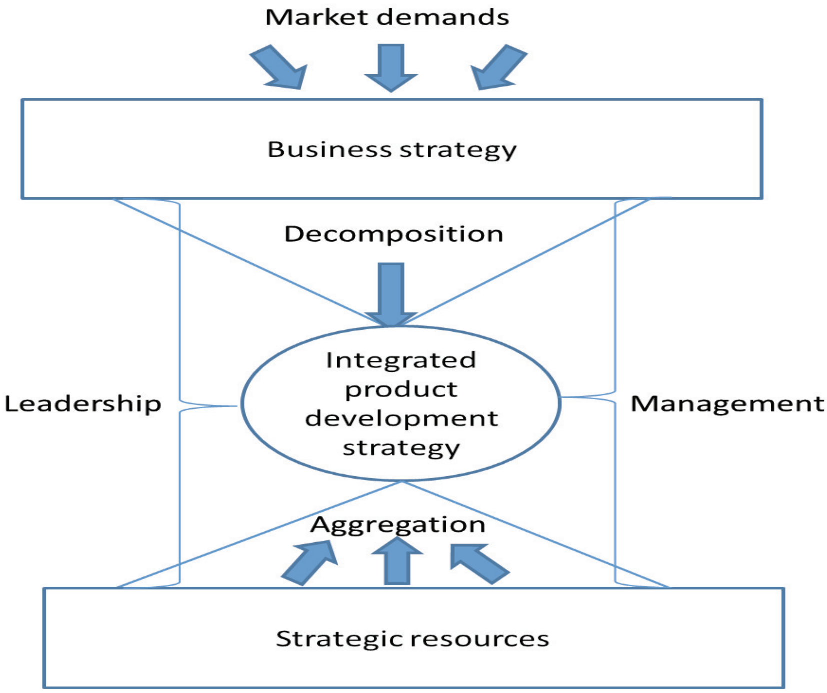

Figure 1. Integrated product development strategy (Barney in Thun, 2008).

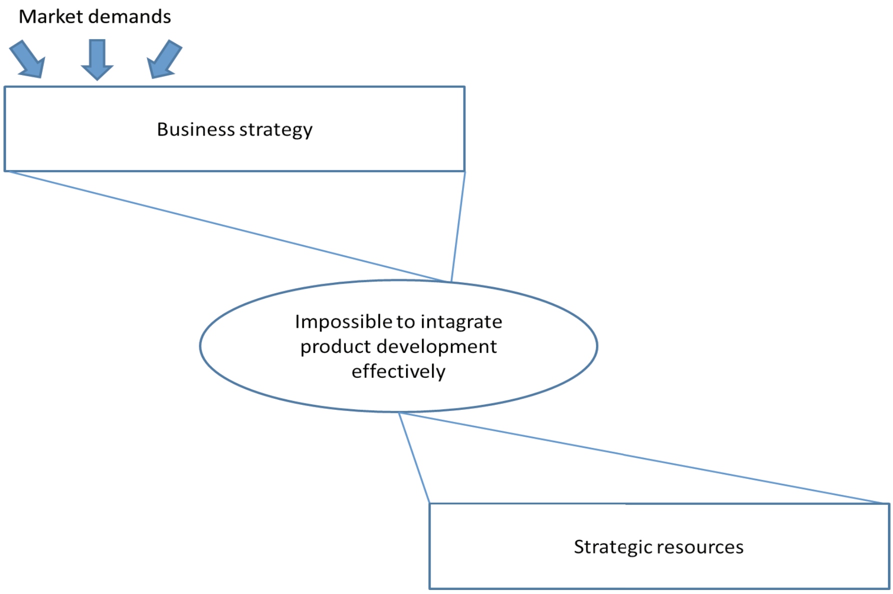

Figure 2. Mismatch based on discontinuous development of strategy and resources.

Further, adaptation of the resources and strategy can be done in a proactive or interactive manner. The best situation is when a company can proactively plan its movements either by tuning or reorientation, based on the nature of a product and product development process in question. For instance, in case of transformational or radical product development, proactive reorientation is needed to achieve a successful outcome. In Figure 3 below different options are described. Reactive adaptations or re-creations are also justified in many situations, e.g. based on feedback or some other immediate need for adaptation observed. Nevertheless, difference in being proactive and reactive is that proactive actions can be taken in a more purposeful manner and it allows more often a possibility for a company to timely achieve the goals set. Reactive actions are mostly taken with a time pressure and, in many cases, by ever shrinking resources because of a proactive competitor already conquering the market.

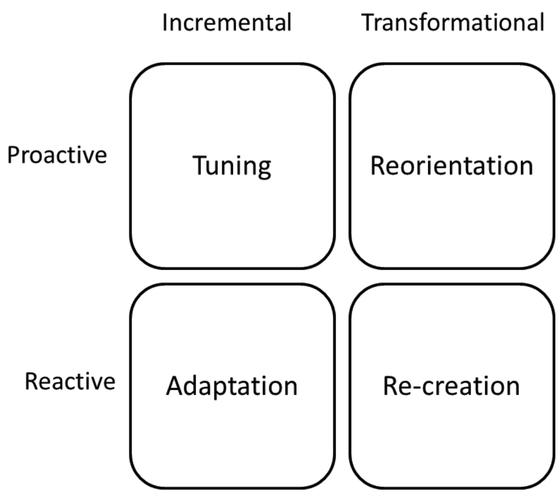

Figure 3.Nature of product development (Hayes, 2010).

On a practical level, in most innovative activities, it is mainly a question about creating favorable circumstances in general and for a specific situation at hand. Nonaka and Konno talk about 'ba' as shared spaces for emerging relationships that provide different kinds of platforms for advancing individual and collective knowledge in innovative activities (Nonaka& Konno, 1998). These platforms can be physical, mental, virtual, or any combination of them, as far as they can be considered serving as a foundation for knowledge creation and refinement. Knowledge separated from "ba" turns into plain information. According to Nonaka and Konno, this kind of "ba" exists at many levels and these levels may be connected with each other to form even greater "ba". From the product development network's point of view, both the composition and coordination of these platforms constitute a critical frame for any project. It can be conceived as a frame in which knowledge is activated as a resource for creation and innovations. Thus, allocating resources and attention to physical infrastructure alone does guarantee the outcome sought after.

Consequently, at least part of all innovation activities is to innovate a human system and the mental models, paying attention to human beings as the very basic building blocks of the company. According to Bhardwaj, these kinds of mental models within the organizational culture must be built using bottom-up philosophy according to which organization culture and management philosophy permit and encourage idea generation among personnel as well as freedom to bring some experiments into effect (Bhardwaj, 2011). Nevertheless, a top-down philosophy is needed as well to steer and control the whole system in a goal-oriented manner. Plain bottom-up philosophy might lead to anarchy and uncontrollable chaos in the project, while plain top-down philosophy might suppress innovativeness and restrain motivation. Thus, a combination of bottom-up and top-down philosophies seems to be the most effective philosophy to apply in this connection.

Latour approaches the innovative networking from the artifact's perspective and questions the relevance of dividing the network elements into human and non-human items. In his actor-network theory (ANT) Latour equalizes all the elements, systems and players within the network (Latour, 2004). ANT takes account all items as critical factors which can either ruin or save the project and hence, according to actor-network theory, the division into human factors and other-than-human factors is irrelevant from the point of view of the final result. However, the active consciousness of these different issues related to both physical and mental facilities in innovative projects might help to tackle the possible setbacks looming.

Eventually, a comprehensive platform for an innovative product development can be best created and maintained by approaching the topic from the evolutionary perspective where both business strategy and strategic resources are simultaneously adapted and synchronized with external and internal requirements and where a human system is taken account as one of the most elementary factor for an innovative outcome.

CONCLUSION

An evolutionary approach in product development activities offers an applicable perspective to examine the processes involved. From the evolutionary point of view the internal factors are often more interesting in the changing situations than the external ones. The ability to modify strategy or resources is often blocked by the company's own rigid models and habits.

A certain amount of controllable chaos might increase the overall quality of innovation activities in an organization. With proactive strategy and resource adaptation the requirements of markets and development of core competences can best be managed, and hence, the competitive advantage sustained. An evolutionary approach to product development is not always incremental, but it is preferably proactive by nature. It could be a useful tool when a company tries to cope with changes in its operation environment by reactive actions. With an evolutionary approach it can go step further by recognizing a company´s core competences timely from a wider perspective and by understanding the company´s purpose instead of just focusing on one objective and application/ solution made from these competencies and resources at a time.

Innovative activity, and product development as an essential part within it, is also a human system and mental model at least as much as the system of tools and physical facilities. Ignoring these human systems that already exist can cause a failure in an innovative process. Thus a lot of attention must be paid to the structure and functioning of an organizational culture. An optimum mixture of bottom-up and top-down philosophies makes the best steering tool for innovation management. Every small step in enhancing the innovation processes must be taken account instead of searching for and registering only the revolutionary ones. The whole business culture and incentive policy should be tuned accordingly.

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ROCKET

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METNET

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